EUROPEAN STANDARD
NORME EUROPÉENNE
EUROPÄISCHE NORM

prEN 1993-1-10 : 2003

17 April 2003

UDC

Descriptors:

English version

Eurocode 3 : Design of steel structures

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

Calcul des structures en acier

Partie 1-10 :

Choix des qualités d'acier vis à vis de la ténacité et des propriétés dans le

Bemessung und Konstruktion von Stahlbauten

Teil 1-10 :

Stahlsortenauswahl im Hinblick auf Bruchzähigkeit und Eigenschaften in Dickenrichtung

Stage 49 draft

CEN

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

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

Ref. No. EN 1993-1-10 : 2003. E

Contents

Page

National annex for EN 1993-1-10

This standard gives alternative procedures, values and recommendations with notes indicating where national choices may have to be made. The National Standard implementing EN 1993-1-10 should have a National Annex containing all Nationally Determined Parameters for the design of steel structures to be constructed in the relevant country.

National choice is allowed in EN 1993-1-10 through clauses:

- 2.2(5)
- 3.1(1)

1 General

1.1 Scope

(1) EN 1993-1-10 contains design guidance for the selection of steel for fracture toughness and for through thickness properties of welded elements where there is a significant risk of lamellar tearing during fabrication.

(2) Section 2 applies to steel grades S 235 to S 690. However section 3 applies to steel grades S 235 to S 460 only.

NOTE EN 1993-1-1 is restricted to steels S235 to S460.

(3) The rules and guidance given in section 2 and 3 assume that the construction will be executed in accordance with EN 1090.

1.2 Normative references

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

NOTE The Eurocodes were published as European Prestandards. The following European Standards which are published or in preparation are cited in normative clauses:

EN 1011-2	Welding. Recommendations for welding of metallic materials: Part 2: Arc welding of ferritic steels
EN 1090	Execution of steel structures
EN 1990	Basis of structural design
EN 1991	Actions on structures
EN 1998	Design provisions for earthquake resistance of structures
EN 10025	Hot rolled products of non-alloy structural steels. Technical delivery conditions
EN 10045-1	Metallic materials - Charpy impact test - Part 1: Test method
EN 10113	Hot-rolled products in weldable fine grain structural steels - Part 1: General delivery conditions; Part 2: Delivery conditions for normalized/normalized rolled steels; Part 3: Delivery conditions for thermomechanical rolled steels”
EN 10137	Plates and wide flats made of high yield strength structural steels in the quenched and tempered or precipitation hardened conditions - Part 1: General delivery conditions; Part 2: Delivery conditions for quenched and tempered steels; Part 3: Delivery conditions for precipitation hardened steels
EN 10155	Structural steels with improved atmospheric corrosion resistance - Technical delivery conditions
EN 10160	Ultrasonic testing of steel flat product of thickness equal or greater than 6 mm (reflection method)
EN 10164	Steel products with improved deformation properties perpendicular to the surface of the product - Technical delivery conditions
EN 10210-1	Hot finished structural hollow sections of non-alloy and fine grain structural steels - Part 1: Technical delivery requirements
EN 10219-1	Cold formed welded structural hollow sections of non-alloy and fine grain steels - Part 1: Technical delivery requirements

1.3 Terms and definitions

1.3.1

K_V-value

The K_V (Charpy V-Notch)-value is the impact energy $A_V(T)$ in Joules [J] required to fracture a Charpy V-notch specimen at a given test temperature T. Steel product standards generally specify that test specimens should not fail at an impact energy lower than 27J at a specified test temperature T.

1.3.2

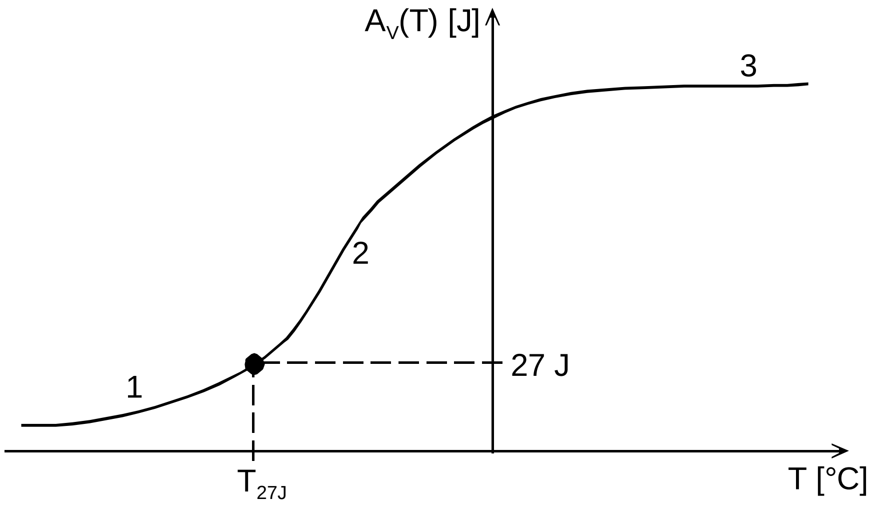
Transition region

The region of the toughness-temperature diagram showing the relationship $A_V(T)$ in which the material toughness decreases with the decrease in temperature and the failure mode changes from ductile to brittle. The temperature values T_{27J} required in the product standards are located in the lower part of this region.

1.3.3

Upper shelf region

The region of the toughness-temperature diagram in which steel elements exhibit elastic-plastic behaviour with ductile modes of failure irrespective of the presence of small flaws and welding discontinuities from fabrication.



1 lower shelf region
2 transition region
3 upper shelf region

Figure 1.1: Relationship between impact energy and temperature

1.3.4

T_{27J}

Temperature at which a minimum energy A_V will not be less than 27J in a Charpy V-notch impact test.

1.3.5

Z-value

The transverse reduction of area in a tensile test of the through-thickness ductility of a specimen, measured as a percentage.

1.3.6

K_{Ic}-value

The plane strain fracture toughness for linear elastic behaviour measured in $N/mm^{3/2}$.

> **NOTE** The two internationally recognized alternative units for the stress intensity factor K are $N/mm^{3/2}$ and $MPa\sqrt{m}$ (ie $MN/m^{3/2}$) where $1\ N/mm^{3/2} = 0{,}032\ MPa\sqrt{m}$.

1.3.7

Degree of cold forming

Permanent strain from cold forming measured as a percentage.

1.4 Symbols

$A_V(T)$ impact energy in Joule [J] in a test at temperature T with Charpy V notch specimen

Z Z-quality [%]

T temperature [°C]

T_{Ed} reference temperature

δ crack tip opening displacement (CTOD) in mm measured on a small specimen to establish its elastic plastic fracture toughness

J elastic plastic fracture toughness value (J-integral value) in N/mm determined as a line or surface integral that encloses the crack front from one crack surface to the other

K_{Ic} elastic fracture toughness value (stress intensity factor) measured in $N/mm^{3/2}$

ε_{cf} degree of cold forming (DCF) in percent

σ_{Ed} stresses accompanying the reference temperature T_{Ed}

2 Selection of materials for fracture toughness

2.1 General

(1) The guidance given in section 2 should be used for the selection of material for new construction. It is not intended to cover the assessment of materials in service. The rules should be used to select a suitable grade of steel from the European Standards for steel products listed in EN 1993-1-1.

(2) The rules are applicable to tension elements, welded and fatigue stressed elements in which some portion of the stress cycle is tensile.

> **NOTE** For elements not subject to tension, welding or fatigue the rules can be conservative. In such cases evaluation using fracture mechanics may be appropriate, see 2.4. Fracture toughness need not be specified for elements only in compression.

(3) The rules should be applied to the properties of materials specified for the toughness quality in the relevant steel product standard. Material of a less onerous grade should not be used even though test results show compliance with the specified grade.

2.2 Procedure

(1) The steel grade shall be selected taking account of the following:

(i) steel material properties:

- yield strength depending on the material thickness $f_y(t)$
- toughness quality expressed in terms of T_{27J} or T_{40J}

(ii) member characteristics:

- member shape and detail
- stress concentrations according to the details in EN 1993-1-9
- element thickness (t)
- appropriate assumptions for fabrication flaws (e.g. as through-thickness cracks or as semi-elliptical surface cracks)

(iii) design situations:

- design value of lowest member temperature
- maximum stresses from permanent and imposed actions derived from the design condition described in (4) below

- residual stress
- assumptions for crack growth from fatigue loading during an inspection interval (if relevant)
- strain rate $\dot{\varepsilon}$ from accidental actions (if relevant)
- degree of cold forming (ε_{cf}) (if relevant)

(2) The permitted thickness of steel elements for fracture should be obtained from section 2.3 and Table 2.1.

(3) Alternative methods may be used to determine the toughness requirement as follows:

- fracture mechanics method:

  In this method the design value of the toughness requirement should not exceed the design value of the toughness property.

- Numerical evaluation:

  This may be carried out using one or more large scale test specimens. To achieve realistic results, the models should be constructed and loaded in a similar way to the actual structure.

(4) The following design condition should be used:

(i) Actions should be appropriate to the following combination:

$$E_d = E\ \{\ A[T_{Ed}]\ \text{"+"}\ \sum G_K\ \text{"+"}\ \psi_1\, Q_{K1}\ \text{"+"}\ \sum \psi_{2,i}\, Q_{Ki}\ \} \tag{2.1}$$

where the leading action A is the reference temperature T_{Ed} that influences the toughness of material of the member considered and might also lead to stress from restraint of movement. $\sum G_K$ are the permanent actions, and $\psi_1\, Q_{K1}$ is the frequent value of the variable load and $\psi_{2i}\, Q_{Ki}$ are the quasi-permanent values of the accompanying variable loads, that govern the level of stresses on the material.

(ii) The combinations factor ψ_1 and ψ_2 should be in accordance with EN 1990.

(iii) The maximum applied stress σ_{Ed} should be the nominal stress at the location of the potential fracture initiation. σ_{Ed} should be calculated as for the serviceability limit state taking into account all combinations of permanent and variable actions as defined in the appropriate part of EN 1991.

NOTE 1 The above combination is considered to be equivalent to an accidental combination, because of the assumption of simultaneous occurrence of lowest temperature, flaw size, location of flaw and material property.

NOTE 2 σ_{Ed} may include stresses from restraint of movement from temperature change.

NOTE 3 As the leading action is the reference temperature T_{Ed} the maximum applied stress σ_{Ed} generally will not exceed 75% of the yield strength.

(5) The reference temperature T_{Ed} at the potential fracture location should be determined using the following expression:

$$T_{Ed} = T_{md} + \Delta T_r + \Delta T_\sigma + \Delta T_R + \Delta T_{\dot{\varepsilon}} + \Delta T_{\varepsilon_{cf}} \tag{2.2}$$

where T_{md} is the lowest air temperature with a specified return period, see EN 1991-1-5

ΔT_r is an adjustment for radiation loss, see EN 1991-1-5

ΔT_σ is the adjustment for stress and yield strength of material, crack imperfection and member shape and dimensions, see 2.4(3)

ΔT_R is a safety allowance, if required, to reflect different reliability levels for different applications

$\Delta T_{\dot{\varepsilon}}$ is the adjustment for a strain rate other than the reference strain rate $\dot{\varepsilon}_0$ (see equation 2.3)

$\Delta T_{\varepsilon_{cf}}$ is the adjustment for the degree of cold forming ε_{cf} (see equation 2.4)

NOTE 1 The safety element ΔT_R to adjust T_{Ed} to other reliability requirements may be given in the National Annex. ΔT_R = 0 °C is recommended, when using the tabulated values according to 2.3.

NOTE 2 In preparing the tabulated values in 2.3 a standard curve has been used for the temperature shift ΔT_σ that envelopes the design values of the stress intensity function [K] from applied stresses σ_{Ed} and residual stresses and includes the Wallin-Sanz-correlation between the stress intensity function [K] and the temperature T. A value of ΔT_σ = 0 °C may be assumed when using the tabulated values according to 2.3.

NOTE 3 The National Annex may give maximum values of the range between T_{Ed} and the test temperature and also the range of σ_{Ed} , to which the validity of values for permissible thicknesses in Table 2.1 may be restricted.

NOTE 4 The application of Table 2.1 may be limited in the National Annex to use of up to S 460 steels.

(6) The reference stresses σ_{Ed} should be determined using an elastic analysis taking into account secondary effects from deformations

2.3 Maximum permitted thickness values

2.3.1 General

(1) Table 2.1 gives the maximum permissible element thickness appropriate to a steel grade, its toughness quality in terms of K_V-value, the reference stress level [σ_{Ed}] and the reference temperature [T_{Ed}].

(2) The tabulated values are based on the following assumptions:

- the values satisfy the reliability requirements of EN 1990 for the general quality of material
- a reference strain rate $\dot{\varepsilon}_0$ = 4×10^{-4}/sec has been used. This covers the dynamic action effects for most transient and persistent design situations. For other strain rates $\dot{\varepsilon}$ (e.g. for impact loads) the tabulated values may be used by reducing T_{Ed} by deducting $\Delta T_{\dot{\varepsilon}}$ given by

$$\Delta T_{\dot{\varepsilon}} = \frac{1440 - f_y(t)}{550} \times \left(\ln \frac{\dot{\varepsilon}}{\dot{\varepsilon}_0} \right)^{1,5} \quad [°C] \tag{2.3}$$

- non cold-formed material with ε_{cf} = 0% has been assumed. To allow for cold forming of non-ageing steels, the tabulated values may be used by adjusting T_{Ed} by deducting $\Delta T_{\varepsilon_{cf}}$ where

$$\Delta T_{\varepsilon_{cf}} = 3 \times \varepsilon_{cf} \quad [°C] \tag{2.4}$$

- the nominal notch toughness values in terms of T_{27J} are based on the following product standards: EN 10025, EN 10113-1 to 3, EN 10137-1 to 3, EN 10155, EN 10210-1, EN 10219-1

  For other values the following correlation has been used

$$\begin{aligned} T_{40J} &= T_{27J} + 10\,[°C] \\ T_{30J} &= T_{27J} + 0\,[°C] \end{aligned} \tag{2.5}$$

- for members subject to fatigue all detail categories for nominal stresses in EN 1993-1-9 are covered

NOTE Fatigue has been taken into account by applying a fatigue load to a member with an assumed initial flaw. The damage assumed is one quarter of the full fatigue damage obtained from EN 1993-1-9. This approach permits the evaluation of a minimum number of “safe periods” between in-service inspections when inspections shall be specified for damage tolerance according to EN 1993-

1-9. The required number [n] of in-service inspections is related to the partial factors γ_{Ff} and γ_{Mf} applied in fatigue design according to EN 1993-1-9 by the expression

$$n = \frac{4}{(\gamma_{Ff}\gamma_{Mf})^m} - 1 ,$$

where m = 5 applies for long life structures such as bridges.

The "safe period" between in-service inspections may also cover the full design life of a structure.

2.3.2 Determination of maximum permissible values of element thickness

(1) Table 2.1 gives the maximum permissible values of element thickness in terms of three stress levels expressed as proportions of the nominal yield strength:

a) $\sigma_{Ed} = 0{,}75\ f_y(t)$ [N/mm²]

b) $\sigma_{Ed} = 0{,}50\ f_y(t)$ [N/mm²] (2.6)

c) $\sigma_{Ed} = 0{,}25\ f_y(t)$ [N/mm²]

where $f_y(t)$ may be determined either from

$$f_y(t) = f_{y,nom} - 0{,}25\frac{t}{t_0} \quad [\mathrm{N/mm^2}]$$

where t is the thickness of the plate in mm

$t_0 = 1$ mm

or taken as R_{eH}-values from the relevant steel standards..

The tabulated values are given in terms of a choice of seven reference temperatures: +10, 0, -10, -20, -30, -40 and -50°C.

Table 2.1: Maximum permissible values of element thickness t in mm

Steel grade	Sub-grade	Charpy energy CVN		Reference temperature T_{Ed} [°C]																				
		at T [°C]	J_{min}	10	0	-10	-20	-30	-40	-50	10	0	-10	-20	-30	-40	-50	10	0	-10	-20	-30	-40	-50
				$\sigma_{Ed} = 0{,}75\ f_y(t)$							$\sigma_{Ed} = 0{,}50\ f_y(t)$							$\sigma_{Ed} = 0{,}25\ f_y(t)$						
S235	JR	20	27	60	50	40	35	30	25	20	90	75	65	55	45	40	35	135	115	100	85	75	65	60
	J0	0	27	90	75	60	50	40	35	30	125	105	90	75	65	55	45	175	155	135	115	100	85	75
	J2	-20	27	125	105	90	75	60	50	40	170	145	125	105	90	75	65	200	200	175	155	135	115	100
S275	JR	20	27	55	45	35	30	25	20	15	80	70	55	50	40	35	30	125	110	95	80	70	60	55
	J0	0	27	75	65	55	45	35	30	25	115	95	80	70	55	50	40	165	145	125	110	95	80	70
	J2	-20	27	110	95	75	65	55	45	35	155	130	115	95	80	70	55	200	190	165	145	125	110	95
	M,N	-20	40	135	110	95	75	65	55	45	180	155	130	115	95	80	70	200	200	190	165	145	125	110
	ML,NL	-50	27	185	160	135	110	95	75	65	200	200	180	155	130	115	95	230	200	200	200	190	165	145
S355	JR	20	27	40	35	25	20	15	15	10	65	55	45	40	30	25	25	110	95	80	70	60	55	45
	J0	0	27	60	50	40	35	25	20	15	95	80	65	55	45	40	30	150	130	110	95	80	70	60
	J2	-20	27	90	75	60	50	40	35	25	135	110	95	80	65	55	45	200	175	150	130	110	95	80
	K2,M,N	-20	40	110	90	75	60	50	40	35	155	135	110	95	80	65	55	200	200	175	150	130	110	95
	ML,NL	-50	27	155	130	110	90	75	60	50	200	180	155	135	110	95	80	210	200	200	200	175	150	130
S420	M,N	-20	40	95	80	65	55	45	35	30	140	120	100	85	70	60	50	200	185	160	140	120	100	85
	ML,NL	-50	27	135	115	95	80	65	55	45	190	165	140	120	100	85	70	200	200	200	185	160	140	120
S460	Q	-20	30	70	60	50	40	30	25	20	110	95	75	65	55	45	35	175	155	130	115	95	80	70
	M,N	-20	40	90	70	60	50	40	30	25	130	110	95	75	65	55	45	200	175	155	130	115	95	80
	QL	-40	30	105	90	70	60	50	40	30	155	130	110	95	75	65	55	200	200	175	155	130	115	95
	ML,NL	-50	27	125	105	90	70	60	50	40	180	155	130	110	95	75	65	200	200	200	175	155	130	115
	QL1	-60	30	150	125	105	90	70	60	50	200	180	155	130	110	95	75	215	200	200	200	175	155	130
S690	Q	0	40	40	30	25	20	15	10	10	65	55	45	35	30	20	20	120	100	85	75	60	50	45
	Q	-20	30	50	40	30	25	20	15	10	80	65	55	45	35	30	20	140	120	100	85	75	60	50
	QL	-20	40	60	50	40	30	25	20	15	95	80	65	55	45	35	30	165	140	120	100	85	75	60
	QL	-40	30	75	60	50	40	30	25	20	115	95	80	65	55	45	35	190	165	140	120	100	85	75
	QL1	-40	40	90	75	60	50	40	30	25	135	115	95	80	65	55	45	200	190	165	140	120	100	85
	QL1	-60	30	110	90	75	60	50	40	30	160	135	115	95	80	65	55	200	200	190	165	140	120	100

NOTE 1 Linear interpolation can be used in applying Table 2.1. Most applications require σ_{Ed} values between $\sigma_{Ed} = 0{,}75\ f_y(t)$ and $\sigma_{Ed} = 0{,}50\ f_y(t)$. $\sigma_{Ed} = 0{,}25\ f_y(t)$ is given for interpolation purposes. Extrapolations beyond the extreme values are not valid.

NOTE 2 For ordering products made of S 690 steels the T_J – values should be specified.

2.4 Evaluation using fracture mechanics

(1) For numerical evaluation using fracture mechanics the toughness requirement and the design toughness property of the materials may be expressed in terms of CTOD values, J-integral values, K_{IC} values, or K_V-values and comparison shall be made using suitable fracture mechanics methods.

(2) The following condition for the reference temperature should be met:

$$T_{Ed} \leq T_{Rd} \qquad (2.7)$$

where T_{Rd} is the temperature at which a safe level of fracture toughness can be relied upon under the conditions being evaluated

(3) The potential failure mechanism should be modelled using a suitable flaw that reduces the net section of the material thus making it more susceptible to failure by fracture of the reduced section. The flaw should meet the following requirements:

- location and the shape should be appropriate for the notch case considered. The fatigue classification tables in EN 1993-1-9 may be used for guidance on appropriate crack positions.
- for members not susceptible to fatigue the size of the flaw should be the maximum likely to have been left uncorrected in inspections carried out to EN 1090. The assumed flaw shall be located at the position of adverse stress concentration.
- for members susceptible to fatigue the size of the flaw should consist of an initial flaw grown by fatigue. The size of the initial crack should be chosen such that it represents the minimum value detectable by the

inspection methods used in accordance with EN 1090. The crack growth from fatigue shall be calculated with an appropriate fracture mechanics model using loads experienced during the design safe working life or an inspection interval (as relevant).

(4) If a structural detail cannot be allocated a specific detail category from EN 1993-1-9 or if more rigorous methods are used to obtain results which are more refined than those given in Table 2.1 then a specific verification should be carried out using actual fracture tests on large scale test specimens.

NOTE The numerical evaluation of the test results may be undertaken using the methodology given in Annex D of EN 1990.

3 Selection of materials for through-thickness properties

3.1 General

(1) The choice of quality class should be selected from Table 3.1 depending on the consequences of lamellar tearing.

Table 3.1: Choice of quality class according to EN 10164

Class	Application of guidance
1	All steel products and all thicknesses listed in European standards for all applications
2	Certain steel products and thicknesses listed in European standards and/or certain listed applications

NOTE The National Annex may choose the relevant class. The use of class 1 is recommended.

(2) Depending on the quality class selected from Table 3.1, either:

– through thickness properties for the steel material should be specified from EN 10164, or

– post fabrication inspection should be used to identify whether lamellar tearing has occurred.

(3) The following aspects should be considered in the selection of steel assemblies or connections to safeguard against lamellar tearing:

– the criticality of the location in terms of applied tensile stress and the degree of redundancy.

– the strain in the through-thickness direction in the element to which the connection is made. This strain arises from the shrinkage of the weld metal as it cools. It is greatly increased where free movement is restrained by other portions of the structure.

– the nature of the joint detail, in particular welded cruciform, tee and corner joints. For example, at the point shown in Figure 3.1, the horizontal plate might have poor ductility in the through-thickness direction. Lamellar tearing is most likely to arise if the strain in the joint acts through the thickness of the material, which occurs if the fusion face is roughly parallel to the surface of the material and the induced shrinkage strain is perpendicular to the direction of rolling of the material. The heavier the weld, the greater is the susceptibility.

– chemical properties of transversely stressed material. High sulfur levels in particular, even if significantly below normal steel product standard limits, can increase the lamellar tearing.

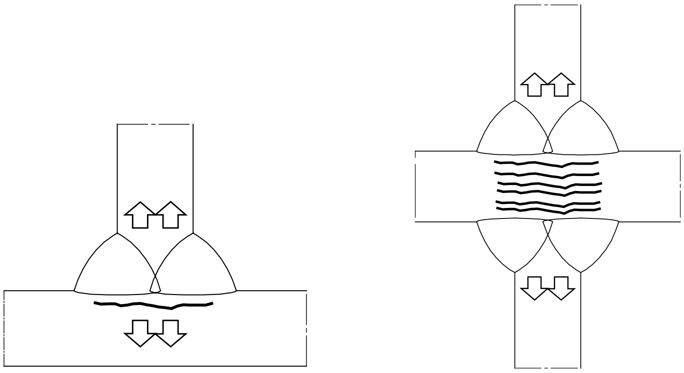


Figure 3.1: Lamellar tearing

(4) The susceptibility of the material should be determined by measuring the through-thickness ductility quality to EN 10164, which is expressed in terms of quality classes identified by Z-values.

NOTE 1 Lamellar tearing is a weld induced flaw in the material which generally becomes evident during ultrasonic inspection. The main risk of tearing is with cruciform, T- and corner joints and with full penetration welds.

NOTE 2 Guidance on the avoidance of lamellar tearing during welding is given in EN 1011-2.

3.2 Procedure

(1) Lamellar tearing may be neglected if the following condition is satisfied:

$$Z_{Ed} \leq Z_{Rd} \tag{3.1}$$

where Z_{Ed} is the required design Z-value resulting from the magnitude of strains from restrained metal shrinkage under the weld beads.

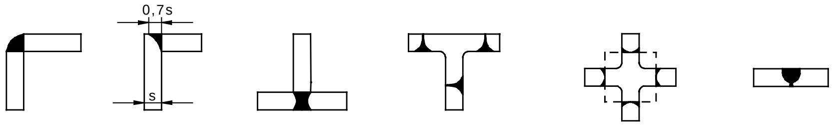
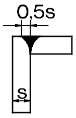
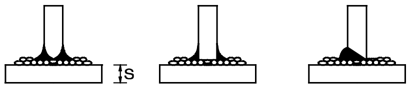
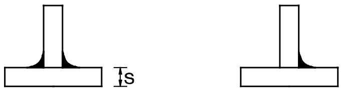
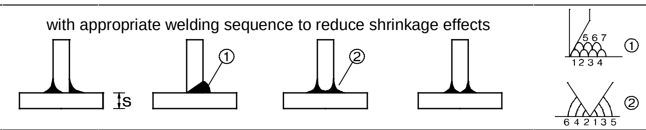
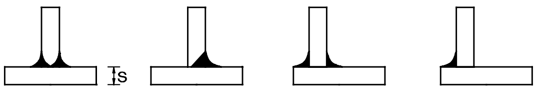
Z_{Rd} is the available design Z-value for the material according to EN 10164.

(2) The required design value Z_{Ed} may be determined using:

$$Z_{Ed} = Z_a + Z_b + Z_c + Z_d + Z_e \tag{3.2}$$

in which Z_a, Z_b, Z_c, Z_d and Z_e are as given in Table 3.2.

Table 3.2: Criteria affecting the target value of Z_{Ed}

a)	Weld depth relevant for straining from metal shrinkage	Effective weld depth a_{eff} (see Figure 3.2) = throat thickn. a of fillet welds		**Z_i**
		$a_{eff} \leq 7$mm	a = 5 mm	**Z_a = 0**
		$7 < a_{eff} \leq 10$mm	a = 7 mm	**Z_a = 3**
		$10 < a_{eff} \leq 20$mm	a = 14 mm	**Z_a = 6**
		$20 < a_{eff} \leq 30$mm	a = 21 mm	**Z_a = 9**
		$30 < a_{eff} \leq 40$mm	a = 28 mm	**Z_a = 12**
		$40 < a_{eff} \leq 50$mm	a = 35 mm	**Z_a = 15**
		$50 < a_{eff}$	a > 35 mm	**Z_a = 15**
b)	Shape and position of welds in T- and cruciform- and corner-connections			**Z_b = -25**
		corner joints		**Z_b = -10**
		single run fillet welds Z_a = 0 or fillet welds with Z_a > 1 with buttering with low strength weld material		**Z_b = -5**
		multi run fillet welds		**Z_b = 0**
		partial and full penetration welds		**Z_b = 3**
		partial and full penetration welds		**Z_b = 5**
		corner joints		**Z_b = 8**
c)	Effect of material thickness *s* on restraint to shrinkage	$s \leq 10$mm		**Z_c = 2***
		$10 < s \leq 20$mm		**Z_c = 4***
		$20 < s \leq 30$mm		**Z_c = 6***
		$30 < s \leq 40$mm		**Z_c = 8***
		$40 < s \leq 50$mm		**Z_c = 10***
		$50 < s \leq 60$mm		**Z_c = 12***
		$60 < s \leq 70$mm		**Z_c = 15***
		$70 < s$		**Z_c = 15***
d)	Remote restraint of shrinkage after welding by other portions of the structure	Low restraint:	Free shrinkage possible (e.g. T-joints)	**Z_d = 0**
		Medium restraint:	Free shrinkage restricted (e.g. diaphragms in box girders)	**Z_d = 3**
		High restraint:	Free shrinkage not possible (e.g. stringers in orthotropic deck plates)	**Z_d = 5**
e)	Influence of preheating	Without preheating		**Z_e = 0**
		Preheating ≥ 100°C		**Z_e = -8**

* May be reduced by 50% for material stressed, in the through-thickness direction, by compression due to predominantly static loads.

Figure 3.2: Effective weld depth a_{eff} for shrinkage

(3) The appropriate Z_{Rd}-class according to EN 10164 may be obtained by applying a suitable classification.

NOTE For classification see EN 1993-1-1 and EN 1993-2 to EN 1993-6.

FINAL DRAFT
prEN 1993-1-1

EUROPEAN STANDARD

NORME EUROPÉENNE

EUROPÄISCHE NORM

December 2003

ICS 91.010.30

Will supersede ENV 1993-1-1:1992

English version

Eurocode 3: Design of steel structures - Part 1-1: General rules and rules for buildings

Eurocode 3: Calcul des structures en acier - Partie 1-1: Règles générales et règles pour les bâtiments

Eurocode 3: Bemessung und Konstruktion von Stahlbauten - Teil 1-1: Allgemeine Bemessungsregeln und Regeln für den Hochbau

This draft European Standard is submitted to CEN members for formal vote. 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.

This draft European Standard was established by CEN in three official versions (English, French, German). A version in any other language made by translation under the responsibility of a CEN member into its own language and notified to the Management Centre has the same status as the official versions.

CEN members are the national standards bodies of Austria, Belgium, Czech Republic, Denmark, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Luxembourg, Malta, Netherlands, Norway, Portugal, Slovakia, Spain, Sweden, Switzerland and United Kingdom.

Warning : This document is not a European Standard. It is distributed for review and comments. It is subject to change without notice and shall not be referred to as a European Standard.

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

Management Centre: rue de Stassart, 36 B-1050 Brussels

Ref. No. prEN 1993-1-1:2003 E

Content

Foreword

This document (prEN 1993-1-1: 2003) has been prepared by Technical Committee CEN/TC 250 "Structural Eurocodes", the secretariat of which is held be BSI.

This document is currently submitted to the Formal Vote.

This document will supersede ENV 1993-1-1.

Background of the Eurocode programme

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

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

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

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

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

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

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

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

Status and field of application of Eurocodes

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

- as a means to prove compliance of building and civil engineering works with the essential requirements of Council Directive 89/106/EEC, particularly Essential Requirement N°1 - Mechanical resistance and stability - and Essential Requirement N°2 - Safety in case of fire;
- as a basis for specifying contracts for construction works and related engineering services;
- as a framework for drawing up harmonised technical specifications for construction products (ENs and ETAs)

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

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

National Standards implementing Eurocodes

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

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

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

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

There is a need for consistency between the harmonised technical specifications for construction products and the technical rules for works[4]. Furthermore, all the information accompanying the CE Marking of the

---

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

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

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

construction products which refer to Eurocodes should clearly mention which Nationally Determined Parameters have been taken into account.

Additional information specific to EN 1993-1

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

EN 1993-1 is the first of six parts of EN 1993 – Design of Steel Structures. It gives generic design rules intended to be used with the other parts EN 1993-2 to EN 1993-6. It also gives supplementary rules applicable only to buildings.

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

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

EN 1993-1 is intended for use by

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

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

National annex for EN 1993-1-1

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

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

- 2.3.1(1)
- 3.1(2)
- 3.2.1(1)
- 3.2.2(1)
- 3.2.3(1)
- 3.2.3(3)B
- 3.2.4(1)B
- 5.2.1(3)
- 5.2.2(8)
- 5.3.2(3)
- 5.3.2(11)
- 5.3.4(3)
- 6.1(1)B
- 6.1(1)
- 6.3.2.2(2)
- 6.3.2.3(1)
- 6.3.2.3(2)
- 6.3.2.4(1)B
- 6.3.2.4(2)B
- 6.3.3(5)
- 6.3.4(1)
- 7.2.1(1)B
- 7.2.2(1)B
- 7.2.3(1)B
- BB.1.3(3)B

1 General

1.1 Scope

1.1.1 Scope of Eurocode 3

(1) Eurocode 3 applies to the design of buildings and civil engineering works in steel. It complies with the principles and requirements for the safety and serviceability of structures, the basis of their design and verification that are given in EN 1990 – Basis of structural design.

(2) Eurocode 3 is 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.

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

– EN 1990 “Basis of structural design”

– EN 1991 “Actions on structures”

– ENs, ETAGs and ETAs for construction products relevant for steel structures

– EN 1090 “Execution of steel structures – Technical requirements”

– EN 1992 to EN 1999 when steel structures or steel components are referred to

(4) Eurocode 3 is subdivided in various parts:

EN 1993-1 Design of Steel Structures : General rules and rules for buildings.

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

EN 1993-3 Design of Steel Structures : Towers, masts and chimneys.

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

EN 1993-5 Design of Steel Structures : Piling.

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

(5) EN 1993-2 to EN 1993-6 refer to the generic rules in EN 1993-1. The rules in parts EN 1993-2 to EN 1993-6 supplement the generic rules in EN 1993-1.

(6) EN 1993-1 “General rules and rules for buildings” comprises:

EN 1993-1-1 Design of Steel Structures : General rules and rules for buildings.

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

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

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

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

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

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

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

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

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

EN 1993-1-11 Design of Steel Structures : Design of structures with tension components made of steel.

1.1.2 Scope of Part 1.1 of Eurocode 3

(1) EN 1993-1-1 gives basic design rules for steel structures with material thicknesses t ≥ 3 mm. It also gives supplementary provisions for the structural design of steel buildings. These supplementary provisions are indicated by the letter “B” after the paragraph number, thus ( )B.

NOTE For cold formed thin gauge members and plate thicknesses t < 3 mm see EN 1993-1-3.

(2) The following subjects are dealt with in EN 1993-1-1:

Section 1: General

Section 2: Basis of design

Section 3: Materials

Section 4: Durability

Section 5: Structural analysis

Section 6: Ultimate limit states

Section 7: Serviceability limit states

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

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

(5) Section 4 gives general rules for durability.

(6) Section 5 refers to the structural analysis of structures, in which the members can be modelled with sufficient accuracy as line elements for global analysis.

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

(8) Section 7 gives rules for serviceability.

1.2 Normative references

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

1.2.1 General reference standards

EN 1090	Execution of steel structures – Technical requirements
EN ISO 12944	Paints and varnishes – Corrosion protection of steel structures by protective paint systems
EN 1461	Hot dip galvanised coatings on fabricated iron and steel articles – specifications and test methods

1.2.2 Weldable structural steel reference standards

EN 10025-1:2002	Hot-rolled products of structural steels - Part 1: General delivery conditions.
EN 10025-2:2002	Hot-rolled products of structural steels - Part 2: Technical delivery conditions for non-alloy structural steels.
EN 10025-3:2002	Hot-rolled products of structural steels - Part 3: Technical delivery conditions for normalized / normalized rolled weldable fine grain structural steels.

EN 10025-4:2002	Hot-rolled products of structural steels - Part 4: Technical delivery conditions for thermomechanical rolled weldable fine grain structural steels.
EN 10025-5:2002	Hot-rolled products of structural steels - Part 5: Technical delivery conditions for structural steels with improved atmospheric corrosion resistance.
EN 10025-6:2002	Hot-rolled products of structural steels - Part 6: Technical delivery conditions for flat products of high yield strength structural steels in the quenched and tempered condition.
EN 10164:1993	Steel products with improved deformation properties perpendicular to the surface of the product - Technical delivery conditions.
EN 10210-1:2002	Hot finished structural hollow sections of non-alloy and fine grain structural steels – Part 1: Technical delivery requirements.
EN 10219-1:2002	Cold formed hollow sections of structural steel - Part 1: Technical delivery requirements.

1.3 Assumptions

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

- fabrication and erection complies with EN 1090

1.4 Distinction between principles and application rules

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

1.5 Terms and definitions

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

(2) The following terms and definitions are used in EN 1993-1-1 with the following meanings:

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

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

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

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

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

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

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

1.5.7
shear lag effect
non-uniform stress distribution in wide flanges due to shear deformation; it is taken into account by using a reduced "effective" flange width in safety assessments

1.5.8
capacity design
design method for achieving the plastic deformation capacity of a member by providing additional strength in its connections and in other parts connected to it

1.5.9
uniform member
member with a constant cross-section along its whole length

1.6 Symbols

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

(2) Additional symbols are defined where they first occur.

NOTE Symbols are ordered by appearance in EN 1993-1-1. Symbols may have various meanings.

Section 1

x-x axis along a member

y-y axis of a cross-section

z-z axis of a cross-section

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

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

b width of a cross section

h depth of a cross section

d depth of straight portion of a web

t_w web thickness

t_f flange thickness

r radius of root fillet

r_1 radius of root fillet

r_2 toe radius

t thickness

Section 2

P_k nominal value of the effect of prestressing imposed during erection

G_k nominal value of the effect of permanent actions

X_K characteristic values of material property

X_n nominal values of material property

R_d design value of resistance

R_k characteristic value of resistance

γ_M general partial factor

γ_{Mi} particular partial factor

γ_{Mf} partial factor for fatigue

η conversion factor

a_d design value of geometrical data

Section 3

f_y yield strength

f_u ultimate strength

R_{eh} yield strength to product standards

R_m ultimate strength to product standards

A_0 original cross-section area

ε_y yield strain

ε_u ultimate strain

Z_{Ed} required design Z-value resulting from the magnitude of strains from restrained metal shrinkage under the weld beads.

Z_{Rd} available design Z-value

E modulus of elasticity

G shear modulus

ν Poisson's ratio in elastic stage

α coefficient of linear thermal expansion

Section 5

α_{cr} factor by which the design loads would have to be increased to cause elastic instability in a global mode

F_{Ed} design loading on the structure

F_{crit} elastic critical buckling load for global instability mode based on initial elastic stiffnesses

H_{Ed} design value of the horizontal reaction at the bottom of the storey to the horizontal loads and fictitious horizontal loads

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

$\delta_{H,Ed}$ horizontal displacement at the top of the storey, relative to the bottom of the storey

h storey height

$\bar{\lambda}$ non dimensional slenderness

N_{Ed} design value of the axial force

ϕ global initial sway imperfection

ϕ_0 basic value for global initial sway imperfection

α_h reduction factor for height h applicable to columns

h height of the structure

α_m reduction factor for the number of columns in a row

m number of columns in a row

e_0 maximum amplitude of a member imperfection

L member length

η_{init} amplitude of elastic critical buckling mode

η_{cr} shape of elastic critical buckling mode

$e_{0,d}$ design value of maximum amplitude of an imperfection

M_{Rk} characteristic moment resistance of the critical cross section

N_{Rk} characteristic resistance to normal force of the critical cross section

α imperfection factor

$EI\,\eta''_{cr}$ bending moment due to η_{cr} at the critical cross section

χ reduction factor for the relevant buckling curve

$\alpha_{ult,k}$ minimum force amplifier to reach the characteristic resistance without taking buckling into account

α_{crit} minimum force amplifier to reach the elastic critical buckling

q equivalent force per unit length

δ_q in-plane deflection of a bracing system

q_d equivalent design force per unit length

M_{Ed} design bending moment

k factor for $e_{0,d}$

ε strain

σ stress

$\sigma_{com,Ed}$ maximum design compressive stress in an element

ℓ length

ε coefficient depending on f_y

c width or depth of a part of a cross section

α portion of a part of a cross section in compression

ψ stress or strain ratio

k_σ plate buckling coefficient

d outer diameter of circular tubular sections

Section 6

γ_{M0} partial factor for resistance of cross-sections whatever the class is

γ_{M1} partial factor for resistance of members to instability assessed by member checks

γ_{M2} partial factor for resistance of cross-sections in tension to fracture

$\sigma_{x,Ed}$ design value of the local longitudinal stress

$\sigma_{z,Ed}$ design value of the local transverse stress

τ_{Ed} design value of the local shear stress

N_{Ed} design normal force

$M_{y,Ed}$ design bending moment, y-y axis

$M_{z,Ed}$ design bending moment, z-z axis

N_{Rd} design values of the resistance to normal forces

$M_{y,Rd}$ design values of the resistance to bending moments, y-y axis

$M_{z,Rd}$ design values of the resistance to bending moments, z-z axis

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

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

n number of holes extending in any diagonal or zig-zag line progressively across the member or part of the member

d diameter of hole

e_N shift of the centroid of the effective area A_{eff} relative to the centre of gravity of the gross cross section

ΔM_{Ed} additional moment from shift of the centroid of the effective area A_{eff} relative to the centre of gravity of the gross cross section

A_{eff} effective area of a cross section

$N_{t,Rd}$ design values of the resistance to tension forces

$N_{pl,Rd}$ design plastic resistance to normal forces of the gross cross-section

$N_{u,Rd}$ design ultimate resistance to normal forces of the net cross-section at holes for fasteners

A_{net} net area of a cross section

$N_{net,Rd}$ design plastic resistance to normal forces of the net cross-section

$N_{c,Rd}$ design resistance to normal forces of the cross-section for uniform compression

$M_{c,Rd}$ design resistance for bending about one principal axis of a cross-section

W_{pl} plastic section modulus

$W_{el,min}$ minimum elastic section modulus

$W_{eff,min}$ minimum effective section modulus

A_f area of the tension flange

$A_{f,net}$ net area of the tension flange

V_{Ed} design shear force

$V_{c,Rd}$ design shear resistance

$V_{pl,Rd}$ plastic design shear resistance

A_v shear area

η factor for shear area

S first moment of area

I second moment of area

A_w area of a web

A_f area of one flange

T_{Ed} design value of total torsional moments

T_{Rd} design resistance to torsional moments

$T_{t,Ed}$ design value of internal St. Venant torsion

$T_{w,Ed}$ design value of internal warping torsion

$\tau_{t,Ed}$ design shear stresses due to St. Venant torsion

$\tau_{w,Ed}$ design shear stresses due to warping torsion

$\sigma_{w,Ed}$ design direct stresses due to the bimoment B_{Ed}

B_{Ed} bimoment

$V_{pl,T,Rd}$ reduced design plastic shear resistance making allowance for the presence of a torsional moment

ρ reduction factor to determine reduced design values of the resistance to bending moments making allowance for the presence of shear forces

$M_{V,,Rd}$ reduced design values of the resistance to bending moments making allowance for the presence of shear forces

$M_{N,,Rd}$ reduced design values of the resistance to bending moments making allowance for the presence of normal forces

n ratio of design normal force to design plastic resistance to normal forces of the gross cross-section

a ratio of web area to gross area

α parameter introducing the effect of biaxial bending

β parameter introducing the effect of biaxial bending

$e_{N,y}$ shift of the centroid of the effective area A_{eff} relative to the centre of gravity of the gross cross section (y-y axis)

$e_{N,z}$ shift of the centroid of the effective area A_{eff} relative to the centre of gravity of the gross cross section (z-z axis)

$W_{eff,min}$ minimum effective section modulus

$N_{b,Rd}$ design buckling resistance of a compression member

χ reduction factor for relevant buckling mode

Φ value to determine the reduction factor χ

a_0, a, b, c, d class indexes for buckling curves

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

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

λ_1 slenderness value to determine the relative slenderness

$\bar{\lambda}_T$ relative slenderness for torsional or torsional-flexural buckling

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

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

$M_{b,Rd}$ design buckling resistance moment

χ_{LT} reduction factor for lateral-torsional buckling

Φ_{LT} value to determine the reduction factor χ_{LT}

α_{LT} imperfection factor

$\bar{\lambda}_{LT}$ non dimensional slenderness for lateral torsional buckling

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

$\bar{\lambda}_{LT,0}$ plateau length of the lateral torsional buckling curves for rolled sections

β correction factor for the lateral torsional buckling curves for rolled sections

$\chi_{LT,mod}$ modified reduction factor for lateral-torsional buckling

f modification factor for χ_{LT}

k_c correction factor for moment distribution

ψ ratio of moments in segment

L_c length between lateral restraints

$\bar{\lambda}_f$ equivalent compression flange slenderness

i_{fz} radius of gyration of compression flange about the minor axis of the section

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

$A_{eff,f}$ effective area of compression flange

$A_{eff,w,c}$ effective area of compressed part of web

$\bar{\lambda}_{c0}$ slenderness parameter

$k_{f\ell}$ modification factor

ΔM_y moments due to the shift of the centroidal y-y axis

ΔM_z moments due to the shift of the centroidal z-z axis

χ_y reduction factor due to flexural buckling (y-y axis)

χ_z reduction factor due to flexural buckling (z-z axis)

k_{yy} interaction factor

k_{yz} interaction factor

k_{zy} interaction factor

k_{zz} interaction factor

$\bar{\lambda}_{op}$ global non dimensional slenderness of a structural component for out-of-plane buckling

χ_{op} reduction factor for the non-dimensional slenderness $\bar{\lambda}_{op}$

$\alpha_{ult,k}$ minimum load amplifier of the design loads to reach the characteristic resistance of the most critical cross section

$\alpha_{cr,op}$ minimum amplifier for the in plane design loads to reach the elastic critical resistance with regard to lateral or lateral torsional buckling

N_{Rk} characteristic value of resistance to compression

$M_{y,Rk}$ characteristic value of resistance to bending moments about y-y axis

$M_{z,Rk}$ characteristic value of resistance to bending moments about z-z axis

Q_m local force applied at each stabilised member at the plastic hinge locations

L_{stable} stable length of segment

L_{ch} buckling length of chord

h_0 distance of centrelines of chords of a built-up column

a distance between restraints of chords

α angle between axes of chord and lacings

i_{min} minimum radius of gyration of single angles

A_{ch} area of one chord of a built-up column

$N_{ch,Ed}$ design chord force in the middle of a built-up member

M^{I}_{Ed} design value of the maximum moment in the middle of the built-up member

I_{eff} effective second moment of area of the built-up member

S_v shear stiffness of built-up member from the lacings or battened panel

n number of planes of lacings

A_d area of one diagonal of a built-up column

d length of a diagonal of a built-up column

A_V area of one post (or transverse element) of a built-up column

I_{ch} in plane second moment of area of a chord

I_b in plane second moment of area of a batten

μ efficiency factor

i_y radius of gyration (y-y axis)

Annex A

C_{my} equivalent uniform moment factor

C_{mz} equivalent uniform moment factor

C_{mLT} equivalent uniform moment factor

μ_y factor

μ_z factor

$N_{cr,y}$ elastic flexural buckling force about the y-y axis

$N_{cr,z}$ elastic flexural buckling force about the z-z axis

C_{yy} factor

C_{yz} factor

C_{zy} factor

C_{zz} factor

w_y factor

w_z factor

n_{pl} factor

$\bar{\lambda}_{max}$ maximum of $\bar{\lambda}_y$ and $\bar{\lambda}_z$

b_{LT} factor

c_{LT} factor

d_{LT} factor

e_{LT} factor

ψ_y ratio of end moments (y-y axis)

$C_{my,0}$ factor

$C_{mz,0}$ factor

a_{LT} factor

I_T St. Venant torsional constant

I_y second moment of area about y-y axis

$M_{i,Ed}(x)$ maximum first order moment

$|\delta_x|$ maximum member displacement along the member

Annex B

α_s factor

α_h factor

C_m equivalent uniform moment factor

Annex AB

γ_G partial factor for permanent loads

G_k characteristic value of permanent loads

γ_Q partial factor for variable loads

Q_k characteristic value of variable loads

Annex BB

$\bar{\lambda}_{eff,v}$ effective slenderness ratio for buckling about v-v axis

$\bar{\lambda}_{eff,y}$ effective slenderness ratio for buckling about y-y axis

$\bar{\lambda}_{eff,z}$ effective slenderness ratio for buckling about z-z axis

L system length

L_{cr} buckling length

S shear stiffness provided by sheeting

I_w warping constant

$C_{\vartheta,k}$ rotational stiffness provided by stabilising continuum and connections

K_{υ} factor for considering the type of analysis

K_{ϑ} factor for considering the moment distribution and the type of restraint

$C_{\vartheta R,k}$ rotational stiffness provided by the stabilising continuum to the beam assuming a stiff connection to the member

$C_{\vartheta C,k}$ rotational stiffness of the connection between the beam and the stabilising continuum

$C_{\vartheta D,k}$ rotational stiffness deduced from an analysis of the distorsional deformations of the beam cross sections

L_m stable length between adjacent lateral restraints

L_k stable length between adjacent torsional restraints

L_s stable length between a plastic hinge location and an adjacent torsional restraint

C_1 modification factor for moment distribution

C_m modification factor for linear moment gradient

C_n modification factor for non-linear moment gradient

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

B_0 factor

B_1 factor

B_2 factor

η ratio of critical values of axial forces

i_s radius of gyration related to centroid of restraining member

β_t ratio of the algebraically smaller end moment to the larger end moment

R_1 moment at a specific location of a member

R_2 moment at a specific location of a member

R_3 moment at a specific location of a member

R_4 moment at a specific location of a member

R_5 moment at a specific location of a member

R_E maximum of R_1 or R_5

R_s maximum value of bending moment anywhere in the length L_y

c taper factor

h_h additional depth of the haunch or taper

h_{max} maximum depth of cross-section within the length L_y

h_{min} minimum depth of cross-section within the length L_y

h_s vertical depth of the un-haunched section

L_h length of haunch within the length L_y

L_y length between restraints

1.7 Conventions for member axes

(1) The convention for member axes is:

x-x - along the member

y-y - axis of the cross-section

z-z - axis of the cross-section

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

– generally:

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

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

– for angle sections:

  y-y - axis parallel to the smaller leg

  z-z - axis perpendicular to the smaller leg

– where necessary:

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

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

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

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

> **NOTE** All rules in this Eurocode relate to principal axis properties, which are generally defined by the axes y-y and z-z but for sections such as angles are defined by the axes u-u and v-v.

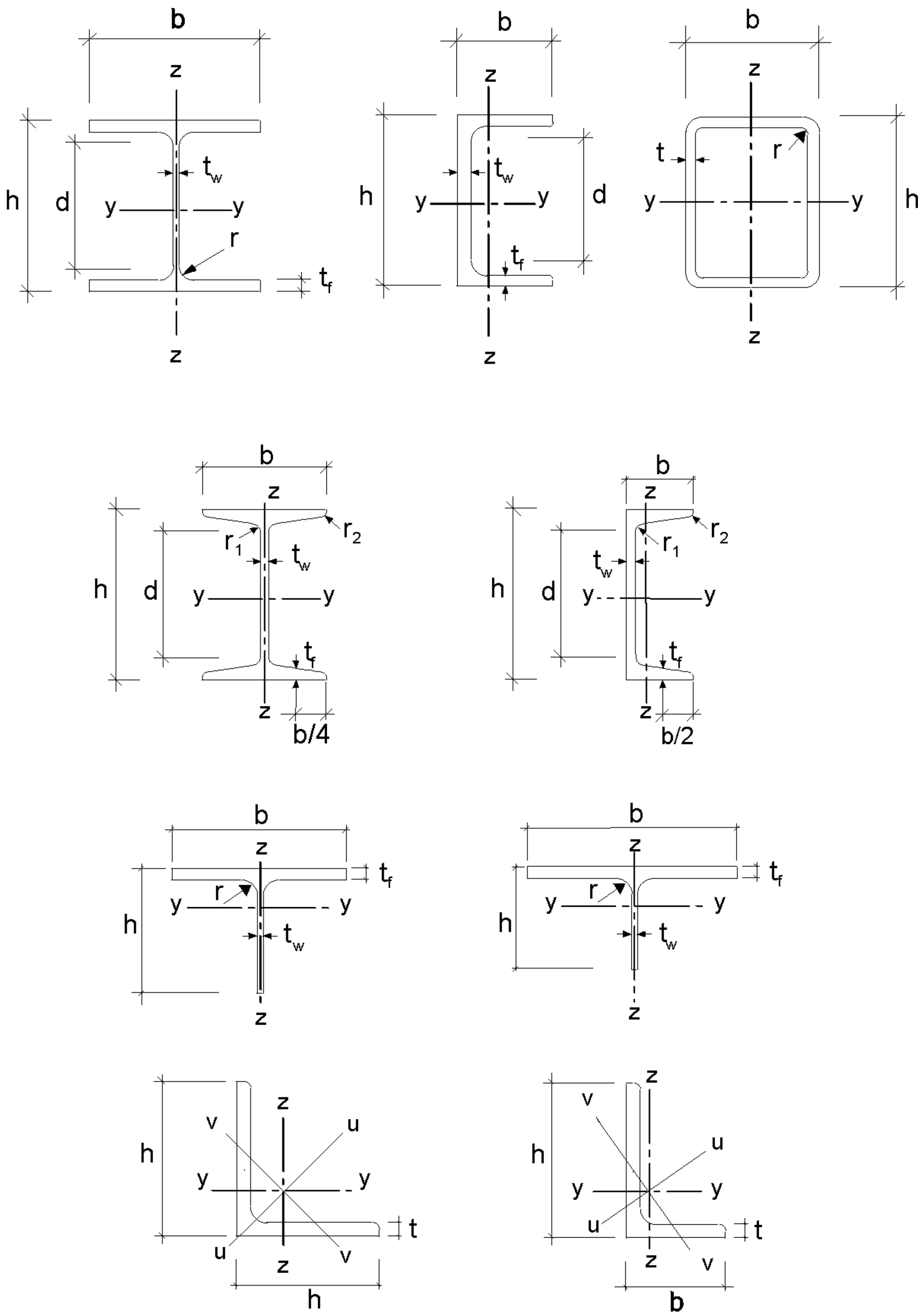


Figure 1.1: Dimensions and axes of sections

2 Basis of design

2.1 Requirements

2.1.1 Basic requirements

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

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

(3) The basic requirements of EN 1990 section 2 shall be deemed be satisfied where limit state design is used in conjunction with the partial factor method and the load combinations given in EN 1990 together with the actions given in EN 1991.

(4) The rules for resistances, serviceability and durability given in the various parts of EN 1993 should be applied.

2.1.2 Reliability management

(1) Where different levels of reliability are required, these levels should preferably be achieved by an appropriate choice of quality management in design and execution, according to EN 1990 Annex C and EN 1090.

2.1.3 Design working life, durability and robustness

2.1.3.1 General

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

- designed against corrosion by means of
  - suitable surface protection (see EN ISO 12944)
  - the use of weathering steel
  - the use of stainless steel (see EN 1993-1-4)
- detailed for sufficient fatigue life (see EN 1993-1-9)
- designed for wearing
- designed for accidental actions (see EN 1991-1-7)
- inspected and maintained.

2.1.3.2 Design working life for buildings

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

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

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

2.1.3.3 Durability for buildings

(1)B To ensure durability, buildings and their components should either be designed for environmental actions and fatigue if relevant or else protected from them.

(2)B The effects of deterioration of material, corrosion or fatigue where relevant should be taken into account by appropriate choice of material, see EN 1993-1-4 and EN 1993-1-10, and details, see EN 1993-1-9, or by structural redundancy and by the choice of an appropriate corrosion protection system.

(3)B If a building includes components that need to be replaceable (e.g. bearings in zones of soil settlement), the possibility of their safe replacement should be verified as a transient design situation.

2.2 Principles of limit state design

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

(2) The resistances specified in this Eurocode Part may therefore be used where the conditions for materials in section 3 are met.

2.3 Basic variables

2.3.1 Actions and environmental influences

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

NOTE 1 The National Annex may define actions for particular regional or climatic or accidental situations.

NOTE 2B For proportional loading for incremental approach, see Annex AB.1.

NOTE 3B For simplified load arrangement, see Annex AB.2.

(2) The actions to be considered in the erection stage should be obtained from EN 1991-1-6.

(3) Where the effects of predicted absolute and differential settlements need to be considered, best estimates of imposed deformations should be used.

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

(5) Fatigue actions not defined in EN 1991 should be determined according to Annex A of EN 1993-1-9.

2.3.2 Material and product properties

(1) Material properties for steels and other construction products and the geometrical data to be used for design should be those specified in the relevant ENs, ETAGs or ETAs unless otherwise indicated in this standard.

2.4 Verification by the partial factor method

2.4.1 Design values of material properties

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

2.4.2 Design values of geometrical data

(1) Geometrical data for cross-sections and systems may be taken from product standards hEN or drawings for the execution to EN 1090 and treated as nominal values.

(2) Design values of geometrical imperfections specified in this standard are equivalent geometric imperfections that take into account the effects of:

– geometrical imperfections of members as governed by geometrical tolerances in product standards or the execution standard,
– structural imperfections due to fabrication and erection,
– residual stresses,
– variation of the yield strength.

2.4.3 Design resistances

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

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

where R_k is the characteristic value of the particular resistance determined with characteristic or nominal values for the material properties and dimensions

γ_M is the global partial factor for the particular resistance

NOTE For the definitions of η_1, η_i, X_{k1}, X_{ki} and a_d see EN 1990.

2.4.4 Verification of static equilibrium (EQU)

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

2.5 Design assisted by testing

(1) The resistances R_k in this standard have been determined using Annex D of EN 1990.

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

$$R_k = R_d \gamma_{Mi} \tag{2.2}$$

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

γ_{Mi} are recommended partial factors.

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

NOTE 2 For characteristic values of fatigue strength and partial factors γ_{Mf} for fatigue see EN 1993-1-9.

NOTE 3 For characteristic values of toughness resistance and safety elements for the toughness verification see EN 1993-1-10.

(3) Where resistances R_k for prefabricated products shall be determined from tests, the procedure in (2) should be followed.

3 Materials

3.1 General

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

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

NOTE For other steel material and products see National Annex.

3.2 Structural steel

3.2.1 Material properties

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

a) either by adopting the values $f_y = R_{eh}$ and $f_u = R_m$ direct from the product standard

b) or by using the simplification given in Table 3.1

NOTE The National Annex may give the choice.

3.2.2 Ductility requirements

(1) For steels a minimum ductility is required that should be expressed in terms of limits for:

- the ratio f_u / f_y of the specified minimum ultimate tensile strength f_u to the specified minimum yield strength f_y;
- the elongation at failure on a gauge length of $5{,}65\sqrt{A_o}$ (where A_0 is the original cross-sectional area);
- the ultimate strain ε_u, where ε_u corresponds to the ultimate strength f_u .

NOTE The limiting values of the ratio f_u / f_y , the elongation at failure and the ultimate strain ε_u may be defined in the National Annex. The following values are recommended:

- $f_u / f_y \geq 1{,}10$;
- elongation at failure not less than 15%;
- $\varepsilon_u \geq 15\varepsilon_y$, where ε_y is the yield strain ($\varepsilon_y = f_y / E$).

(2) Steel conforming with one of the steel grades listed in Table 3.1 should be accepted as satisfying these requirements.

3.2.3 Fracture toughness

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

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

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

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

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

(4) For selecting steels for members with hot dip galvanized coatings see EN 1064.

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

Standard and steel grade	Nominal thickness of the element t [mm]			
	$t \leq 40$ mm		40 mm $< t \leq 80$ mm	
	f_y [N/mm^2]	f_u [N/mm^2]	f_y [N/mm^2]	f_u [N/mm^2]
EN 10025-2				
S 235	235	360	215	360
S 275	275	430	255	410
S 355	355	510	335	470
S 450	440	550	410	550
EN 10025-3				
S 275 N/NL	275	390	255	370
S 355 N/NL	355	490	335	470
S 420 N/NL	420	520	390	520
S 460 N/NL	460	540	430	540
EN 10025-4				
S 275 M/ML	275	370	255	360
S 355 M/ML	355	470	335	450
S 420 M/ML	420	520	390	500
S 460 M/ML	460	540	430	530
EN 10025-5				
S 235 W	235	360	215	340
S 355 W	355	510	335	490
EN 10025-6				
S 460 Q/QL/QL1	460	570	440	550

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

Standard and steel grade	Nominal thickness of the element t [mm]			
	$t \leq 40$ mm		$40 \text{ mm} < t \leq 65$ mm	
	f_y [N/mm^2]	f_u [N/mm^2]	f_y [N/mm^2]	f_u [N/mm^2]
EN 10210-1				
S 235 H	235	360	215	340
S 275 H	275	430	255	410
S 355 H	355	510	335	490
S 275 NH/NLH	275	390	255	370
S 355 NH/NLH	355	490	335	470
S 420 NH/NHL	420	540	390	520
S 460 NH/NLH	460	560	430	550
EN 10219-1				
S 235 H	235	360		
S 275 H	275	430		
S 355 H	355	510		
S 275 NH/NLH	275	370		
S 355 NH/NLH	355	470		
S 460 NH/NLH	460	550		
S 275 MH/MLH	275	360		
S 355 MH/MLH	355	470		
S 420 MH/MLH	420	500		
S 460 MH/MLH	460	530		

3.2.4 Through-thickness properties

(1) Where steel with improved through-thickness properties is necessary according to EN 1993-1-10, steel according to the required quality class in EN 10164 should be used.

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

NOTE 2B Particular care should be given to welded beam to column connections and welded end plates with tension in the through-thickness direction.

NOTE 3B The National Annex may give the relevant allocation of target values Z_{Ed} according to 3.2(3) of EN 1993-1-10 to the quality class in EN 10164. The allocation in Table 3.2 is recommended for buildings:

Table 3.2: Choice of quality class according to EN 10164

Target value of Z_{Ed} according to EN 1993-1-10	Required value of Z_{Rd} according to EN 10164
$Z_{Ed} \leq 10$	—
$10 < Z_{Ed} \leq 20$	Z 15
$20 < Z_{Ed} \leq 30$	Z 25
$Z_{Ed} > 30$	Z 35

3.2.5 Tolerances

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

(2) For welded components the tolerances given in EN 1090 should be applied.

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

3.2.6 Design values of material coefficients

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

- modulus of elasticity $E = 210\,000\ \mathrm{N/mm^2}$
- shear modulus $G = \dfrac{E}{2(1+\nu)} \approx 81\,000\ \mathrm{N/mm^2}$
- Poisson's ratio in elastic stage $\nu = 0{,}3$
- coefficient of linear thermal expansion $\alpha = 12 \times 10^{-6}$ per°C (for T ≤ 100 °C)

NOTE For calculating the structural effects of unequal temperatures in composite concrete-steel structures to EN 1994 the coefficient of linear thermal expansion is taken as $\alpha = 10 \times 10^{-6}$ per °C.

3.3 Connecting devices

3.3.1 Fasteners

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

3.3.2 Welding consumables

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

3.4 Other prefabricated products in buildings

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

4 Durability

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

(2) The means of executing the protective treatment undertaken off-site and on-site shall be in accordance with EN 1090.

NOTE EN 1090 lists the factors affecting execution that need to be specified during design.

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

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

a) Members supporting lifting appliances or rolling loads

b) Members subject to repeated stress cycles from vibrating machinery

c) Members subject to wind-induced vibrations

d) Members subject to crowd-induced oscillations

(5) For elements that cannot be inspected an appropriate corrosion allowance should be included.

(6)B Corrosion protection does not need to be applied to internal building structures, if the internal relative humidity does not exceed 80%.

5 Structural analysis

5.1 Structural modelling for analysis

5.1.1 Structural modelling and basic assumptions

(1) Analysis shall be based upon calculation models of the structure that are appropriate for the limit state under consideration.

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

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

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

5.1.2 Joint modelling

(1) The effects of the behaviour of the joints on the distribution of internal forces and moments within a structure, and on the overall deformations of the structure, may generally be neglected, but where such effects are significant (such as in the case of semi-continuous joints) they should be taken into account, see EN 1993-1-8.

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

- simple, in which the joint may be assumed not to transmit bending moments;
- continuous, in which the stiffness and/or the resistance of the joint allow full continuity of the members to be assumed in the analysis;
- semi-continuous, in which the behaviour of the joint needs to be taken into account in the analysis

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

5.1.3 Ground-structure interaction

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

NOTE EN 1997 gives guidance for calculation of soil-structure interaction.

5.2 Global analysis

5.2.1 Effects of deformed geometry of the structure

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

– first-order analysis, using the initial geometry of the structure or

– second-order analysis, taking into account the influence of the deformation of the structure.

(2) The effects of the deformed geometry (second-order effects) shall be considered if they increase the action effects significantly or modify significantly the structural behaviour.

(3) First order analysis may be used for the structure, if the increase of the relevant internal forces or moments or any other change of structural behaviour caused by deformations can be neglected. This condition may be assumed to be fulfilled, if the following criterion is satisfied:

$$\alpha_{cr} = \frac{F_{cr}}{F_{Ed}} \geq 10 \quad \text{for elastic analysis}$$
$$\alpha_{cr} = \frac{F_{cr}}{F_{Ed}} \geq 15 \quad \text{for plastic analysis} \tag{5.1}$$

where α_{cr} is the factor by which the design loading would have to be increased to cause elastic instability in a global mode

F_{Ed} is the design loading on the structure

F_{cr} is the elastic critical buckling load for global instability mode based on initial elastic stiffnesses

NOTE A greater limit for α_{cr} for plastic analysis is given in equation (5.1) because structural behaviour may be significantly influenced by non linear material properties in the ultimate limit state (e.g. where a frame forms plastic hinges with moment redistributions or where significant non linear deformations from semi-rigid connections occur). Where substantiated by more accurate approaches the National Annex may give a lower limit for α_{cr} for certain types of frames.

(4)B Portal frames with shallow roof slopes and beam-and-column type plane frames in buildings may be checked for sway mode failure with first order analysis if the criterion (5.1) is satisfied for each storey. In these structures α_{cr} may be calculated using the following approximative formula, provided that the axial compression in the beams or rafters is not significant:

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) \tag{5.2}$$

where H_{Ed} is the design value of the horizontal reaction at the bottom of the storey to the horizontal loads and fictitious horizontal loads, see 5.3.2(7)

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

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

h is the storey height

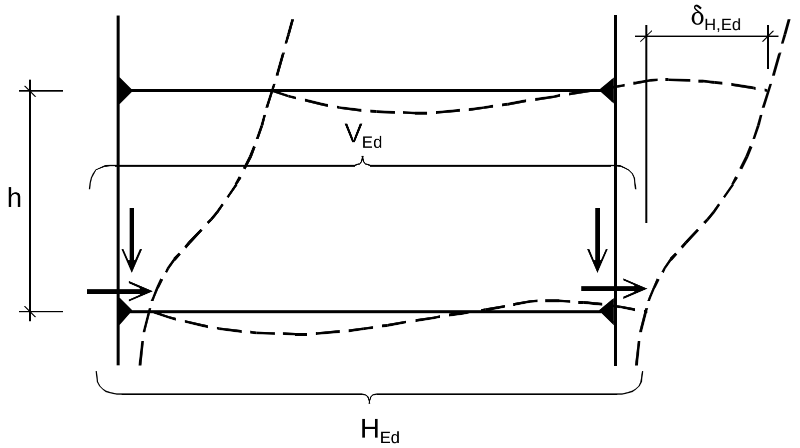


Figure 5.1: Notations for 5.2.1(2)

NOTE 1B For the application of (4)b in the absence of more detailed information a roof slope may be taken to be shallow if it is not steeper that 1:2 (26°).

NOTE 2B For the application of (4)B in the absence of more detailed information the axial compression in the beams or rafters may be assumed to be significant if

$$\bar{\lambda} \geq 0{,}3\sqrt{\frac{A\ f_y}{N_{Ed}}} \qquad (5.3)$$

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

$\bar{\lambda}$ is the inplane non dimensional slenderness calculated for the beam or rafters considered as hinged at its ends of the system length measured along the beams of rafters.

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

NOTE For rolled sections and welded sections with similar dimensions shear lag effects may be neglected.

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

5.2.2 Structural stability of frames

(1) If according to 5.2.1 the influence of the deformation of the structure has to be taken into account (2) to (6) should be applied to consider these effects and to verify the structural stability.

(2) The verification of the stability of frames or their parts should be carried out considering imperfections and second order effects.

(3) According to the type of frame and the global analysis, second order effects and imperfections may be accounted for by one of the following methods:

a) both totally by the global analysis,

b) partially by the global analysis and partially through individual stability checks of members according to 6.3,

c) for basic cases by individual stability checks of equivalent members according to 6.3 using appropriate buckling lengths according to the global buckling mode of the structure.

(4) Second order effects may be calculated by using an analysis appropriate to the structure (including step-by-step or other iterative procedures). For frames where the first sway buckling mode is predominant first order elastic analysis should be carried out with subsequent amplification of relevant action effects (e.g. bending moments) by appropriate factors.

(5)B For single storey frames designed on the basis of elastic global analysis second order sway effects due to vertical loads may be calculated by increasing the horizontal loads H_{Ed} (e.g. wind) and equivalent loads $V_{Ed}\ \phi$ due to imperfections (see 5.3.2(7)) and other possible sway effects according to first order theory by the factor:

$$\frac{1}{1-\frac{1}{\alpha_{cr}}} \tag{5.4}$$

provided that $\alpha_{cr} \geq 3{,}0$,

where α_{cr} may be calculated according to (5.2) in 5.2.1(4)B, provided that the roof slope is shallow and that the axial compression in the beams or rafters is not significant as defined in 5.2.1(4)B.

NOTE B For $\alpha_{cr} < 3{,}0$ a more accurate second order analysis applies.

(6)B For multi-storey frames second order sway effects may be calculated by means of the method given in (5)B provided that all storeys have a similar

- distribution of vertical loads and
- distribution of horizontal loads and
- distribution of frame stiffness with respect to the applied storey shear forces.

NOTE B For the limitation of the method see also 5.2.1(4)B.

(7) In accordance with 5.2.2(3) the stability of individual members should be checked according to the following:

a) If second order effects in individual members and relevant member imperfections (see 5.3.4) are totally accounted for in the global analysis of the structure, no individual stability check for the members according to 6.3 is necessary.

b) If second order effects in individual members or certain individual member imperfections (e.g. member imperfections for flexural and/or lateral torsional buckling, see 5.3.4) are not totally accounted for in the global analysis, the individual stability of members shall be checked according to the relevant criteria in 6.3 for the effects not included in the global analysis. This verification should take account of end moments and forces from the global analysis of the structure, including global second order effects and global imperfections (see 5.3.2) when relevant and may be based on a buckling length equal to the system length

(8) Where the stability of a frame is assessed by a check with the equivalent column method according to 6.3 the buckling length values should be based on a global buckling mode of the frame accounting for the stiffness behaviour of members and joints, the presence of plastic hinges and the distribution of compressive forces under the design loads. In this case internal forces to be used in resistance checks are calculated according to first order theory without considering imperfections.

NOTE The National Annex may give information on the scope of application.

5.3 Imperfections

5.3.1 Basis

(1) Appropriate allowances shall be incorporated in the structural analysis to cover the effects of imperfections, including residual stresses and geometrical imperfections such as lack of verticality, lack of

straightness, lack of flatness, lack of fit and any minor eccentricities present in joints of the unloaded structure.

(2) Equivalent geometric imperfections, see 5.3.2 and 5.3.3, should be used, with values which reflect the possible effects of all type of imperfections unless these effects are included in the resistance formulae for member design, see section 5.3.4.

(3) The following imperfections should be taken into account:

a) global imperfections for frames and bracing systems

b) local imperfections for individual members

5.3.2 Imperfections for global analysis of frames

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

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

(3) For frames sensitive to buckling in a sway mode the effect of imperfections should be allowed for in frame analysis by means of an equivalent imperfection in the form of an initial sway imperfection and individual bow imperfections of members. The imperfections may be determined from:

a) global initial sway imperfections, see Figure 5.2:

$$\phi = \phi_0 \, \alpha_h \, \alpha_m \qquad (5.5)$$

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

α_h is the reduction factor for height h applicable to columns:

$$\alpha_h = \frac{2}{\sqrt{h}} \text{ but } \frac{2}{3} \leq \alpha_h \leq 1{,}0$$

h is the height of the structure in meters

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

m is the number of columns in a row including only those columns which carry a vertical load N_{Ed} not less than 50% of the average value of the column in the vertical plane considered

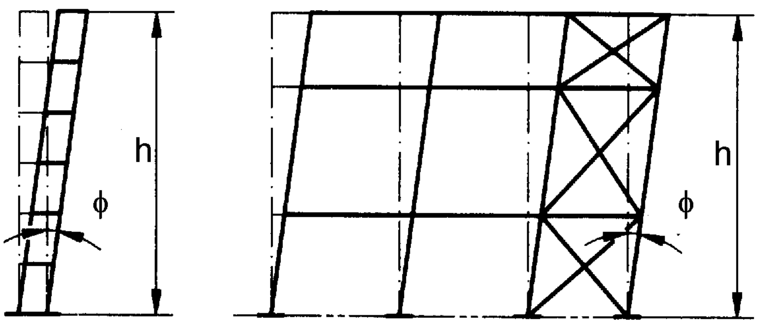


Figure 5.2: Equivalent sway imperfections

b) relative initial local bow imperfections of members for flexural buckling

$$e_0 / L \qquad (5.6)$$

where L is the member length

NOTE The values e_0 / L may be chosen in the National Annex. Recommended values are given in Table 5.1.

Table 5.1: Design values of initial bow imperfection e_0 / L

Buckling curve acc. to Table 6.1	elastic analysis e_0 / L	plastic analysis e_0 / L
a_0	1 / 350	1 / 300
a	1 / 300	1 / 250
b	1 / 250	1 / 200
c	1 / 200	1 / 150
d	1 / 150	1 / 100

(4)B For building frames sway imperfections may be disregarded where

$$H_{Ed} \geq 0{,}15\ V_{Ed} \tag{5.7}$$

(5)B For the determination of horizontal forces to floor diaphragms the configuration of imperfections as given in Figure 5.3 should be applied, where ϕ is a sway imperfection obtained from (5.5) assuming a single storey with height h, see (3) a).

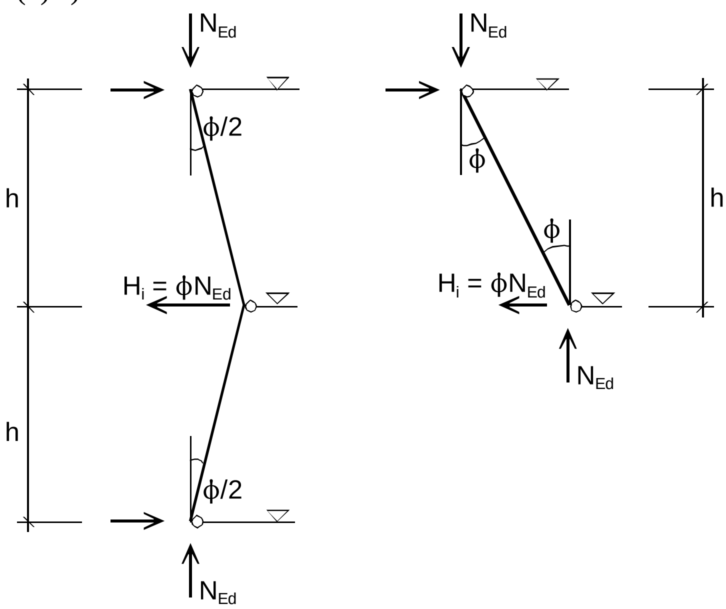


Figure 5.3: Configuration of sway imperfections ϕ for horizontal forces on floor diaphragms

(6) When performing the global analysis for determining end forces and end moments to be used in member checks according to 6.3 local bow imperfections may be neglected. However for frames sensitive to second order effects local bow imperfections of members additionally to global sway imperfections (see 5.2.1(3)) should be introduced in the structural analysis of the frame for each compressed member where the following conditions are met:

– at least one moment resistant joint at one member end

– $$\bar{\lambda} > 0{,}5\sqrt{\frac{A\ f_y}{N_{Ed}}} \tag{5.8}$$

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

and $\bar{\lambda}$ is the in-plane non-dimensional slenderness calculated for the member considered as hinged at its ends

NOTE Local bow imperfections are taken into account in member checks, see 5.2.2 (3) and 5.3.4.

(7) The effects of initial sway imperfection and bow imperfections may be replaced by systems of equivalent horizontal forces, introduced for each column, see Figure 5.3 and Figure 5.4.

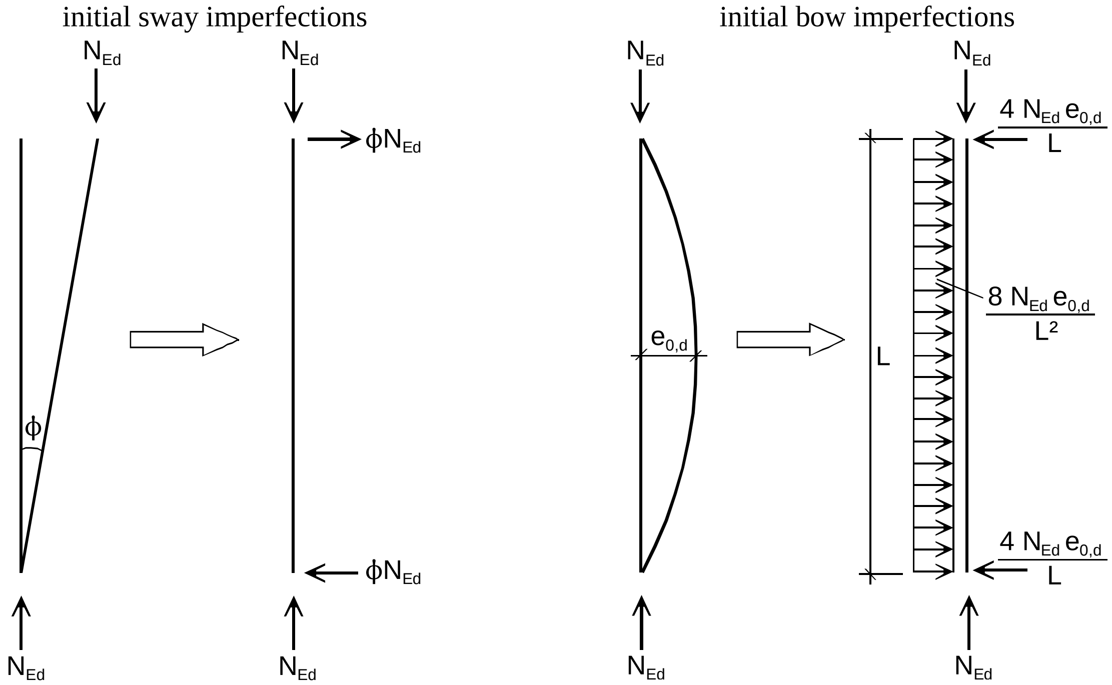
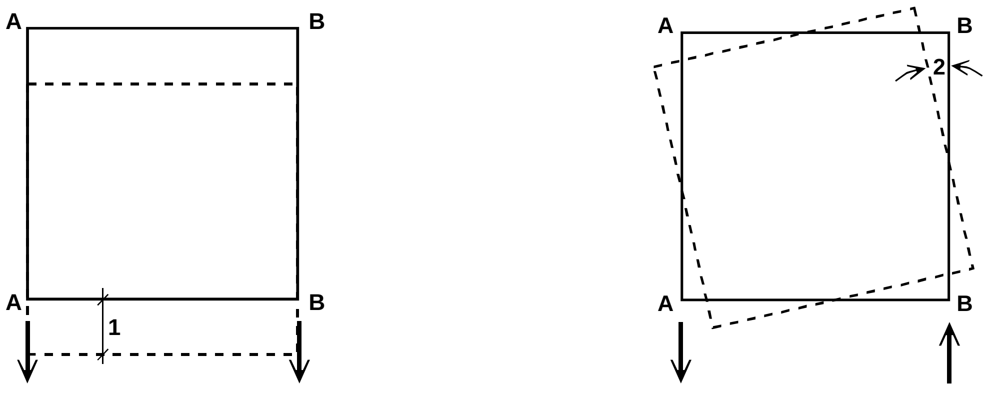


Figure 5.4: Replacement of initial imperfections by equivalent horizontal forces

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

(9)B Where, in multi-storey beam-and-column building frames, equivalent forces are used they should be applied at each floor and roof level.

(10) The possible torsional effects on a structure caused by anti-symmetric sways at the two opposite faces, should also be considered, see Figure 5.5.



(a) Faces A-A and B-B sway in same direction

(b) Faces A-A and B-B sway in opposite direction

1 translational sway
2 rotational sway

Figure 5.5: Translational and torsional effects (plan view)

(11) As an alternative to (3) and (6) the shape of the elastic critical buckling mode η_{cr} of the structure may be applied as a unique global and local imperfection. The amplitude of this imperfection may be determined from:

$$\eta_{init} = e_{0,d} \frac{M_{Rk}}{EI\,\eta''_{cr,max}} \eta_{cr} \tag{5.9}$$

where:

$$e_{0,d} = \frac{\alpha\left(\bar{\lambda} - 0,2\right)}{\bar{\lambda}^2} \frac{M_{Rk}}{N_{Rk}} \frac{1 - \frac{\chi \bar{\lambda}^2}{\gamma_{M1}}}{1 - \chi \bar{\lambda}^2} \quad \text{for } \bar{\lambda} > 0,2 \tag{5.10}$$

and $\bar{\lambda} = \sqrt{\dfrac{\alpha_{ult,k}}{\alpha_{cr}}}$ is the relative slenderness of the structure (5.11)

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

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

$\alpha_{ult,k}$ is the minimum force amplifier for the axial force configuration N_{Ed} in members to reach the characteristic resistance N_{Rk} of the most axially stressed cross section without taking buckling into account

α_{cr} is the minimum force amplifier for the axial force configuration N_{Ed} in members to reach the elastic critical buckling

M_{Rk} is the characteristic moments resistance of the critical cross section, e.g $M_{el,Rk}$ or $M_{pl,Rk}$ as relevant

N_{Rk} is the characteristic resistance to normal force of the critical cross section, i.e. $N_{pl,Rk}$

$EI\,\eta''_{cr}$ is the bending moment due to η_{cr} at the critical cross section

NOTE 1 For calculating the amplifiers $\alpha_{ult,k}$ and α_{cr} the members of the structure may be considered to be loaded by axial forces N_{Ed} only that result from the first order elastic analysis of the structure for the design loads.

NOTE 2 The National Annex may give informations for the scope of application of (11).

5.3.3 Imperfection for analysis of bracing systems

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

$$e_0 = \alpha_m L / 500 \tag{5.12}$$

where L is the span of the bracing system

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

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

(2) For convenience, the effects of the initial bow imperfections of the members to be restrained by a bracing system, may be replaced by the equivalent stabilising force as shown in Figure 5.6:

$$q = \sum N_{Ed}\, 8 \frac{e_0 + \delta_q}{L^2} \tag{5.13}$$

where δ_q is the inplane deflection of the bracing system due to q plus any external loads calculated from first order analysis

NOTE δ_q may be taken as 0 if second order theory is used.

(3) Where the bracing system is required to stabilise the compression flange of a beam of constant height, the force N_{Ed} in Figure 5.6 may be obtained from:

$$N_{Ed} = M_{Ed} / h \tag{5.14}$$

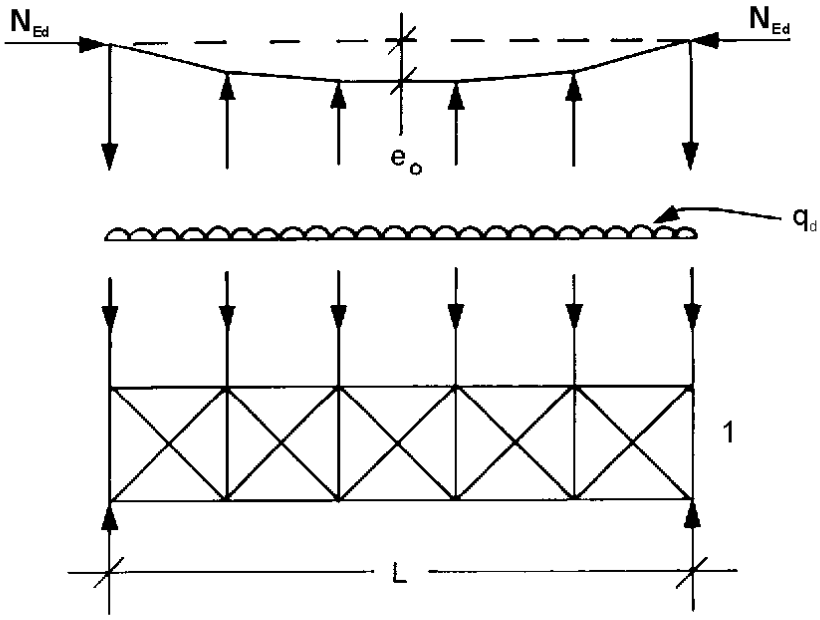
where M_{Ed} is the maximum moment in the beam

and h is the overall depth of the beam.

NOTE Where a beam is subjected to external compression N_{Ed} should include a part of the compression force.

(4) At points where beams or compression members are spliced, it should also be verified that the bracing system is able to resist a local force equal to $\alpha_m N_{Ed} / 100$ applied to it by each beam or compression member which is spliced at that point, and to transmit this force to the adjacent points at which that beam or compression member is restrained, see Figure 5.7.

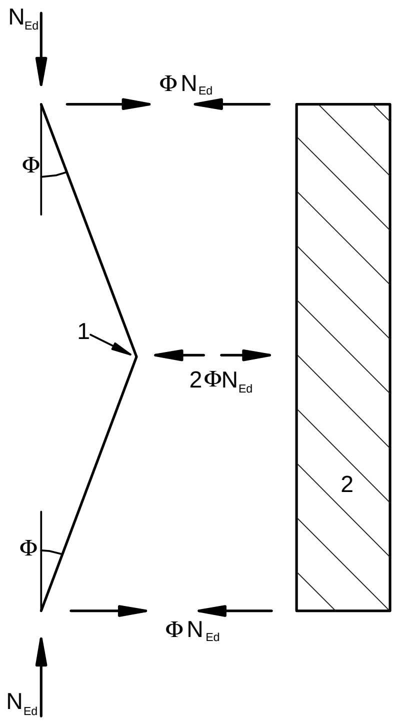
(5) For checking for the local force according to clause (4), any external loads acting on bracing systems should also be included, but the forces arising from the imperfection given in (1) may be omitted.



e_0 imperfection
q_0 equivalent force per unit length
1 bracing system

The force N_{Ed} is assumed uniform within the span L of the bracing system.
For non-uniform forces this is slightly conservative.

Figure 5.6: Equivalent stabilising force



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

$$2\Phi N_{Ed} = \alpha_m N_{Ed} / 100$$

1 splice
2 bracing system

Figure 5.7: Bracing forces at splices in compression elements

5.3.4 Member imperfections

(1) The effects of imperfections of members are incorporated within the formulas given for buckling resistance for members, see section 6.3.

(2) Where the stability of members is accounted for by second order analysis according to 5.2.2(5)a) for compression members imperfections $e_{0,d}$ according to 5.3.2(3)b) or 5.3.2(5) or (6) should be considered.

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

NOTE The National Annex may choose the value of k. The value k = 0,5 is recommended.

5.4 Methods of analysis considering material non-linearities

5.4.1 General

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

a) elastic global analysis

b) plastic global analysis.

NOTE For finite element model (FEM) analysis see EN 1993-1-5.

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

(3) Plastic global analysis may be used only where the structure has sufficient rotation capacity at the actual location of the plastic hinge, whether this is in the members or in the joints. Where a plastic hinge occurs in a member, the member cross sections should be double symmetric or single symmetric with a plane of symmetry in the same plane as the rotation of the plastic hinge and it should satisfy the requirements specified in 5.6. Where a plastic hinge occurs in a joint the joint should either have sufficient strength to ensure the hinge remains in the member or should be able to sustain the plastic resistance for a sufficient rotation, see EN 1993-1-8.

(4)B As a simplified method for a limited plastic redistribution of moments in continuous beams where following an elastic analysis some peak moments exceed the plastic bending resistance of 15 % maximum, the parts in excess of these peak moments may be redistributed in any member, provided, that:

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

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

c) lateral torsional buckling of the members is prevented.

5.4.2 Elastic global analysis

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

NOTE For the choice of a semi-continuous joint model see 5.1.2(2) to (4).

(2) Internal forces and moments may be calculated according to elastic global analysis even if the resistance of a cross section is based on its plastic resistance, see 6.2.

(3) Elastic global analysis may also be used for cross sections the resistances of which are limited by local buckling, see 6.2.

5.4.3 Plastic global analysis

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

- by elastic-plastic analysis with plastified sections and/or joints as plastic hinges,
- by non-linear plastic analysis considering the partial plastification of members in plastic zones,
- by rigid plastic analysis neglecting the elastic behaviour between hinges.

(2) Plastic global analysis may be used where the members are capable of sufficient rotation capacity to enable the required redistributions of bending moments to develop, see 5.5 and 5.6.

(3) Plastic global analysis should only be used where the stability of members at plastic hinges can be assured, see 6.3.5.

(4) The bi-linear stress-strain relationship indicated in Figure 5.8 may be used for the grades of structural steel specified in section 3. Alternatively, a more precise relationship may be adopted, see EN 1993-1-5.

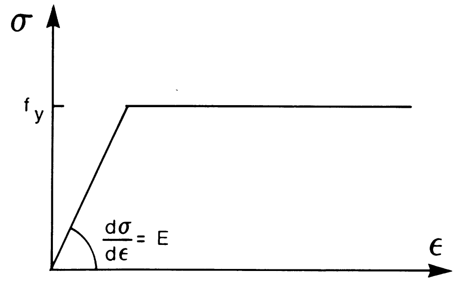


Figure 5.8: Bi-linear stress-strain relationship

(5) Rigid plastic analysis may be applied if no effects of the deformed geometry (e.g. second-order effects) have to be considered. In this case joints are classified only by strength, see EN 1993-1-8.

(6) The effects of deformed geometry of the structure and the structural stability of the frame should be verified according to the principles in 5.2.

> **NOTE** The maximum resistance of a frame with significantly deformed geometry may occur before all hinges of the first order collapse mechanism have formed.

5.5 Classification of cross sections

5.5.1 Basis

(1) The role of cross section classification is to identify the extent to which the resistance and rotation capacity of cross sections is limited by its local buckling resistance.

5.5.2 Classification

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

- Class 1 cross-sections are those which can form a plastic hinge with the rotation capacity required from plastic analysis without reduction of the resistance.
- Class 2 cross-sections are those which can develop their plastic moment resistance, but have limited rotation capacity because of local buckling.
- Class 3 cross-sections are those in which the stress in the extreme compression fibre of the steel member assuming an elastic distribution of stresses can reach the yield strength, but local buckling is liable to prevent development of the plastic moment resistance.
- Class 4 cross-sections are those in which local buckling will occur before the attainment of yield stress in one or more parts of the cross-section.

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

(3) The classification of a cross-section depends on the width to thickness ratio of the parts subject to compression.

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

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

(6) A cross-section is classified according to the highest (least favourable) class of its compression parts. Exceptions are specified in 6.2.1(10) and 6.2.2.4(1).

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

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

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

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

(11) Cross-sections with a Class 3 web and Class 1 or 2 flanges may be classified as class 2 cross sections with an effective web in accordance with 6.2.2.3.

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

NOTE For flange induced web buckling see EN 1993-1-5.

5.6 Cross-section requirements for plastic global analysis

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

(2) In a uniform member sufficient rotation capacity may be assumed at a plastic hinge if both the following requirements are satisfied:

a) the member has Class 1 cross-sections at the plastic hinge location;

b) where a transverse force that exceeds 10 % of the shear resistance of the cross section, see 6.2.6, is applied to the web at the plastic hinge location, web stiffeners should be provided within a distance along the member of h/2 from the plastic hinge location, where h is the height of the cross section at this location.

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

a) Adjacent to plastic hinge locations, the thickness of the web should not be reduced for a distance each way along the member from the plastic hinge location of at least 2d, where d is the clear depth of the web at the plastic hinge location.

b) Adjacent to plastic hinge locations, the compression flange should be Class 1 for a distance each way along the member from the plastic hinge location of not less than the greater of:

- 2d, where d is as defined in (3)a)
- the distance to the adjacent point at which the moment in the member has fallen to 0,8 times the plastic moment resistance at the point concerned.

c) Elsewhere in the member the compression flange should be class 1 or class 2 and the web should be class 1, class 2 or class 3.

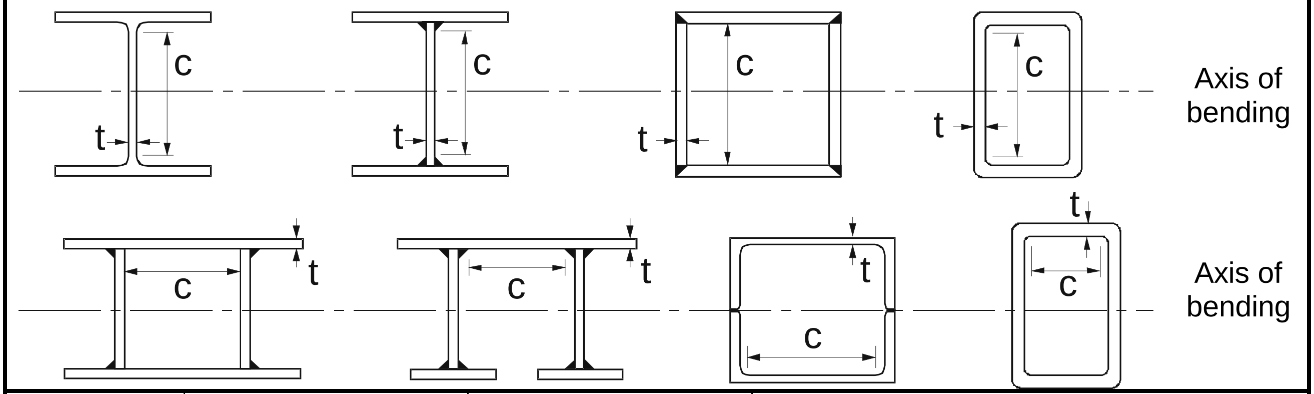
(4) Adjacent to plastic hinge locations, any fastener holes in tension should satisfy 6.2.5(4) for a distance such as defined in (3)b) each way along the member from the plastic hinge location.

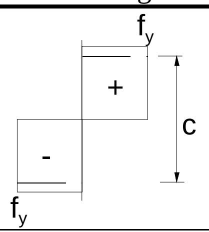
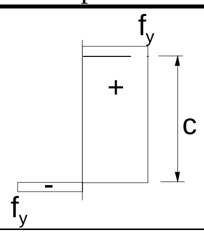
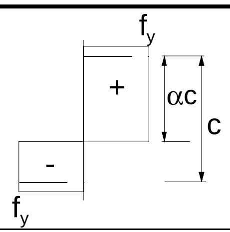
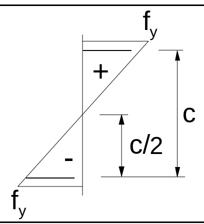
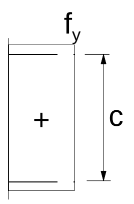
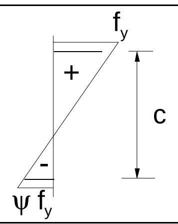
(5) For plastic design of a frame, regarding cross section requirements, the capacity of plastic redistribution of moments may be assumed sufficient if the requirements in (2) to (4) are satisfied for all members where plastic hinges exist, may occur or have occurred under design loads.

(6) In cases where methods of plastic global analysis are used which consider the real stress and strain behaviour along the member including the combined effect of local, member and global buckling the requirements (2) to (5) need not be applied.

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

Internal compression parts



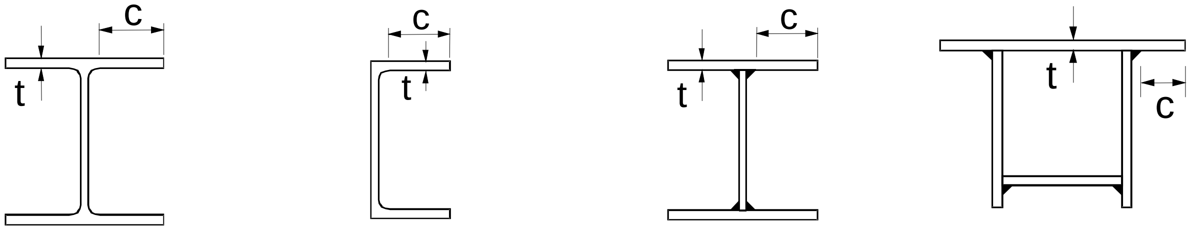
Class	Part subject to bending	Part subject to compression	Part subject to bending and compression
Stress distribution in parts (compression positive)			
1	$c/t \le 72\varepsilon$	$c/t \le 33\varepsilon$	when $\alpha > 0{,}5$: $c/t \le \dfrac{396\varepsilon}{13\alpha - 1}$ when $\alpha \le 0{,}5$: $c/t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

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

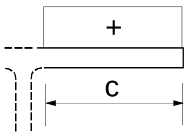
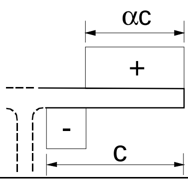
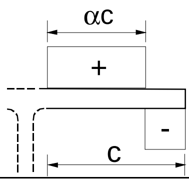
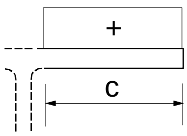
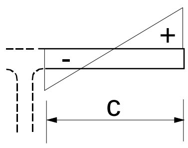
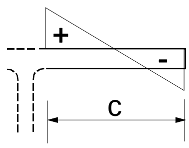
*) $\psi \le -1$ applies where either the compression stress $\sigma < f_y$ or the tensile strain $\varepsilon_y > f_y/E$

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

Outstand flanges



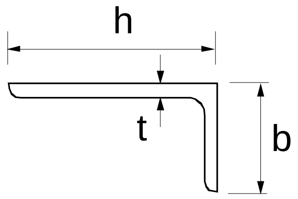
Rolled sections | Welded sections

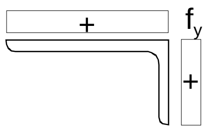
Class	Part subject to compression	Part subject to bending and compression	
		Tip in compression	Tip in tension
Stress distribution in parts (compression positive)			
1	$c/t \leq 9\varepsilon$	$c/t \leq \dfrac{9\varepsilon}{\alpha}$	$c/t \leq \dfrac{9\varepsilon}{\alpha\sqrt{\alpha}}$
2	$c/t \leq 10\varepsilon$	$c/t \leq \dfrac{10\varepsilon}{\alpha}$	$c/t \leq \dfrac{10\varepsilon}{\alpha\sqrt{\alpha}}$
Stress distribution in parts (compression positive)			
3	$c/t \leq 14\varepsilon$	$c/t \leq 21\varepsilon\sqrt{k_\sigma}$ For k_σ see EN 1993-1-5	

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

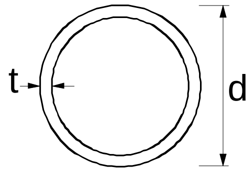
Table 5.2 (sheet 3 of 3): Maximum width-to-thickness ratios for compression parts

Angles

Refer also to “Outstand flanges” (see sheet 2 of 3)		Does not apply to angles in continuous contact with other components

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

Tubular sections



Class	Section in bending and/or compression
1	$d/t \leq 50\varepsilon^2$
2	$d/t \leq 70\varepsilon^2$
3	$d/t \leq 90\varepsilon^2$ **NOTE** For $d/t > 90\varepsilon^2$ see EN 1993-1-6.

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71
	ε^2	1,00	0,85	0,66	0,56	0,51

6 Ultimate limit states

6.1 General

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

- 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 joints: see EN 1993-1-8

NOTE 1B Partial factors γ_{Mi} for buildings may be defined in the National Annex. The following numerical values are recommended for buildings:

$\gamma_{M0} = 1,00$

$\gamma_{M1} = 1,00$

$\gamma_{M2} = 1,25$

NOTE 2 For other recommended numerical values see EN 1993 Part 2 to Part 6. For structures not covered by EN 1993 Part 2 to Part 6 the National Annex may give information.

6.2 Resistance of cross-sections

6.2.1 General

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

(2) Shear lag effects and local buckling effects should be included by an effective width according to EN 1993-1-5. Shear buckling effects should also be considered according to EN 1993-1-5.

(3) The design values of resistance should depend on the classification of the cross-section.

(4) Elastic verification according to the elastic resistance may be carried out for all cross sectional classes provided the effective cross sectional properties are used for the verification of class 4 cross sections.

(5) For the elastic verification the following yield criterion for a critical point of the cross section may be used unless other interaction formulae apply, see 6.2.8 to 6.2.10.

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

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

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

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

NOTE The verification according to (5) can be conservative as it excludes partial plastic stress distribution, which is permitted in elastic design. Therefore it should only be performed where the interaction of on the basis of resistances N_{Rd} , M_{Rd} , V_{Rd} cannot be performed.

(6) The plastic resistance of cross sections should be verified by finding a stress distribution which is in equilibrium with the internal forces and moments without exceeding the yield strength. This stress distribution should be compatible with the associated plastic deformations.

(7) As a conservative approximation for all cross section classes a linear summation of the utilisation ratios for each stress resultant may be used. For class 1, class 2 or class 3 cross sections subjected to the combination of N_{Ed} , $M_{y,Ed}$ and $M_{z,Ed}$ this method may be applied by using the following criteria:

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

where N_{Rd} , $M_{y,Rd}$ and $M_{z,Rd}$ are the design values of the resistance depending on the cross sectional classification and including any reduction that may be caused by shear effects, see 6.2.8.

NOTE For class 4 cross sections see 6.2.9.3(2).

(8) Where all the compression parts of a cross-section are Class 2, the cross-section may be taken as capable of developing its full plastic resistance in bending.

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

NOTE The extreme fibres may be assumed at the midplane of the flanges for ULS checks. For fatigue see EN 1993-1-9.

(10) Where yielding first occurs on the tension side of the cross section, the plastic reserves of the tension zone may be utilised by accounting for partial plastification when determining the resistance of a Class 3 cross-section.

6.2.2 Section properties

6.2.2.1 Gross cross-section

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

6.2.2.2 Net area

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

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

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

NOTE The maximum sum denotes the position of the critical fracture line.

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

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

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

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

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

t is the thickness;

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

d is the diameter of hole

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

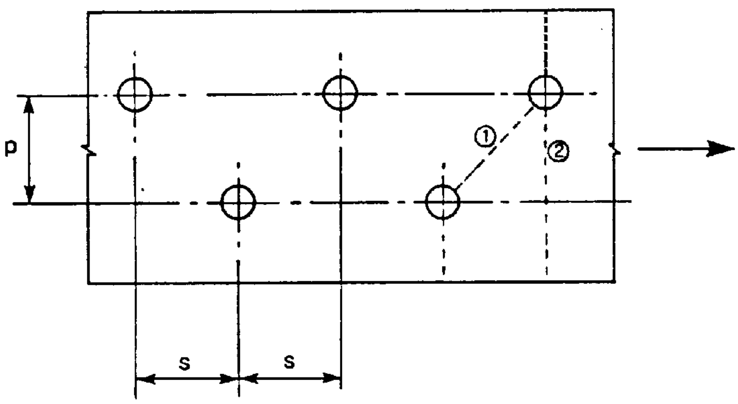


Figure 6.1: Staggered holes and critical fracture lines 1 and 2

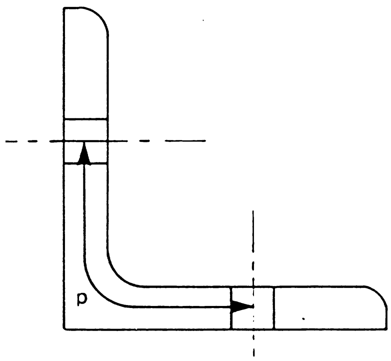


Figure 6.2: Angles with holes in both legs

6.2.2.3 Shear lag effects

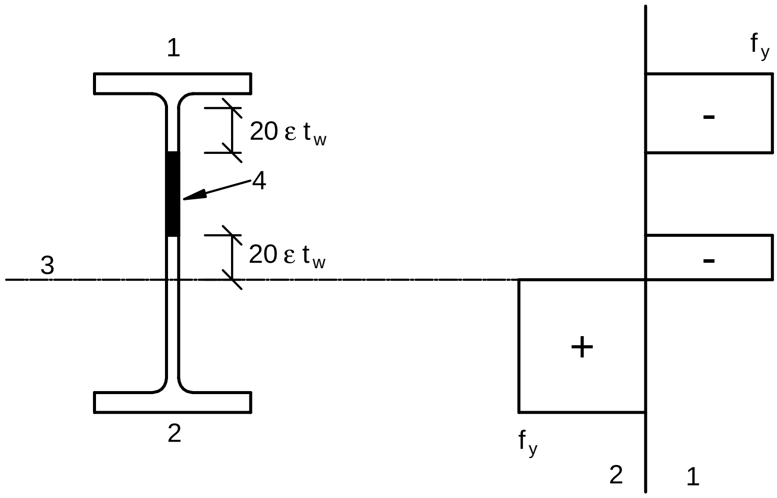
(1) The calculation of the effective widths is covered in EN 1993-1-5.

(2) In class 4 sections the interaction between shear lag and local buckling shall be considered according to EN 1993-1-5.

NOTE For cold formed thin gauge members see EN 1993-1-3.

6.2.2.4 Effective properties of cross sections with class 3 webs and class 1 or 2 flanges

(1) Where cross-sections with a class 3 web and class 1 or 2 flanges are classified as effective Class 2 cross-sections, see 5.5.2(11), the proportion of the web in compression should be replaced by a part of $20\varepsilon t_w$ adjacent to the compression flange, with another part of $20\varepsilon t_w$ adjacent to the plastic neutral axis of the effective cross-section in accordance with Figure 6.3.



1 compression
2 tension
3 plastic neutral axis
4 neglect

Figure 6.3: Effective class 2 web

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

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

(2) For cold formed thin walled sections see 1.1.2(1) and EN 1993-1-3.

(3) The effective widths of planar compression parts should be obtained from EN 1993-1-5.

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

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

NOTE The sign of the additional moment depends on the effect in the combination of internal forces and moments, see 6.2.9.3(2).

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

6.2.3 Tension

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

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

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

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

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

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

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

(3) Where capacity design is requested, see EN 1998, the design plastic resistance $N_{pl,Rd}$ (as given in 6.2.3(2) a)) should be less than the design ultimate resistance of the net section at fasteners holes $N_{u,Rd}$ (as given in 6.2.3(2) b)).

(4) In category C connections (see EN 1993-1-8, 3.1.1(4)), the design tension resistance $N_{t,Rd}$ in 6.2.3(1) of the net section at holes for fasteners should be taken as $N_{net,Rd}$, where:

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

(5) For angles connected through one leg, see also EN 1993-1-8, 3.6.3. Similar consideration should also be given to other type of sections connected through outstands.

6.2.4 Compression

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

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

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

$$N_{c,Rd} = \frac{A\,f_y}{\gamma_{M0}} \quad \text{for class 1, 2 or 3 cross-sections} \tag{6.10}$$

$$N_{c,Rd} = \frac{A_{eff}\,f_y}{\gamma_{M0}} \quad \text{for class 4 cross-sections} \tag{6.11}$$

(3) Fastener holes except for oversize and slotted holes as defined in EN 1090 need not be allowed for in compression members, provided that they are filled by fasteners.

(4) In the case of unsymmetrical Class 4 sections, the method given in 6.2.9.3 should be used to allow for the additional moment ΔM_{Ed} due to the eccentricity of the centroidal axis of the effective section, see 6.2.2.5(4).

6.2.5 Bending moment

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

$$\frac{M_{Ed}}{M_{c,Rd}} \leq 1{,}0 \qquad (6.12)$$

where $M_{c,Rd}$ is determined considering fastener holes, see (4) to (6).

(2) The design resistance for bending about one principal axis of a cross-section is determined as follows:

$$M_{c,Rd} = M_{pl,Rd} = \frac{W_{pl}\ f_y}{\gamma_{M0}} \quad \text{for class 1 or 2 cross sections} \qquad (6.13)$$

$$M_{c,Rd} = M_{el,Rd} = \frac{W_{el,min}\ f_y}{\gamma_{M0}} \quad \text{for class 3 cross sections} \qquad (6.14)$$

$$M_{c,Rd} = \frac{W_{eff,min}\ f_y}{\gamma_{M0}} \quad \text{for class 4 cross sections} \qquad (6.15)$$

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

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

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

$$\frac{A_{f,net}\, 0{,}9\, f_u}{\gamma_{M2}} \geq \frac{A_f\ f_y}{\gamma_{M0}} \qquad (6.16)$$

where A_f is the area of the tension flange.

NOTE The criterion in (4) provides capacity design (see 1.5.8) in the region of plastic hinges.

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

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

6.2.6 Shear

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

$$\frac{V_{Ed}}{V_{c,Rd}} \leq 1{,}0 \qquad (6.17)$$

where $V_{c,Rd}$ is the design shear resistance. For plastic design $V_{c,Rd}$ is the design plastic shear resistance $V_{pl,Rd}$ as given in (2). For elastic design $V_{c,Rd}$ is the design elastic shear resistance calculated using (4) and (5).

(2) In the absence of torsion the design plastic shear resistance is given by:

$$V_{pl,Rd} = \frac{A_v \left(f_y / \sqrt{3}\right)}{\gamma_{M0}} \qquad (6.18)$$

where A_v is the shear area.

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

a) rolled I and H sections, load parallel to web $A - 2bt_f + (t_w + 2r)\,t_f$ but not less than $\eta h_w t_w$

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

c) rolled T-section, load parallel to web $0{,}9\,(A - bt_f)$

d) welded I, H and box sections, load parallel to web $\eta \sum (h_w t_w)$

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

f) rolled rectangular hollow sections of uniform thickness:

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

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

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

where A is the crosssectional area;

b is the overall breadth;

h is the overall depth;

h_w is the depth of the web;

r is the root radius;

t_f is the flange thickness;

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

η see EN 1993-1-5.

NOTE η may be conservatively taken equal 1,0.

(4) For verifying the design elastic shear resistance $V_{c,Rd}$ the following criterion for a critical point of the cross section may be used unless the buckling verification in section 5 of EN 1993-1-5 applies:

$$\frac{\tau_{Ed}}{f_y \Big/ \left(\sqrt{3}\,\gamma_{M0}\right)} \leq 1{,}0 \tag{6.19}$$

where τ_{Ed} may be obtained from: $$\tau_{Ed} = \frac{V_{Ed}\,S}{I\,t} \tag{6.20}$$

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

S is first moment of the area above the examined point

I is second moment of area of the whole cross section

t is the thickness at the examined point

NOTE The verification according to (4) is conservative as it excludes partial plastic shear distribution, which is permitted in elastic design, see (5). Therefore it should only be carried out where the verification on the basis of $V_{c,Rd}$ according to equation (6.17) cannot be performed.

(5) For I- or H-sections the shear stress in the web may be taken as:

$$\tau_{Ed} = \frac{V_{Ed}}{A_w} \quad \text{if } A_f / A_w \geq 0{,}6 \tag{6.21}$$

where A_f is the area of one flange;

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

(6) In addition the shear buckling resistance for webs without intermediate stiffeners shall be according to section 5 of EN 1993-1-5, if

$$\frac{h_w}{t_w} > 72 \frac{\varepsilon}{\eta} \tag{6.22}$$

For η see section 5 of EN 1993-1-5.

NOTE η may be conservatively taken equal to 1,0.

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

(8) Where the shear force is combined with a torsional moment, the plastic shear resistance $V_{pl,Rd}$ should be reduced as specified in 6.2.7(9).

6.2.7 Torsion

(1) For members subject to torsion for which distortional deformations may be disregarded the design value of the torsional moment T_{Ed} at each cross-section should satisfy:

$$\frac{T_{Ed}}{T_{Rd}} \leq 1{,}0 \tag{6.23}$$

where T_{Rd} is the design torsional resistance of the cross section

(2) The total torsional moment T_{Ed} at any cross- section should be considered as the sum of two internal effects:

$$T_{Ed} = T_{t,Ed} + T_{w,Ed} \tag{6.24}$$

where $T_{t,Ed}$ is the internal St. Venant torsion;

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

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

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

– the shear stresses $\tau_{t,Ed}$ due to St. Venant torsion $T_{t,Ed}$

– the direct stresses $\sigma_{w,Ed}$ due to the bimoment B_{Ed} and shear stresses $\tau_{w,Ed}$ due to warping torsion $T_{w,Ed}$

(5) For the elastic verification the yield criterion in 6.2.1(5) may be applied.

(6) For determining the plastic moment resistance of a cross section due to bending and torsion only torsion effects B_{Ed} should be derived from elastic analysis, see (3).

(7) As a simplification, in the case of a member with a closed hollow cross-section, such as a structural hollow section, it may be assumed that the effects of torsional warping can be neglected. Also as a simplification, in the case of a member with open cross section, such as I or H, it may be assumed that the effects of St. Venant torsion can be neglected.

(8) For the calculation of the resistance T_{Rd} of closed hollow sections the design shear strength of the individual parts of the cross section according to EN 1993-1-5 should be taken into account.

(9) For combined shear force and torsional moment the plastic shear resistance accounting for torsional effects should be reduced from $V_{pl,Rd}$ to $V_{pl,T,Rd}$ and the design shear force should satisfy:

$$\frac{V_{Ed}}{V_{pl,T,Rd}} \leq 1 \tag{6.25}$$

in which $V_{pl,TEd,Rd}$ may be derived as follows:

– for an I or H section:

$$V_{pl,T,Rd} = \sqrt{1 - \frac{\tau_{t,Ed}}{1,25\left(f_y/\sqrt{3}\right)/\gamma_{M0}}}\; V_{pl,Rd} \tag{6.26}$$

– for a channel section:

$$V_{pl,T,Rd} = \left[\sqrt{1 - \frac{\tau_{t,Ed}}{1,25\left(f_y/\sqrt{3}\right)/\gamma_{M0}}} - \frac{\tau_{w,Ed}}{\left(f_y/\sqrt{3}\right)/\gamma_{M0}}\right] V_{pl,Rd} \tag{6.27}$$

– for a structural hollow section:

$$V_{pl,T,Rd} = \left[1 - \frac{\tau_{t,Ed}}{\left(f_y/\sqrt{3}\right)/\gamma_{M0}}\right] V_{pl,Rd} \tag{6.28}$$

where $V_{pl,Rd}$ is given in 6.2.6.

6.2.8 Bending and shear

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

(2) Where the shear force is less than half the plastic shear resistance its effect on the moment resistance may be neglected except where shear buckling reduces the section resistance, see EN 1993-1-5.

(3) Otherwise the reduced moment resistance should be taken as the design resistance of the cross-section, calculated using a reduced strength

$$(1 - \rho)\, f_y \tag{6.29}$$

for the shear area,

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

NOTE See also 6.2.10(3).

(4) When torsion is present ρ should be obtained from $\rho = \left(\frac{2\,V_{Ed}}{V_{pl,T,Rd}} - 1\right)^2$, see 6.2.7, but should be taken as 1 for $V_{Ed} \leq 0,5V_{pl,T,Rd}$.

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

$$M_{y,V,Rd} = \frac{\left[W_{pl,y} - \frac{\rho A_w^{\ 2}}{4 t_w}\right] f_y}{\gamma_{M0}} \quad \text{but } M_{y,V,Rd} \leq M_{y,c,Rd} \qquad (6.30)$$

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

and $A_w = h_w t_w$

(6) For the interaction of bending, shear and transverse loads see section 7 of EN 1993-1-5.

6.2.9 Bending and axial force

6.2.9.1 Class 1 and 2 cross-sections

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

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

$$M_{Ed} \leq M_{N,Rd} \qquad (6.31)$$

where $M_{N,Rd}$ is the design plastic moment resistance reduced due to the axial force N_{Ed}.

(3) For a rectangular solid section without bolt holes $M_{N,Rd}$ should be taken as:

$$M_{N,Rd} = M_{pl,Rd}\left[1 - \left(N_{Ed} / N_{pl,Rd}\right)^2\right] \qquad (6.32)$$

(4) For doubly symmetrical I- and H-sections or other flanges sections, allowance need not be made for the effect of the axial force on the plastic resistance moment about the y-y axis when both the following criteria are satisfied:

$$N_{Ed} \leq 0{,}25\, N_{pl,Rd} \text{ and} \qquad (6.33)$$

$$N_{Ed} \leq \frac{0{,}5\, h_w t_w f_y}{\gamma_{M0}} \qquad (6.34)$$

For doubly symmetrical I- and H-sections, allowance need not be made for the effect of the axial force on the plastic resistance moment about the z-z axis when:

$$N_{Ed} \leq \frac{h_w t_w f_y}{\gamma_{M0}} \qquad (6.35)$$

(5) For cross-sections where bolt holes are not to be accounted for, the following approximations may be used for standard rolled I or H sections and for welded I or H sections with equal flanges:

$$M_{N,y,Rd} = M_{pl,y,Rd}\,(1-n)/(1-0{,}5a) \quad \text{but } M_{N,y,Rd} \leq M_{pl,y,Rd} \qquad (6.36)$$

$$\text{for } n \leq a: \quad M_{N,z,Rd} = M_{pl,z,Rd} \qquad (6.37)$$

$$\text{for } n > a: \quad M_{N,z,Rd} = M_{pl,z,Rd}\left[1 - \left(\frac{n-a}{1-a}\right)^2\right] \qquad (6.38)$$

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

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

For cross-sections where bolt holes are not to be accounted for, the following approximations may be used for rectangular structural hollow sections of uniform thickness and for welded box sections with equal flanges and equal webs:

$$M_{N,y,Rd} = M_{pl,y,Rd}\,(1 - n)/(1 - 0{,}5a_w) \quad \text{but } M_{N,y,Rd} \leq M_{pl,y.Rd} \tag{6.39}$$

$$M_{N,z,Rd} = M_{pl,z,Rd}\,(1 - n)/(1 - 0{,}5a_f) \quad \text{but } M_{N,z,Rd} \leq M_{pl,z,Rd} \tag{6.40}$$

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

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

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

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

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

$$\left[\frac{M_{y,Ed}}{M_{N,y,Rd}}\right]^{\alpha} + \left[\frac{M_{z,Ed}}{M_{N,z,Rd}}\right]^{\beta} \leq 1 \tag{6.41}$$

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

– I and H sections:

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

– circular hollow sections:

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

– rectangular hollow sections:

$$\alpha = \beta = \frac{1{,}66}{1 - 1{,}13n^2} \qquad \text{but } \alpha = \beta \leq 6$$

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

6.2.9.2 Class 3 cross-sections

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

$$\sigma_{x,Ed} \leq \frac{f_y}{\gamma_{M0}} \tag{6.42}$$

where $\sigma_{x,Ed}$ is the design value of the local longitudinal stress due to moment and axial force taking account of bolt holes where relevant, see 6.2.4 and 6.2.5

6.2.9.3 Class 4 cross-sections

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

$$\sigma_{x,Ed} \leq \frac{f_y}{\gamma_{M0}} \tag{6.43}$$

where $\sigma_{x,Ed}$ is the design value of the local longitudinal stress due to moment and axial force taking account of bolt holes where relevant, see 6.2.4 and 6.2.5

(2) The following criterion should be met:

$$\frac{N_{Ed}}{A_{eff}\ f_y / \gamma_{M0}} + \frac{M_{y,Ed} + N_{Ed}\ e_{Ny}}{W_{eff,y,min}\ f_y / \gamma_{M0}} + \frac{M_{z,Ed} + N_{Ed}\ e_{Nz}}{W_{eff,z,min}\ f_y / \gamma_{M0}} \leq 1 \qquad (6.44)$$

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

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

e_N is the shift of the relevant centroidal axis when the cross-section is subjected to compression only, see 6.2.2.5(4)

NOTE The signs of N_{Ed} , $M_{y,Ed}$, $M_{z,Ed}$ and $\Delta M_i = N_{Ed}\ e_{Ni}$ depend on the combination of the respective direct stresses.

6.2.10 Bending, shear and axial force

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

(2) Provided that the design value of the shear force V_{Ed} does not exceed 50% of the design plastic shear resistance $V_{pl.Rd}$ no reduction of the resistances defined for bending and axial force in 6.2.9 need be made, except where shear buckling reduces the section resistance, see EN 1993-1-5.

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

$$(1-\rho)f_y \qquad (6.45)$$

for the shear area

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

NOTE Instead of reducing the yield strength also the plate thickness of the relevant part of the cross section may be reduced.

6.3 Buckling resistance of members

6.3.1 Uniform members in compression

6.3.1.1 Buckling resistance

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

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1{,}0 \qquad (6.46)$$

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

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

(2) For members with non-symmetric Class 4 sections allowance should be made for the additional moment ΔM_{Ed} due to the eccentricity of the centroidal axis of the effective section, see also 6.2.2.5(4), abd the interaction should be carried out to 6.3.4 or 6.3.3.

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

$$N_{b,Rd} = \frac{\chi A f_y}{\gamma_{M1}} \quad \text{for Class 1, 2 and 3 cross-sections} \tag{6.47}$$

$$N_{b,Rd} = \frac{\chi A_{eff} f_y}{\gamma_{M1}} \quad \text{for Class 4 cross-sections} \tag{6.48}$$

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

> **NOTE** For determining the buckling resistance of members with tapered sections along the member or for non-uniform distribution of the compression force second order analysis according to 5.3.4(2) may be performed. For out-of-plane buckling see also 6.3.4.

(4) In determining A and A_{eff} holes for fasteners at the column ends need not to be taken into account.

6.3.1.2 Buckling curves

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

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \quad \text{but } \chi \leq 1{,}0 \tag{6.49}$$

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

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

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

α is an imperfection factor

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

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

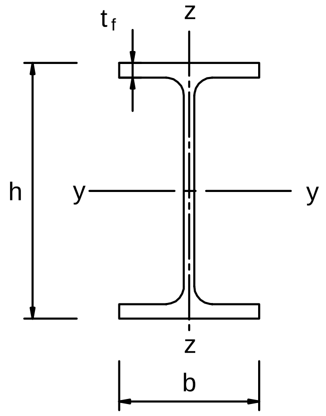
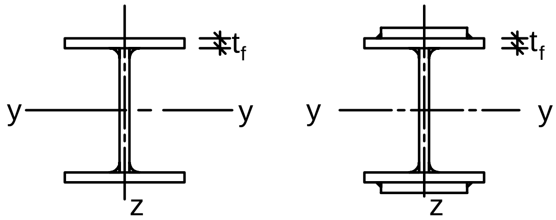
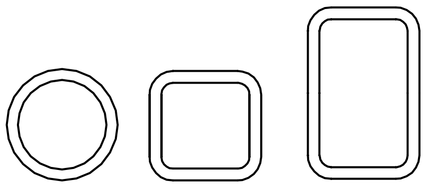
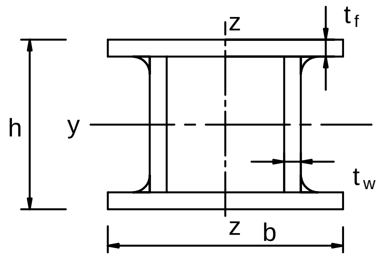
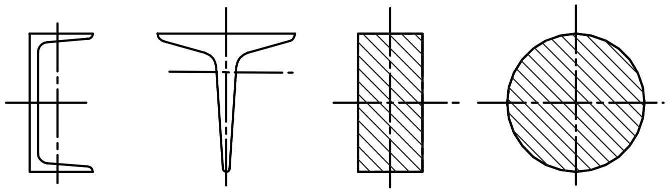
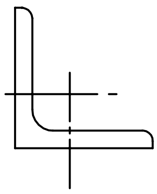
Table 6.1: Imperfection factors for buckling curves

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

(3) Values of the reduction factor χ for the appropriate non-dimensional slenderness $\bar{\lambda}$ may be obtained from Figure 6.4.

(4) For slenderness $\bar{\lambda} \leq 0{,}2$ or for $\frac{N_{Ed}}{N_{cr}} \leq 0{,}04$ the buckling effects may be ignored and only cross sectional checks apply.

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

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

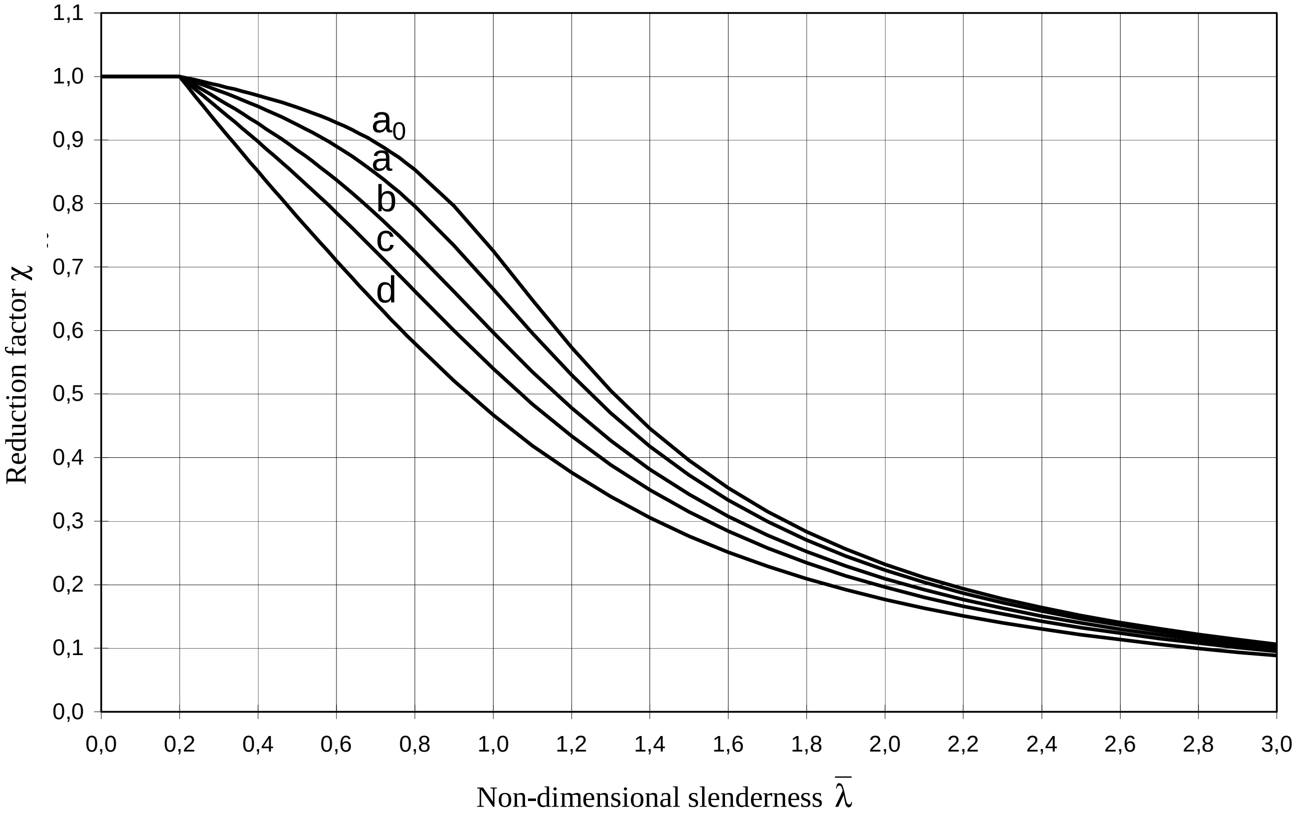


Figure 6.4: Buckling curves

6.3.1.3 Slenderness for flexural buckling

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

$$\bar{\lambda} = \sqrt{\frac{A f_y}{N_{cr}}} = \frac{L_{cr}}{i} \frac{1}{\lambda_1} \quad \text{for Class 1, 2 and 3 cross-sections} \tag{6.50}$$

$$\bar{\lambda} = \sqrt{\frac{A_{eff} f_y}{N_{cr}}} = \frac{L_{cr}}{i} \frac{\sqrt{\frac{A_{eff}}{A}}}{\lambda_1} \quad \text{for Class 4 cross-sections} \tag{6.51}$$

where L_{cr} is the buckling length in the buckling plane considered

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

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

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

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

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

6.3.1.4 Slenderness for torsional and torsional-flexural buckling

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

(2) The non-dimensional slenderness $\bar{\lambda}_T$ for torsional or torsional-flexural buckling should be taken as:

$$\bar{\lambda}_T = \sqrt{\frac{Af_y}{N_{cr}}} \quad \text{for Class 1, 2 and 3 cross-sections} \qquad (6.52)$$

$$\bar{\lambda}_T = \sqrt{\frac{A_{eff}f_y}{N_{cr}}} \quad \text{for Class 4 cross-sections} \qquad (6.53)$$

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

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

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

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

6.3.2 Uniform members in bending

6.3.2.1 Buckling resistance

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

$$\frac{M_{Ed}}{M_{b,Rd}} \leq 1{,}0 \qquad (6.54)$$

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

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

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

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

$$M_{b,Rd} = \chi_{LT} W_y \frac{f_y}{\gamma_{M1}} \qquad (6.55)$$

where W_y is the appropriate section modulus as follows:

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

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

NOTE 1 For determining the buckling resistance of beams with tapered sections second order analysis according to 5.3.4(3) may be performed. For out-of-plane buckling see also 6.3.4.

NOTE 2B For buckling of components of building structures see also Annex BB.

(4) In determining W_y holes for fasteners at the beam end need not to be taken into account.

6.3.2.2 Lateral torsional buckling curves – General case

(1) Unless otherwise specified, see 6.3.2.3, for bending members of constant cross-section, the value of χ_{LT} for the appropriate non-dimensional slenderness $\bar{\lambda}_{LT}$, should be determined from:

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \bar{\lambda}_{LT}^2}} \quad \text{but } \chi_{LT} \leq 1{,}0 \qquad (6.56)$$

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

α_{LT} is an imperfection factor

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

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

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

NOTE The imperfection factor α_{LT} corresponding to the appropriate buckling curve may be obtained from the National Annex. The recommended values α_{LT} are given in Table 6.3.

Table 6.3: Imperfection factors for lateral torsional buckling curves

Buckling curve	a	b	c	d
Imperfection factor α_{LT}	0,21	0,34	0,49	0,76

The recommendations for buckling curves are given in Table 6.4.

Table 6.4: Lateral torsional buckling curve for cross sections using equation (6.56)

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

(3) Values of the reduction factor χ_{LT} for the appropriate non-dimensional slenderness $\bar{\lambda}_{LT}$ may be obtained from Figure 6.4.

(4) For slendernesses $\bar{\lambda}_{LT} \leq 0{,}2$ (or $\bar{\lambda}_{LT} \leq 0{,}4$ (see 6.3.2.3)) or for $\frac{M_{Ed}}{M_{cr}} \leq 0{,}04$ (or $\frac{M_{Ed}}{M_{cr}} \leq 0{,}16$ (see 6.3.2.3)) lateral torsional buckling effects may be ignored and only cross sectional checks apply.

6.3.2.3 Lateral torsional buckling curves for rolled sections or equivalent welded sections

(1) For rolled or equivalent welded sections in bending the values of χ_{LT} for the appropriate non-dimensional slenderness may be determined from

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \beta\bar{\lambda}_{LT}^2}} \quad \text{but} \quad \begin{cases} \chi_{LT} \le 1{,}0 \\ \chi_{LT} \le \dfrac{1}{\bar{\lambda}_{LT}^2} \end{cases} \tag{6.57}$$

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

NOTE The parameters $\bar{\lambda}_{LT,0}$ and β and any limitation of validity concerning the beam depth or h/b ratio may be given in the National Annex. The following values are recommended for rolled sections:

$\bar{\lambda}_{LT,0} = 0{,}4$ (maximum value)

$\beta = 0{,}75$ (minimum value)

The recommendations for buckling curves are given in Table 6.5.

Table 6.5: Selection of lateral torsional buckling curve for cross sections using equation (6.57)

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

(2) For taking into account the moment distribution between the lateral restraints of members the reduction factor χ_{LT} may be modified as follows:

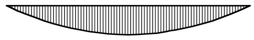
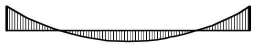
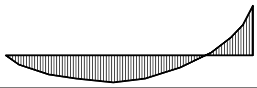
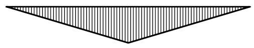
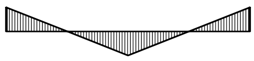
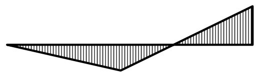
$$\chi_{LT,mod} = \frac{\chi_{LT}}{f} \quad \text{but } \chi_{LT,mod} \le 1 \tag{6.58}$$

NOTE The values f may be defined in the National Annex. The following minimum values are recommended:

$$f = 1 - 0{,}5(1 - k_c)[1 - 2{,}0(\bar{\lambda}_{LT} - 0{,}8)^2] \qquad \text{but } f \le 1{,}0$$

k_c is a correction factor according to Table 6.6

Table 6.6: Correction factors k_c

Moment distribution	k_c
 $\psi = 1$	1,0
 $-1 \le \psi \le 1$	$\frac{1}{1{,}33 - 0{,}33\psi}$
	0,94
	0,90
	0,91
	0,86
	0,77
	0,82

6.3.2.4 Simplified assessment methods for beams with restraints in buildings

(1)B Members with discrete lateral restraint to the compression flange are not susceptible to lateral-torsional buckling if the length L_c between restraints or the resulting equivalent compression flange slenderness $\bar{\lambda}_f$ satisfies:

$$\bar{\lambda}_f = \frac{k_c L_c}{i_{f,z}\lambda_1} \le \bar{\lambda}_{c0} \frac{M_{c,Rd}}{M_{y,Ed}} \tag{6.59}$$

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

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

W_y is the appropriate section modulus corresponding to the compression flange

k_c is a slenderness correction factor for moment distribution between restraints, see Table 6.6

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

$\bar{\lambda}_{c0}$ is the slenderness parameter of the above compression element

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

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

NOTE 1B For Class 4 cross-sections $i_{f,z}$ may be taken as

$$i_{f,z} = \sqrt{\frac{I_{eff,f}}{A_{eff,f} + \frac{1}{3} A_{eff,w,c}}}$$

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

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

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

NOTE 2B The slenderness limit $\bar{\lambda}_{c0}$ may be given in the National Annex. A limit value $\bar{\lambda}_{c0} = \bar{\lambda}_{LT,0} + 0{,}1$ is recommended, see 6.3.2.3.

(2)B If the slenderness of the compression flange $\bar{\lambda}_f$ exceeds the limit given in (1)B, the design buckling resistance moment may be taken as:

$$M_{b,Rd} = k_{f\ell} \chi \, M_{c,Rd} \quad \text{but} \quad M_{b.Rd} \le M_{c.Rd} \qquad (6.60)$$

where χ is the reduction factor of the equivalent compression flange determined with $\bar{\lambda}_f$

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

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

(3)B The buckling curves to be used in (2)B should be taken as follows:

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

curve c for all other sections

where h is the overall depth of the cross-section

t is the thickness of the compression flange

NOTE B For lateral torsional buckling of components of building structures with restraints see also Annex BB.3.

6.3.3 Uniform members in bending and axial compression

(1) Unless second order analysis is carried out using the imperfections as given in 5.3.2, the stability of uniform members with double symmetric cross sections for sections not susceptible to distortional deformations should be checked as given in the following clauses, where a distinction is made for:

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

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

NOTE 1 The interaction formulae are based on the modelling of simply supported single span members with end fork conditions and with or without continuous lateral restraints, which are subjected to compression forces, end moments and/or transverse loads.

NOTE 2 In case the conditions of application expressed in (1) and (2) are not fulfilled, see 6.3.4.

(3) For members of structural systems the resistance check may be carried out on the basis of the individual single span members regarded as cut out of the system. Second order effects of the sway system (P-Δ-effects) have to be taken into account, either by the end moments of the member or by means of appropriate buckling lengths respectively.

(4) Members which are subjected to combined bending and axial compression should satisfy:

$$\frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{yz}\frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1 \quad (6.61)$$

$$\frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{zz}\frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1 \quad (6.62)$$

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

$\Delta M_{y,Ed}$, $\Delta M_{z,Ed}$ are the moments due to the shift of the centroidal axis according to 6.2.9.3 for class 4 sections, see Table 6.7,

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

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

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

Table 6.7: Values for $N_{Rk} = f_y A_i$, $M_{i,Rk} = f_y W_i$ and $\Delta M_{i,Ed}$

Class	1	2	3	4
A_i	A	A	A	A_{eff}
W_y	$W_{pl,y}$	$W_{pl,y}$	$W_{el,y}$	$W_{eff,y}$
W_z	$W_{pl,z}$	$W_{pl,z}$	$W_{el,z}$	$W_{eff,z}$
$\Delta M_{y,Ed}$	0	0	0	$e_{N,y}\, N_{Ed}$
$\Delta M_{z,Ed}$	0	0	0	$e_{N,z}\, N_{Ed}$

NOTE For members not susceptible to torsional deformation χ_{LT} would be $\chi_{LT} = 1{,}0$.

(5) The interaction factors k_{yy} , k_{yz} , k_{zy} , k_{zz} depend on the method which is chosen.

NOTE 1 The interaction factors k_{yy}, k_{yz}, k_{zy} and k_{zz} have been derived from two alternative approaches. Values of these factors may be obtained from Annex A (alternative method 1) or from Annex B (alternative method 2).

NOTE 2 The National Annex may give a choice from alternative method 1 or alternative method 2.

NOTE 3 For simplicity verifications may be performed in the elastic range only.

6.3.4 General method for lateral and lateral torsional buckling of structural components

(1) The following method may be used where the methods given in 6.3.1, 6.3.2 and 6.3.3 do not apply. It allows the verification of the resistance to lateral and lateral torsional buckling for structural components such as

– single members, built-up or not, uniform or not, with complex support conditions or not, or

– plane frames or subframes composed of such members, which are subject to compression and/or mono-axial bending in the plane, but which do not contain rotative plastic hinges.

NOTE The National Annex may specify the field and limits of application of this method.

(2) Overall resistance to out-of-plane buckling for any structural component conforming to the scope in (1) can be verified by ensuring that:

$$\frac{\chi_{op}\alpha_{ult,k}}{\gamma_{M1}} \geq 1{,}0 \qquad (6.63)$$

where $\alpha_{ult,k}$ is the minimum load amplifier of the design loads to reach the characteristic resistance of the most critical cross section of the structural component considering its in plane behaviour without taking lateral or lateral torsional buckling into account however accounting for all effects due to in plane geometrical deformation and imperfections, global and local, where relevant

χ_{op} is the reduction factor for the non-dimensional slenderness $\bar{\lambda}_{op}$, see (3), to take account of lateral and lateral torsional buckling

(3) The global non dimensional slenderness $\bar{\lambda}_{op}$ for the structural component should be determined from

$$\bar{\lambda}_{op} = \sqrt{\frac{\alpha_{ult,k}}{\alpha_{cr,op}}} \qquad (6.64)$$

where $\alpha_{ult,k}$ is defined in (2)

$\alpha_{cr,op}$ is the minimum amplifier for the in plane design loads to reach the elastic critical resistance of the structural component with regards to lateral or lateral torsional buckling without accounting for in plane flexural buckling

NOTE In determining $\alpha_{cr,op}$ and $\alpha_{ult,k}$ Finite Element analysis may be used.

(4) The reduction factor χ_{op} may be determined from either of the following methods:

a) the minimum value of

χ for lateral buckling according to 6.3.1

χ_{LT} for lateral torsional buckling according to 6.3.2

each calculated for the global non dimensional slenderness $\bar{\lambda}_{op}$

NOTE For example where $\alpha_{ult,k}$ is determined by the cross section check $\frac{1}{\alpha_{ult,k}} = \frac{N_{Ed}}{N_{Rk}} + \frac{M_{y,Ed}}{M_{y,Rk}}$ this method leads to:

$$\frac{N_{Ed}}{N_{Rk}/\gamma_{M1}} + \frac{M_{y,Ed}}{M_{y,Rk}/\gamma_{M1}} \leq \chi_{op} \qquad (6.65)$$

b) a value interpolated between the values χ and χ_{LT} as determined in a) by using the formula for $\alpha_{ult,k}$ corresponding to the critical cross section

NOTE For example where $\alpha_{ult,k}$ is determined by the cross section check $\frac{1}{\alpha_{ult,k}} = \frac{N_{Ed}}{N_{Rk}} + \frac{M_{y,Ed}}{M_{y,Rk}}$ this method leads to:

$$\frac{N_{Ed}}{\chi N_{Rk}/\gamma_{M1}} + \frac{M_{y,Ed}}{\chi_{LT} M_{y,Rk}/\gamma_{M1}} \le 1 \qquad (6.66)$$

6.3.5 Lateral torsional buckling of members with plastic hinges

6.3.5.1 General

(1)B Structures may be designed with plastic analysis provided lateral torsional buckling in the frame is prevented by the following means:

a) restraints at locations of "rotated" plastic hinges, see 6.3.5.2, and

b) verification of stable length of segment between such restraints and other lateral restraints, see 6.3.5.3

(2)B Where under all ultimate limit state load combinations, the plastic hinge is "not-rotated" no restraints are necessary for such a plastic hinge.

6.3.5.2 Restraints at rotated plastic hinges

(1)B At each rotated plastic hinge location the cross section should have an effective lateral and torsional restraint with appropriate resistance to lateral forces and torsion induced by local plastic deformations of the member at this location.

(2)B Effective restraint should be provided

- for members carrying either moment or moment and axial force by lateral restraint to both flanges. This may be provided by lateral restraint to one flange and a stiff torsional restraint to the cross-section preventing the lateral displacement of the compression flange relative to the tension flange, see Figure 6.5.
- for members carrying either moment alone or moment and axial tension in which the compression flange is in contact with a floor slab, by lateral and torsional restraint to the compression flange (e.g. by connecting it to a slab, see Figure 6.6). For cross-sections that are more slender than rolled I and H sections the distorsion of the cross section should be prevented at the plastic hinge location (e.g. by means of a web stiffener also connected to the compression flange with a stiff connection from the compression flange into the slab).

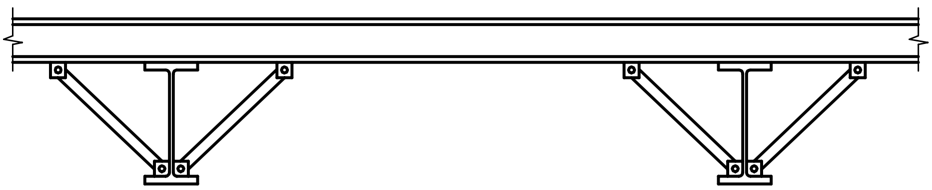
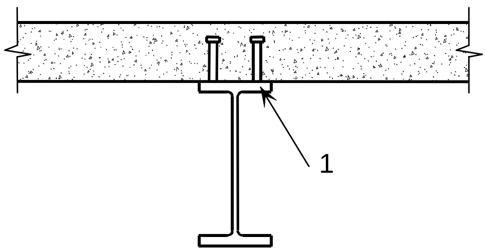


Figure 6.5: Typical stiff torsional restraint



1 compression flange

Figure 6.6: Typical lateral and torsional restraint by a slab to the compression flange

(3)B At each plastic hinge location, the connection (e.g. bolts) of the compression flange to the resisting element at that point (e.g. purlin), and any intermediate element (e.g. diagonal brace) should be designed to resist a local force equal to 2,5% of $N_{f,Ed}$ (defined in 6.3.5.2(5)B) transmitted by the flange in its plane and perpendicular to the web plane, without any combination with other loads.

(4)B Where it is not practicable providing such a restraint directly at the hinge location, it should be provided within a distance of h/2 along the length of the member, where h is its overall depth at the plastic hinge location.

(5)B For the design of bracing systems, see 5.3.3, it should be verified by a check in addition to the check for imperfection according to 5.3.3 that the bracing system is able to resist the effects of local forces Q_m applied at each stabilised member at the plastic hinge locations, where;

$$Q_m = 1{,}5\,\alpha_m \frac{N_{f,Ed}}{100} \tag{6.67}$$

where $N_{f,Ed}$ is the axial force in the compressed flange of the stabilised member at the plastic hinge location

α_m is according to 5.3.3(1)

NOTE For combination with external loads see also 5.3.3(5).

6.3.5.3 Verification of stable length of segment

(1)B The lateral torsional buckling verification of segments between restraints may be performed by checking that the length between restraints is not greater than the stable length.

For uniform beam segments with I or H cross sections with $\frac{h}{t_f} \le 40\varepsilon$ under linear moment and without significant axial compression the stable length may be taken from

$$\begin{aligned} L_{stable} &= 35\,\varepsilon\, i_z && \text{for } 0{,}625 \le \psi \le 1 \\ L_{stable} &= (60 - 40\psi)\,\varepsilon\, i_z && \text{for } -1 \le \psi \le 0{,}625 \end{aligned} \tag{6.68}$$

where $\varepsilon = \sqrt{\frac{235}{f_y\left[N/mm^2\right]}}$

$\psi = \frac{M_{Ed,min}}{M_{pl,Rd}}$ = ratio of end moments in the segment

NOTE B For the stable length of a segment see also Annex BB.3.

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

a) the restraint at the plastic hinge location should be within a distance h/2 along the length of the tapered segment, not the uniform segment

b) the compression flange of the haunch remains elastic throughout its length

NOTE B For more information see Annex BB.3.

6.4 Uniform built-up compression members

6.4.1 General

(1) Uniform built-up compression members with hinged ends that are laterally supported should be designed with the following model, see Figure 6.7.

1. The member may be considered as a column with a bow imperfection $e_0 = \dfrac{L}{500}$
2. The elastic deformations of lacings or battenings, see Figure 6.7, may be considered by a continuous (smeared) shear stiffness S_V of the column.

NOTE For other end conditions appropriate modifications may be performed.

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

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

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

(3) The design procedure is applicable to built-up members with lacings in two directions, see Figure 6.8.

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

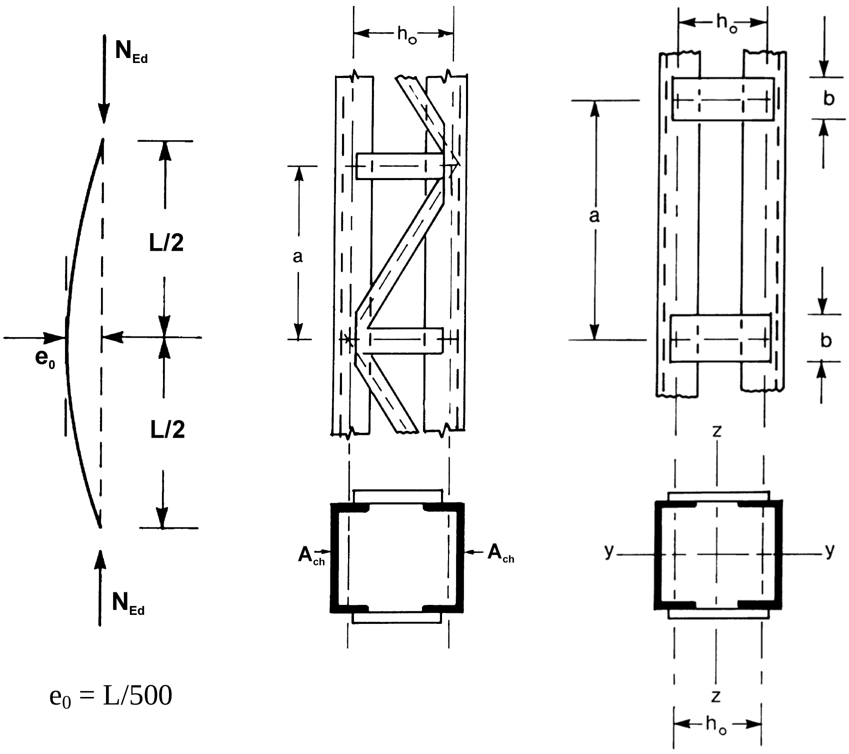


Figure 6.7: Uniform built-up columns with lacings and battenings

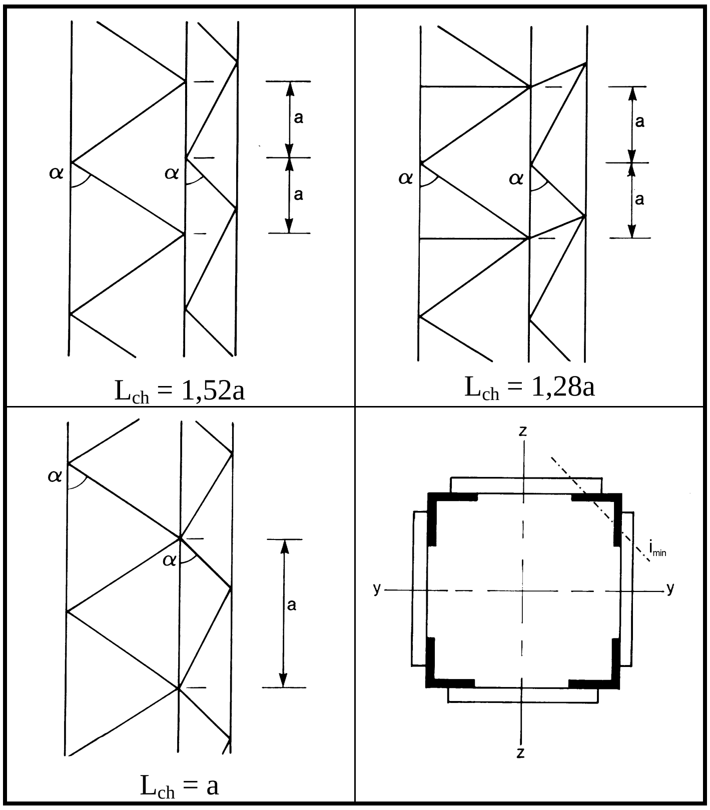


Figure 6.8: Lacings on four sides and buckling length L_{ch} of chords

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

(6) For a member with two identical chords the design force $N_{ch,Ed}$ should be determined from:

$$N_{ch,Ed} = 0{,}5N_{Ed} + \frac{M_{Ed} h_0 A_{ch}}{2I_{eff}} \tag{6.69}$$

where $M_{Ed} = \dfrac{N_{Ed} e_0 + M^I_{Ed}}{1 - \dfrac{N_{Ed}}{N_{cr}} - \dfrac{N_{Ed}}{S_v}}$

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

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

M_{Ed} is the design value of the maximum moment in the middle of the built-up member considering second order effects

M^I_{Ed} is the design value of the maximum moment in the middle of the built-up member without second order effects

h_0 is the distance between the centroids of chords

A_{ch} is the cross-sectional area of one chord

I_{eff} is the effective second moment of area of the built-up member, see 6.4.2 and 6.4.3

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

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

$$V_{Ed} = \pi \frac{M_{Ed}}{L} \tag{6.70}$$

6.4.2 Laced compression members

6.4.2.1 Resistance of components of laced compression members

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

NOTE Secondary moments may be neglected.

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

$$\frac{N_{ch,Ed}}{N_{b,Rd}} \leq 1{,}0 \tag{6.71}$$

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

and $N_{b,Rd}$ is the design value of the buckling resistance of the chord taking the buckling length L_{ch} from Figure 6.8.

(3) The shear stiffness S_V of the lacings should be taken from Figure 6.9.

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

$$I_{eff} = 0{,}5 h_0^2 A_{ch} \tag{6.72}$$

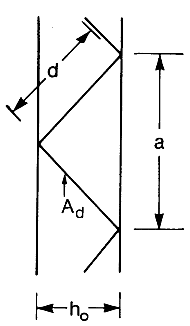
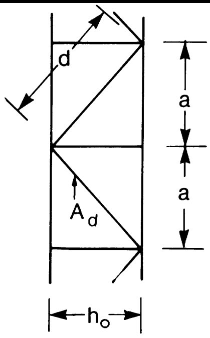
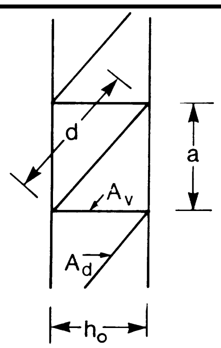
System			
S_V	$\dfrac{nEA_d a h_0^2}{2d^3}$	$\dfrac{nEA_d a h_0^2}{d^3}$	$\dfrac{nEA_d a h_0^2}{d^3\left[1+\dfrac{A_d h_0^3}{A_V d^3}\right]}$
n is the number of planes of lacings A_d and A_V refer to the cross sectional area of the bracings			

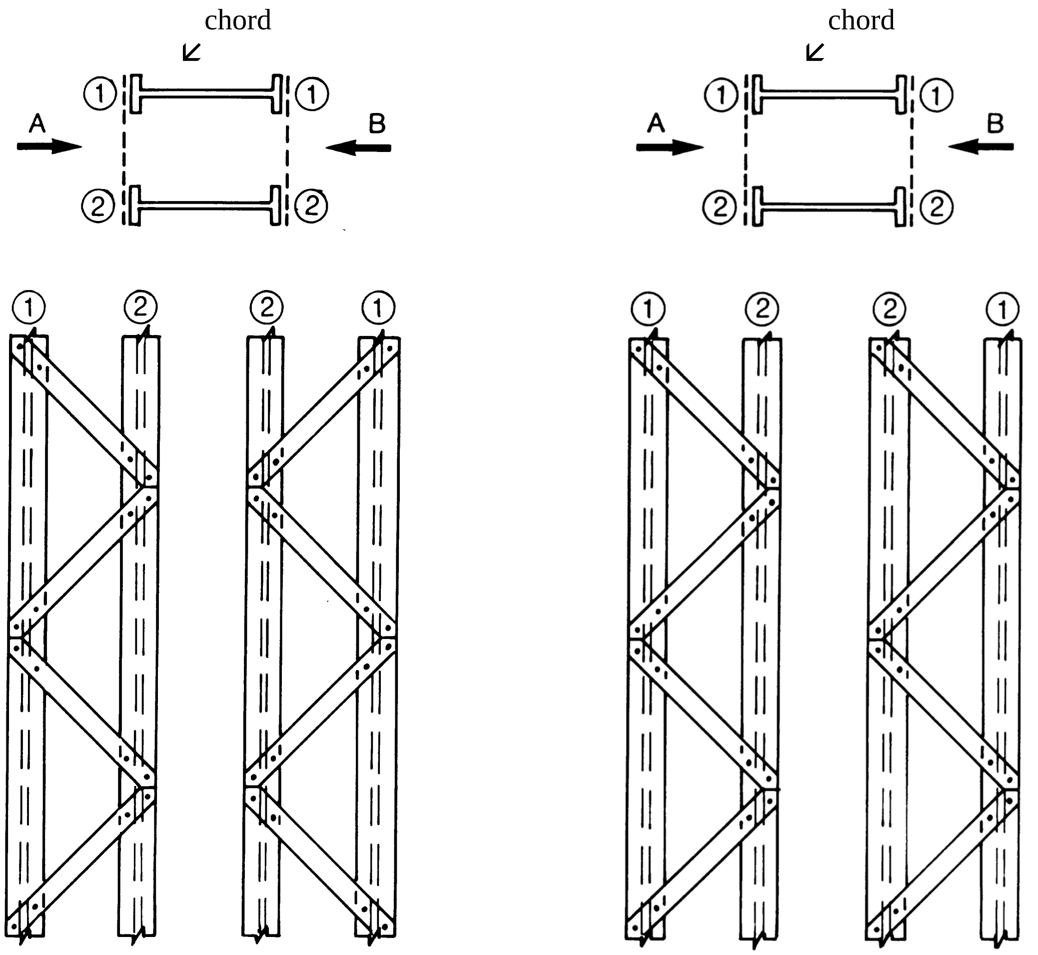
Figure 6.9: Shear stiffness of lacings of built-up members

6.4.2.2 Constructional details

(1) Single lacing systems in opposite faces of the built-up member with two parallel laced planes should be corresponding systems as shown in Figure 6.10(a), arranged so that one is the shadow of the other.

(2) When the single lacing systems on opposite faces of a built-up member with two parallel laced planes are mutually opposed in direction as shown in Figure 6.10(b), the resulting torsional effects in the member should be taken into account.

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



Lacing on face A Lacing on face B

a) Corresponding lacing system
(Recommended system)

Lacing on face A Lacing on face B

b) Mutually opposed lacing system
(Not recommended)

Figure 6.10: Single lacing system on opposite faces of a built-up member with two parallel laced planes

6.4.3 Battened compression members

6.4.3.1 Resistance of components of battened compression members

(1) The chords and the battens and their joints to the chords should be checked for the actual moments and forces in an end panel and at mid-span as indicated in Figure 6.11.

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

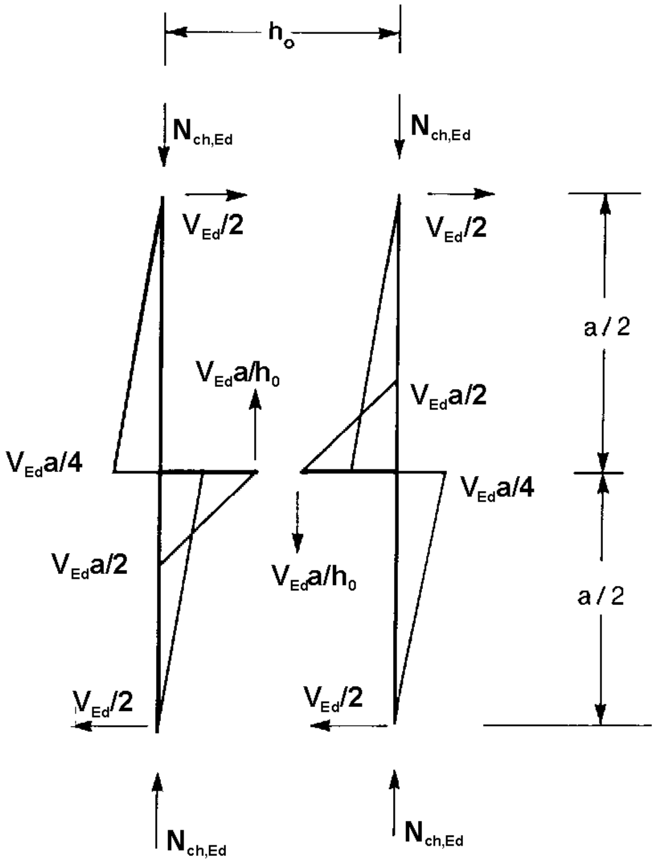


Figure 6.11: Moments and forces in an end panel of a battened built-up member

(2) The shear stiffness S_V should be taken as follows:

$$S_v = \frac{24EI_{ch}}{a^2\left[1+\frac{2I_{ch}}{nI_b}\frac{h_0}{a}\right]} \leq \frac{2\pi^2 EI_{ch}}{a^2} \tag{6.73}$$

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

$$I_{eff} = 0,5h_0^2 A_{ch} + 2\mu I_{ch} \tag{6.74}$$

where I_{ch} = in plane second moment of area of one chord

I_b = in plane second moment of area of one batten

μ = efficiency factor from Table 6.8

Table 6.8: Efficiency factor μ

criterion	efficiency factor μ
$\lambda \geq 150$	0
$75 < \lambda < 150$	$\mu = 2 - \frac{\lambda}{75}$
$\lambda \leq 75$	1,0
where $\lambda = \frac{L}{i_0}$; $i_0 = \sqrt{\frac{I_1}{2A_{ch}}}$; $I_1 = 0,5h_0^2 A_{ch} + 2I_{ch}$	

6.4.3.2 Design details

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

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

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

6.4.4 Closely spaced built-up members

(1) Built-up compression members with chords in contact or closely spaced and connected through packing plates, see Figure 6.12, or star battened angle members connected by pairs of battens in two perpendicular planes, see Figure 6.13 should be checked for buckling as a single integral member ignoring the effect of shear stiffness ($S_V = \infty$), when the conditions in Table 6.9 are met.

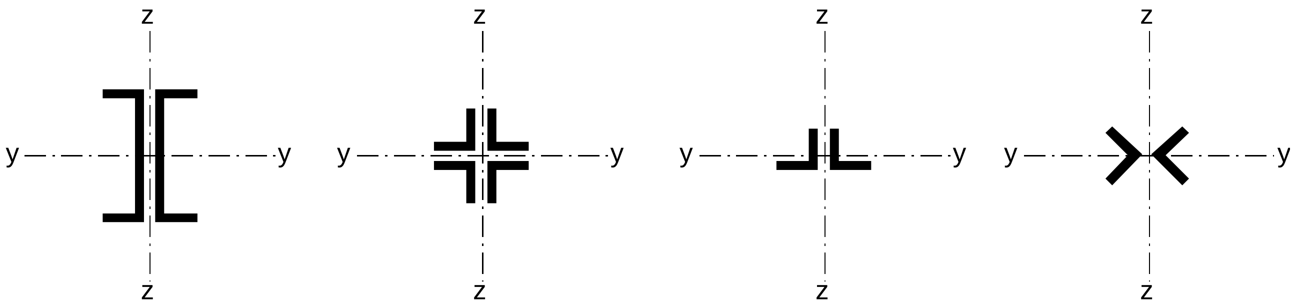


Figure 6.12: Closely spaced built-up members

Table 6.9: Maximum spacings for interconnections in closely spaced built-up or star battened angle members

Type of built-up member	Maximum spacing between interconnections *)
Members according to Figure 6.12 connected by bolts or welds	15 i_{min}
Members according to Figure 6.13 connected by pair of battens	70 i_{min}
*) centre-to-centre distance of interconnections i_{min} is the minimum radius of gyration of one chord or one angle	

(2) The shear forces to be transmitted by the battens should be determined from 6.4.3.1(1).

(3) In the case of unequal-leg angles, see Figure 6.13, buckling about the y-y axis may be verified with:

$$i_y = \frac{i_0}{1{,}15} \qquad (6.75)$$

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

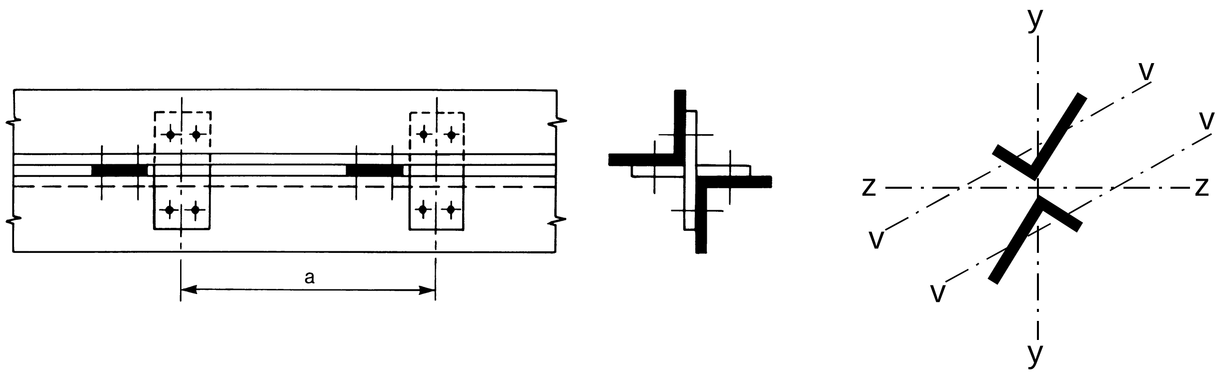


Figure 6.13: Star-battened angle members

7 Serviceability limit states

7.1 General

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

(2) The basic requirements for serviceability limit states are given in 3.4 of EN 1990.

(3) Any serviceability limit state and the associated loading and analysis model should be specified for a project.

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

7.2 Serviceability limit states for buildings

7.2.1 Vertical deflections

(1)B With reference to EN 1990 – Annex A1.4 limits for vertical deflections according to Figure A1.1 should be specified for each project and agreed with the client.

NOTE B The National Annex may specify the limits.

7.2.2 Horizontal deflections

(1)B With reference to EN 1990 – Annex A1.4 limits for horizontal deflections according to Figure A1.2 should be specified for each project and agreed with the client.

NOTE B The National Annex may specify the limits.

7.2.3 Dynamic effects

(1)B With reference to EN 1990 – Annex A1.4.4 the vibrations of structures on which the public can walk should be limited to avoid significant discomfort to users, and limits should be specified for each project and agreed with the client.

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

Annex A [informative] – Method 1: Interaction factors k_{ij} for interaction formula in 6.3.3(4)

Table A.1: Interaction factors k_{ij} (6.3.3(4))

Interaction factors	Design assumptions	
	elastic cross-sectional properties class 3, class 4	plastic cross-sectional properties class 1, class 2
k_{yy}	$C_{my} C_{mLT} \dfrac{\mu_y}{1-\dfrac{N_{Ed}}{N_{cr,y}}}$	$C_{my} C_{mLT} \dfrac{\mu_y}{1-\dfrac{N_{Ed}}{N_{cr,y}}} \dfrac{1}{C_{yy}}$
k_{yz}	$C_{mz} \dfrac{\mu_y}{1-\dfrac{N_{Ed}}{N_{cr,z}}}$	$C_{mz} \dfrac{\mu_y}{1-\dfrac{N_{Ed}}{N_{cr,z}}} \dfrac{1}{C_{yz}} 0{,}6 \sqrt{\dfrac{w_z}{w_y}}$
k_{zy}	$C_{my} C_{mLT} \dfrac{\mu_z}{1-\dfrac{N_{Ed}}{N_{cr,y}}}$	$C_{my} C_{mLT} \dfrac{\mu_z}{1-\dfrac{N_{Ed}}{N_{cr,y}}} \dfrac{1}{C_{zy}} 0{,}6 \sqrt{\dfrac{w_y}{w_z}}$
k_{zz}	$C_{mz} \dfrac{\mu_z}{1-\dfrac{N_{Ed}}{N_{cr,z}}}$	$C_{mz} \dfrac{\mu_z}{1-\dfrac{N_{Ed}}{N_{cr,z}}} \dfrac{1}{C_{zz}}$

Auxiliary terms:

$$\mu_y = \frac{1-\dfrac{N_{Ed}}{N_{cr,y}}}{1-\chi_y \dfrac{N_{Ed}}{N_{cr,y}}}$$

$$\mu_z = \frac{1-\dfrac{N_{Ed}}{N_{cr,z}}}{1-\chi_z \dfrac{N_{Ed}}{N_{cr,z}}}$$

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

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

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

C_{my} see Table A.2

$$a_{LT} = 1 - \frac{I_T}{I_y} \ge 0$$

$$C_{yy} = 1 + \left(w_y - 1\right)\left[\left(2 - \frac{1{,}6}{w_y} C_{my}^2 \bar{\lambda}_{max} - \frac{1{,}6}{w_y} C_{my}^2 \bar{\lambda}_{max}^2\right) n_{pl} - b_{LT}\right] \ge \frac{W_{el,y}}{W_{pl,y}}$$

with $b_{LT} = 0{,}5\, a_{LT}\, \bar{\lambda}_0^2 \dfrac{M_{y,Ed}}{\chi_{LT} M_{pl,y,Rd}} \dfrac{M_{z,Ed}}{M_{pl,z,Rd}}$

$$C_{yz} = 1 + \left(w_z - 1\right)\left[\left(2 - 14 \frac{C_{mz}^2 \bar{\lambda}_{max}^2}{w_z^5}\right) n_{pl} - c_{LT}\right] \ge 0{,}6 \sqrt{\frac{w_z}{w_y}} \frac{W_{el,z}}{W_{pl,z}}$$

with $c_{LT} = 10\, a_{LT} \dfrac{\bar{\lambda}_0^2}{5 + \bar{\lambda}_z^4} \dfrac{M_{y,Ed}}{C_{my} \chi_{LT} M_{pl,y,Rd}}$

$$C_{zy} = 1 + \left(w_y - 1\right)\left[\left(2 - 14 \frac{C_{my}^2 \bar{\lambda}_{max}^2}{w_y^5}\right) n_{pl} - d_{LT}\right] \ge 0{,}6 \sqrt{\frac{w_y}{w_z}} \frac{W_{el,y}}{W_{pl,y}}$$

with $d_{LT} = 2\, a_{LT} \dfrac{\bar{\lambda}_0}{0{,}1 + \bar{\lambda}_z^4} \dfrac{M_{y,Ed}}{C_{my} \chi_{LT} M_{pl,y,Rd}} \dfrac{M_{z,Ed}}{C_{mz} M_{pl,z,Rd}}$

$$C_{zz} = 1 + \left(w_z - 1\right)\left[\left(2 - \frac{1{,}6}{w_z} C_{mz}^2 \bar{\lambda}_{max} - \frac{1{,}6}{w_z} C_{mz}^2 \bar{\lambda}_{max}^2\right) n_{pl} - e_{LT}\right] \ge \frac{W_{el,z}}{W_{pl,z}}$$

with $e_{LT} = 1{,}7\, a_{LT} \dfrac{\bar{\lambda}_0}{0{,}1 + \bar{\lambda}_z^4} \dfrac{M_{y,Ed}}{C_{my} \chi_{LT} M_{pl,y,Rd}}$

Table A.1 (continued)

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

$\bar{\lambda}_0$ = non-dimensional slenderness for lateral-torsional buckling due to uniform bending moment, i.e. ψ_y =1,0 in Table A.2

$\bar{\lambda}_{LT}$ = non-dimensional slenderness for lateral-torsional buckling

For $\bar{\lambda}_0 = 0$:

$C_{my} = C_{my,0}$
$C_{mz} = C_{mz,0}$
$C_{mLT} = 1{,}0$

For $\bar{\lambda}_0 > 0$:

$$C_{my} = C_{my,0} + \left(1 - C_{my,0}\right)\frac{\sqrt{\varepsilon_y}\,a_{LT}}{1 + \sqrt{\varepsilon_y}\,a_{LT}}$$

$$C_{mz} = C_{mz,0}$$

$$C_{mLT} = C_{my}^2 \frac{a_{LT}}{\sqrt{\left(1 - \dfrac{N_{Ed}}{N_{cr,z}}\right)\left(1 - \dfrac{N_{Ed}}{N_{cr,T}}\right)}}$$

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

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

$N_{cr,y}$ = elastic flexural buckling force about the y-y axis

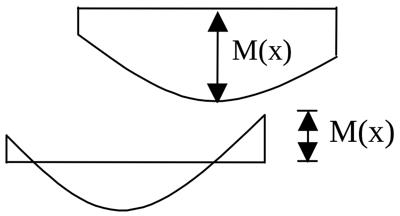
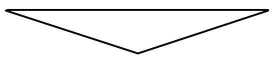
$N_{cr,z}$ = elastic flexural buckling force about the z-z axis

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

I_T = St. Venant torsional constant

I_y = second moment of area about y-y axis

Table A.2: Equivalent uniform moment factors $C_{mi,0}$

Moment diagram	$C_{mi,0}$
M_1  ψM_1 $-1 \le \psi \le 1$	$C_{mi,0} = 0{,}79 + 0{,}21\psi_i + 0{,}36(\psi_i - 0{,}33)\dfrac{N_{Ed}}{N_{cr.i}}$
	$C_{mi,0} = 1 + \left(\dfrac{\pi^2 EI_i \lvert\delta_x\rvert}{L^2 \lvert M_{i,Ed}(x)\rvert} - 1\right)\dfrac{N_{Ed}}{N_{cr.i}}$ $M_{i,Ed}$ (x) is the maximum moment $M_{y,Ed}$ or $M_{z,Ed}$ $\lvert\delta_x\rvert$ is the maximum member displacement along the member
 	$C_{mi,0} = 1 - 0{,}18\dfrac{N_{Ed}}{N_{cr.i}}$ $C_{mi,0} = 1 + 0{,}03\dfrac{N_{Ed}}{N_{cr.i}}$

Annex B [informative] – Method 2: Interaction factors k_{ij} for interaction formula in 6.3.3(4)

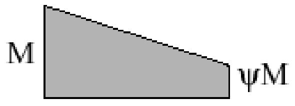
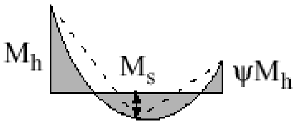
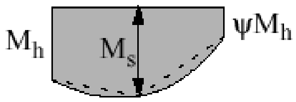
Table B.1: Interaction factors k_{ij} for members not susceptible to torsional deformations

Interaction factors	Type of sections	Design assumption: elastic cross-sectional properties class 3, class 4	Design assumption: plastic cross-sectional properties class 1, class 2
k_{yy}	I-sections RHS-sections	$C_{my}\left(1+0,6\bar{\lambda}_y \frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$ $\le C_{my}\left(1+0,6\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$	$C_{my}\left(1+\left(\bar{\lambda}_y-0,2\right)\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$ $\le C_{my}\left(1+0,8\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$
k_{yz}	I-sections RHS-sections	k_{zz}	$0,6\ k_{zz}$
k_{zy}	I-sections RHS-sections	$0,8\ k_{yy}$	$0,6\ k_{yy}$
k_{zz}	I-sections	$C_{mz}\left(1+0,6\bar{\lambda}_z \frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$ $\le C_{mz}\left(1+0,6\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$	$C_{mz}\left(1+\left(2\bar{\lambda}_z-0,6\right)\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$ $\le C_{mz}\left(1+1,4\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$
	RHS-sections		$C_{mz}\left(1+\left(\bar{\lambda}_z-0,2\right)\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$ $\le C_{mz}\left(1+0,8\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$
For I- and H-sections and rectangular hollow sections under axial compression and uniaxial bending $M_{y,Ed}$ the coefficient k_{zy} may be $k_{zy} = 0$.			

Table B.2: Interaction factors k_{ij} for members susceptible to torsional deformations

Interaction factors	Design assumptions: elastic cross-sectional properties class 3, class 4	Design assumptions: plastic cross-sectional properties class 1, class 2
k_{yy}	k_{yy} from Table B.1	k_{yy} from Table B.1
k_{yz}	k_{yz} from Table B.1	k_{yz} from Table B.1
k_{zy}	$\left[1-\frac{0,05\bar{\lambda}_z}{(C_{mLT}-0,25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right]$ $\ge\left[1-\frac{0,05}{(C_{mLT}-0,25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right]$	$\left[1-\frac{0,1\bar{\lambda}_z}{(C_{mLT}-0,25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right]$ $\ge\left[1-\frac{0,1}{(C_{mLT}-0,25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right]$ for $\bar{\lambda}_z < 0,4$: $k_{zy} = 0,6+\bar{\lambda}_z \le 1-\frac{0,1\bar{\lambda}_z}{(C_{mLT}-0,25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}$
k_{zz}	k_{zz} from Table B.1	k_{zz} from Table B.1

Table B.3: Equivalent uniform moment factors C_m in Tables B.1 and B.2

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

For members with sway buckling mode the equivalent uniform moment factor should be taken $C_{my} = 0,9$ or $C_{Mz} = 0,9$ respectively.

C_{my} , C_{mz} and C_{mLT} shall be obtained according to the bending moment diagram between the relevant braced points as follows:

moment factor	bending axis	points braced in direction
C_{my}	y-y	z-z
C_{mz}	z-z	y-y
C_{mLT}	y-y	y-y

Annex AB [informative] – Additional design provisions

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

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

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

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

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

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

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

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

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

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

BB.1 Flexural buckling of members in triangulated and lattice structures

BB.1.1 General

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

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

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

(4)B Under these conditions, in normal triangulated structures the buckling length L_{cr} of web members for in-plane buckling may be taken as 0,9L, except for angle sections, see BB.1.2.

BB.1.2 Angles as web members

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

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

$$\overline{\lambda}_{eff,y} = 0{,}50 + 0{,}7\overline{\lambda}_y \qquad \text{for buckling about y-y axis} \tag{BB.1}$$

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

where $\overline{\lambda}$ is as defined in 6.3.1.2.

(2)B When only one bolt is used for end connections of angle web members the eccentricity should be taken into account using 6.2.9 and the buckling length L_{cr} should be taken as equal to the system length L.

BB.1.3 Hollow sections as members

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

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

(3)B For latticed girders with parallel chords and braces, for which the brace to chord diameter or width ratio β is less than 0,6 the buckling length L_{cr} of a hollow section brace member without cropping or flattening, welded around its perimeter to hollow section chords, may generally be taken as 0,75L for both in-plane and out-of-plane buckling, unless smaller values may be justified by tests or by calculations.

NOTE The National Annex may give more information on buckling lengths.

BB.2 Continuous restraints

BB.2.1 Continuous lateral restraints

(1)B If trapezoidal sheeting according to EN 1993-1-3 is connected to a beam and the condition expressed by equation (BB.2) is met, the beam at the connection may be regarded as being laterally restrained in the plane of the sheeting.

$$S \geq \left(EI_w \frac{\pi^2}{L^2} + GI_t + EI_z \frac{\pi^2}{L^2} 0{,}25h^2 \right) \frac{70}{h^2} \quad \text{(BB.2)}$$

where S is the shear stiffness (per unit of beam length) provided by the sheeting to the beam regarding its deformation in the plane of the sheeting to be connected to the beam at each rib.

I_w is the warping constant

I_t is the torsion constant

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

L is the beam length

h is the depth of the beam

If the sheeting is connected to a beam at every second rib only, S should be substituted by 0,20S.

NOTE Eqation (BB.2) may also be used to determine the lateral stability of beam flanges used in combination with other types of cladding than trapezoidal sheeting, provided that the connections are of suitable design.

BB.2.2 Continuous torsional restraints

(1)B A beam may be considered as sufficiently restraint from torsional deformations if

$$C_{\vartheta,k} > \frac{M_{pl,k}^2}{EI_z} K_\vartheta K_\upsilon \quad \text{(BB.3)}$$

where $C_{\vartheta,k}$ = rotational stiffness (per unit of beam length) provided to the beam by the stabilising continuum (e.g. roof structure) and the connections

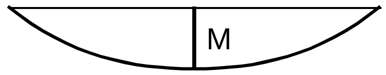
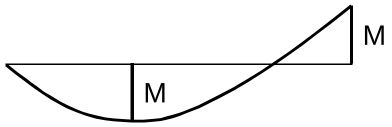
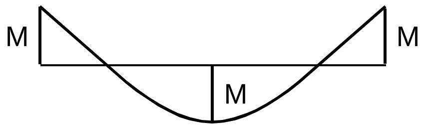
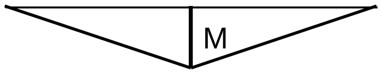
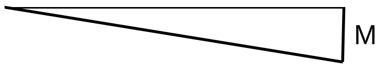
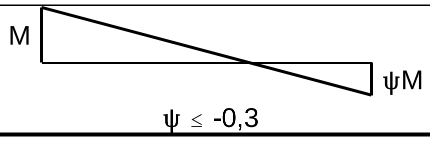
K_υ = 0,35 for elastic analysis

K_υ = 1,00 for plastic analysis

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

$M_{pl,k}$ = characteristic value of the plastic moment of the beam

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

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

(2)B The rotational stiffness provided by the stabilising continuum to the beam may be calculated from

$$\frac{1}{C_{\vartheta,k}} = \frac{1}{C_{\vartheta R,k}} + \frac{1}{C_{\vartheta C,k}} + \frac{1}{C_{\vartheta D,k}} \qquad \text{(BB.4)}$$

where $C_{\vartheta R,k}$ = rotational stiffness (per unit of the beam length) provided by the stabilising continuum to the beam assuming a stiff connection to the member

$C_{\vartheta C,k}$ = rotational stiffness (per unit of the beam length) of the connection between the beam and the stabilising continuum

$C_{\vartheta D,k}$ = rotational stiffness (per unit of the beam length) deduced from an analysis of the distorsional deformations of the beam cross sections, where the flange in compression is the free one; where the compression flange is the connected one or where distorsional deformations of the cross sections may be neglected (e.g. for usual rolled profiles) $C_{\vartheta D,k} = \infty$

NOTE For more information see EN 1993-1-3.

BB.3 Stable lengths of segment containing plastic hinges for out-of-plane buckling

BB.3.1 Uniform members made of rolled sections or equivalent welded I-sections

BB.3.1.1 Stable lengths between adjacent lateral restraints

(1)B Lateral torsional buckling effects may be ignored where the length L of the segment of a member between the restrained section at a plastic hinge location and the adjacent lateral restraint is not greater than L_m, where:

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

where N_{Ed} is the design value of the compression force [N] in the member

A is the cross section area [mm²] of the member

$W_{pl,y}$ is the plastic section modulus of the member

I_t is the torsion constant of the member

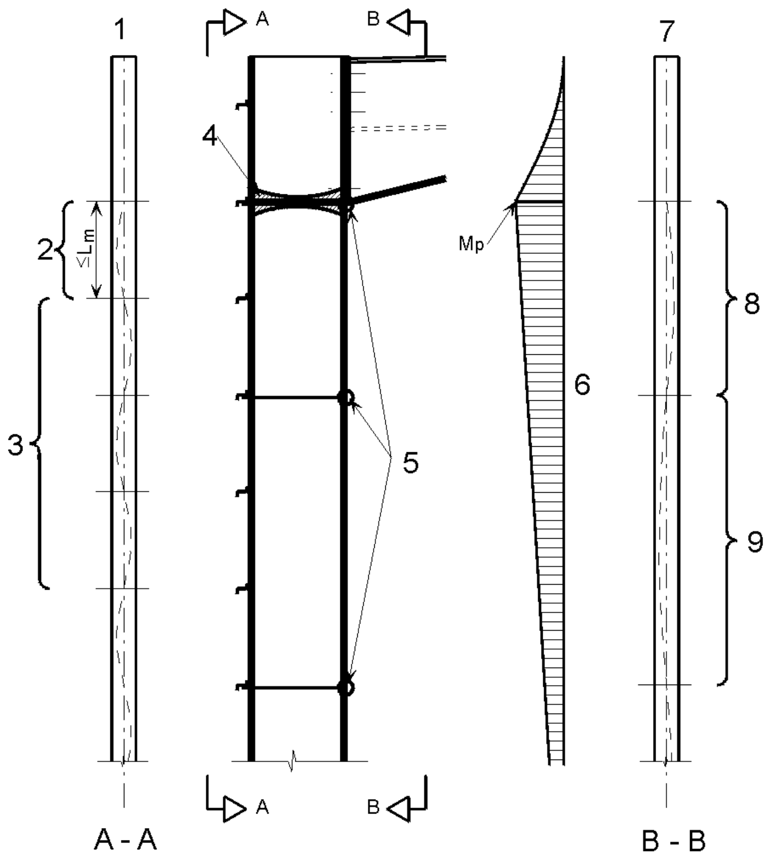
f_y is the yield strength in [N/mm²]

provided that the member is restrained at the hinge as required by 6.3.5 and that the other end of the segment is restrained

- either by a lateral restraint to the compression flange where one flange is in compression throughout the length of the segment,
- or by a torsional restraint,
- or by a lateral restraint at the end of the segment and a torsional restraint to the member at a distance that satisfies the requirements for L_s,

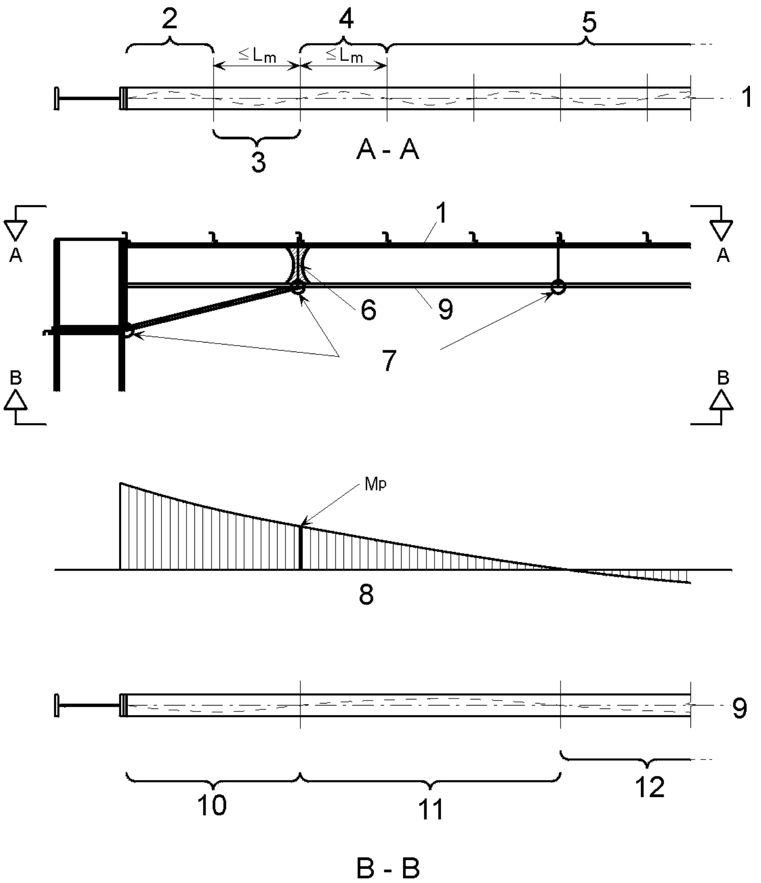
see Figure BB.1, Figure BB.2 and Figure BB3.

NOTE In general L_s is greater than L_m.



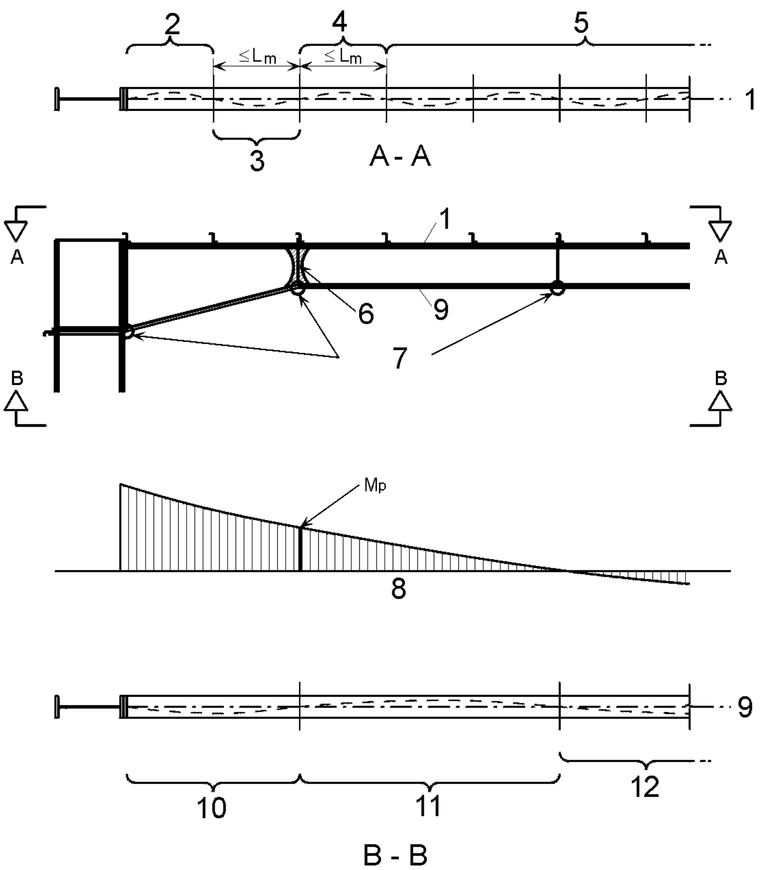
1 *tension flange*
2 *plastic stable length (see BB.3.1.1)*
3 *elastic section (see 6.3)*
4 *plastic hinge*
5 *restraints*
6 *bending moment diagram*
7 *compression flange*
8 *plastic with tension flange restraint, stable length = L_s (see BB.3.1.2, equation (BB.7) or (BB.8))*
9 *elastic with tension flange restraint (see 6.3), χ and χ_{LT} from N_{cr} and M_{cr} including tension flange restraint*

Figure BB.1: Checks in a member without a haunch



1 tension flange
2 elastic section (see 6.3)
3 plastic stable length (see BB.3.2.1) or elastic (see 6.3.5.3(2)B)
4 plastic stable length (see BB.3.1.1)
5 elastic section (see 6.3)
6 plastic hinge
7 restraints
8 bending moment diagram
9 compression flange
10 plastic stable length (see BB.3.2) or elastic (see 6.3.5.3(2)B)
11 plastic stable length (see BB.3.1.2)
12 elastic section (see 6.3), χ and χ_{LT} from N_{cr} and M_{cr} including tension flange restraint

Figure BB.2: Checks in a member with a three flange haunch



1 tension flange
2 elastic section (see 6.3)
3 plastic stable length (see BB.3.2.1)
4 plastic stable length (see BB.3.1.1)
5 elastic section (see 6.3)
6 plastic hinge
7 restraints
8 bending moment diagram
9 compression flange
10 plastic stable length (see BB.3.2)
11 plastic stable length (see BB.3.1.2)
12 elastic section (see 6.3), χ and χ_{LT} from N_{cr} and M_{cr} including tension flange restraint

Figure BB.3: Checks in a member with a two flange haunch

BB.3.1.2 Stable length between torsional restraints

(1)B Lateral torsional buckling effects may be ignored where the length L of the segment of a member between the restrained section at a plastic hinge location and the adjacent torsional restraint subject to a constant moment is not greater than L_k, provided that

- the member is restrained at the hinge as required by 6.3.5 and
- there are one or more intermediate lateral restraints between the torsional restraints at a spacing that satisfies the requirements for L_m, see BB.3.1.1,

where

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

(2)B Lateral torsional buckling effects may be ignored where the length L of the segment of a member between the restrained section at a plastic hinge location and the adjacent torsional restraint subject to a linear moment gradient and axial compression is not greater than L_s, provided that

- the member is restrained at the hinge as required by 6.3.5 and
- there are one or more intermediate lateral restraints between the torsional restraints at a spacing that satisfies the requirements for L_m, see BB.3.1.1,

where $$L_s = \sqrt{C_m}\, L_k \left(\frac{M_{pl,y,Rk}}{M_{N,y,Rk} + aN_{Ed}}\right) \qquad \text{(BB.7)}$$

C_m is the modification factor for linear moment gradient, see BB.3.3.1;

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

$M_{pl,y,Rk}$ is the characteristic plastic moment resistance of the cross section about the y-y axis

$M_{N,y,Rk}$ is the characteristic plastic moment resistance of the cross section about the y-y axis with reduction due to the axial force N_{Ed}

(3)B Lateral torsional buckling effects may be ignored where the length L of a segment of a member between the restrained section at a plastic hinge location and the adjacent torsional restraint subject to a non linear moment gradient and axial compression is not greater than L_s, provided that

- the member is restrained at the hinge as required by 6.3.5 and
- there are one or more intermediate lateral restraints between the torsional restraints at a spacing that satisfies the requirements for L_m, see BB3.1.1

where $$L_s = \sqrt{C_n}\, L_k \qquad \text{(BB.8)}$$

C_n is the modification factor for non-linear moment gradient, see BB.3.3.2,

see Figure BB.1, Figure BB.2 and Figure BB.3.

BB.3.2 Haunched or tapered members made of rolled sections or equivalent welded I-sections

BB.3.2.1 Stable length between adjacent lateral restraints

(1)B Lateral torsional buckling effects may be ignored where the length L of the segment of a member between the restrained section at a plastic hinge location and the adjacent lateral restraint is not greater than L_m, where

– for three flange haunches (see Figure BB.2)

$$L_m = \frac{38 i_z}{\sqrt{\frac{1}{57{,}4}\left(\frac{N_{Ed}}{A}\right) + \frac{1}{756\,C_1^2}\left(\frac{W_{pl,y}^2}{A I_t}\right)\left(\frac{f_y}{235}\right)^2}} \quad \text{(BB.9)}$$

– for two flange haunches (see Figure BB.3)

$$L_m = 0{,}85 \frac{38 i_z}{\sqrt{\frac{1}{57{,}4}\left(\frac{N_{Ed}}{A}\right) + \frac{1}{756\,C_1^2}\left(\frac{W_{pl,y}^2}{A I_t}\right)\left(\frac{f_y}{235}\right)^2}} \quad \text{(BB.10)}$$

where N_{Ed} is the design value of the compression force [N] in the member

$\frac{W_{pl,y}^2}{A I_t}$ is the maximum value in the segment

A is the cross sectional area [mm²] at the location where $\frac{W_{pl,y}^2}{A I_t}$ is a maximum of the tapered member

$W_{pl,y}$ is the plastic section modulus of the member

I_t is the torsional constant of the member

f_y is the yield strength in [N/mm²]

i_z is the minimum value of the radius of gyration in the segment

provided that the member is restrained at the hinge as required by 6.3.5 and that the other end of segment is restrained

– either by a lateral restraint to the compression flange where one flange is in compression throughout the length of the segment,

– or by a torsional restraint,

– or by a lateral restraint at the end of the segment and a torsional restraint to the member at a distance that satisfies the requirements for L_s.

BB.3.2.2 Stable length between torsional restraints

(1)B For non uniform members with constant flanges under linear or non-linear moment gradient and axial compression, lateral torsional buckling effects may be ignored where the length L of the segment of a member between the restrained section at a plastic hinge location and the adjacent torsional restraint is not greater than L_s, provided that

– the member is restrained at the hinge as required by 6.3.5 and

– there are one or more intermediate lateral restraints between the torsional restraints at a spacing that satisfies the requirements for L_m, see BB.3.2.1, where

– for three flange haunches (see Figure BB.2)

$$L_s = \frac{\sqrt{C_n}\ L_k}{c} \tag{BB.11}$$

– for two flange haunches (see Figure BB.3)

$$L_s = 0{,}85\frac{\sqrt{C_n}\ L_k}{c} \tag{BB.12}$$

where L_k is the length derived for a uniform member with a cross-section equal to the shallowest section, see BB.3.1.2, using the minimum value for i_z in the segment and the maximum value of (h/t_f) in the segment

C_n see BB.3.3.2

c is the taper factor defined in BB.3.3.3

BB.3.3 Modification factors for moment gradients in members laterally restrained along the tension flange

BB.3.3.1 Linear moment gradients

(1)B The modification factor C_m may be determined from

$$C_m = \frac{1}{B_0 + B_1\beta_t + B_2\beta_t^{\,2}} \tag{BB.13}$$

in which

$$B_0 = \frac{1+10\eta}{1+20\eta}$$

$$B_1 = \frac{5\sqrt{\eta}}{\pi+10\sqrt{\eta}}$$

$$B_2 = \frac{0{,}5}{1+\pi\sqrt{\eta}} - \frac{0{,}5}{1+20\eta}$$

$$\eta = \frac{N_{crE}}{N_{crT}}$$

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

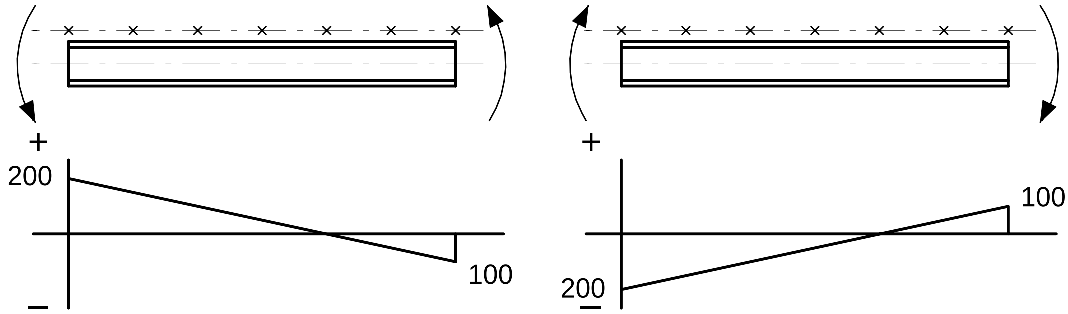
L_t is the distance between the torsional restraints

$$N_{crT} = \frac{1}{i_s^{\,2}}\left(\frac{\pi^2 EI_z a^2}{L_t^{\,2}} + \frac{\pi^2 EI_w}{L_t^{\,2}} + GI_t\right)$$ is the elastic critical buckling force for an I-section between restraints to both flanges at spacing L_t with intermediate lateral restraints to the tension flange.

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

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

β_t is the ratio of the algebraically smaller end moment to the larger end moment. Moments that produce compression in the non-restrained flange should be taken as positive. If the ratio is less than –1,0 the value of β_t should be taken as –1,0, see Figure BB.4.



$$\beta_t = \frac{-100}{+200} = -0{,}5$$

$$\beta_t = \frac{-200}{+100} = -2$$

but $\beta_t \geq -1{,}0$ thus $\beta_t = -1{,}0$

Figure BB.4: Value of β_t

BB.3.3.2 Non linear moment gradients

(1)B The modification factor C_n may be determined from

$$C_n = \frac{12}{\left[R_1 + 3R_2 + 4R_3 + 3R_4 + R_5 + 2(R_S - R_E)\right]} \tag{BB.14}$$

in which R_1 to R_5 are the values of R according to (2)B at the ends, quarter points and mid-length, see Figure BB.5, and only positive values of R should be included.

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

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

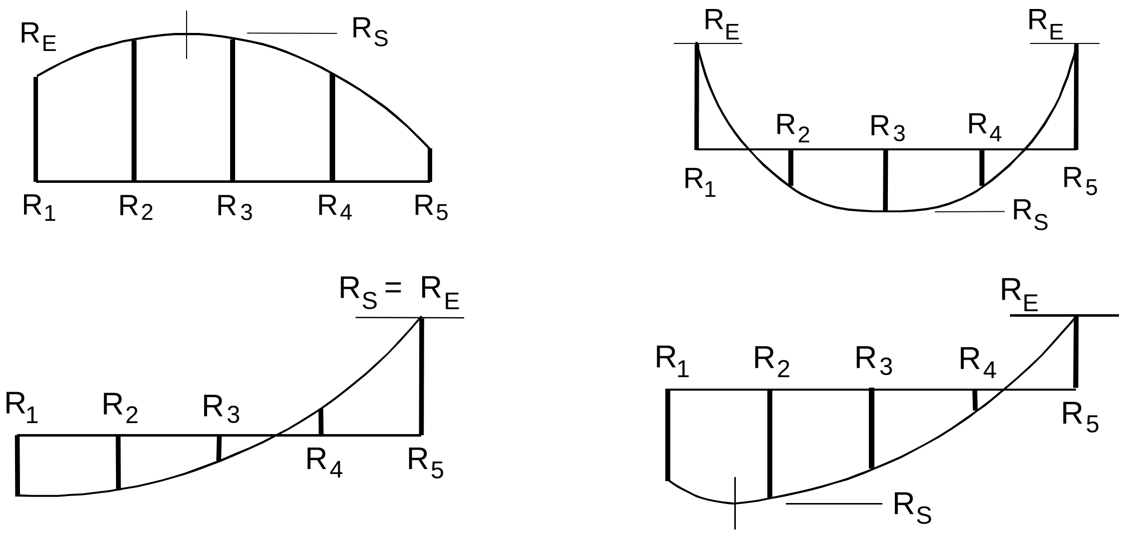


Figure BB.5: Moment ratios

(2)B The value of R should be obtained from:

$$R = \frac{M_{y,Ed} + a\,N_{Ed}}{f_y W_{pl,y}} \tag{BB.15}$$

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

BB.3.3.3 Taper factor

(1)B For a non uniform member with constant flanges, for which $h \geq 1{,}2b$ and $h/t_f \geq 20$ the taper factor c should be obtained as follows:

– for tapered members or segments:

$$c = 1 + \frac{3}{\left(\frac{h}{t_f} - 9\right)} \left(\frac{h_{max}}{h_{min}} - 1\right)^{2/3} \tag{BB.16}$$

– for haunched members or segments:

$$c = 1 + \frac{3}{\left(\frac{h}{t_f} - 9\right)} \left(\frac{h_h}{h_s}\right)^{2/3} \sqrt{\frac{L_h}{L_y}} \tag{BB.17}$$

where h_h is the additional depth of the haunch or taper, see Figure BB.6;

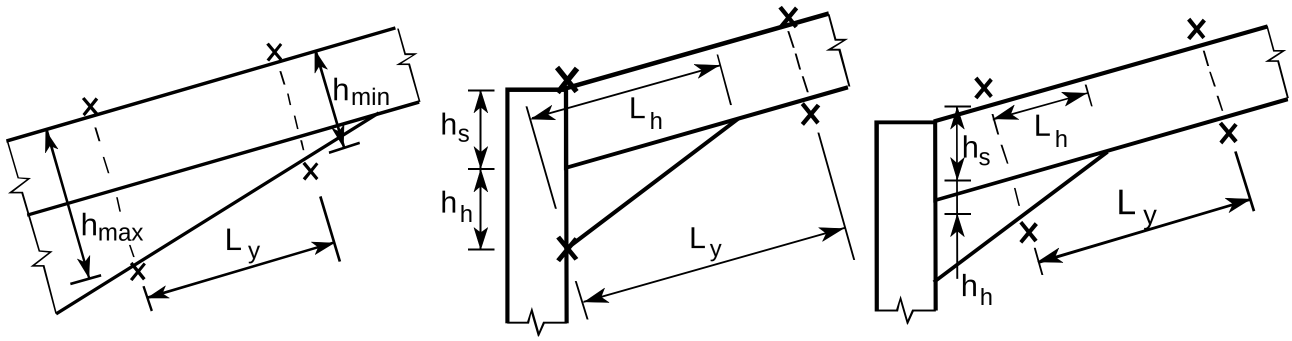
h_{max} is the maximum depth of cross-section within the length L_y, see Figure BB.6;

h_{min} is the minimum depth of cross-section within the length L_y, see Figure BB.6;

h_s is the vertical depth of the un-haunched section, see Figure BB.6;

L_h is the length of haunch within the length L_y, see Figure BB.6;

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



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

x = restraint

Figure BB.6: Dimensions defining taper factor

EUROPEAN STANDARD
NORME EUROPÉENNE
EUROPÄISCHE NORM

prEN 1993-1-11 : 20xx

13 February 2003

UDC

Descriptors:

English version

Eurocode 3 : Design of steel structures

Part 1.11 : Design of structures with tension components

Calcul des structures en acier	Bemessung und Konstruktion von Stahlbauten
Partie 1.11 :	Teil 1.11 :
Calcul des structures à câbles ou éléments tendus	Bemessung und Konstruktion von Stahlbauten mit Zuggliedern

CEN

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

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

Ref. No. EN 1993-1.11 : 20xx. E

Contents

National annex for EN 1993-1-11

This standard gives alternative procedures, values and recommendations for classes with notes indicating where national choices may have to be made. Therefore the National Standard implementing EN 1993-1-11 should have a National Annex containing all Nationally Determined Parameters to be used for the design of steel structures to be constructed in the relevant country.

National choice is allowed in EN 1993-1-11 through:

- 2.3.6(1)
- 2.3.6(2)
- 2.4.1(1)
- 3.1(1)
- 4.4(2)
- 4.5(4)
- 6.2(2)
- 6.3.2(1)
- 6.3.4(1)
- 6.4.1(1)
- 7.2(2)
- A.4.5.1(1)
- A.4.5.2(1)

1 General

1.1 Scope

(1) This Part 11 of prEN1993-1 gives design rules for structures with tension components made of steel which due to their connections with the structure are adjustable and replaceable.

NOTE Due to the requirement of adjustability and replaceability such tension components are mostly prefabricated products delivered to site and installed into the structure as a whole. Tension components that are not adjustable or replaceable, e.g. air spun cables of suspension bridges, are outside the scope of this part though rules of this part may be applicable.

(2) This part also gives rules for determining the technical requirements for prefabricated tension components for a structure and for assessing their safety, serviceability and durability.

(3) This part deals with tension components as given in Table 1.1.

Table 1.1: Groups of tension components

Group	Main tensile element	Component
A	rod (bar)	tension rod (bar) system, prestressing bar
B	circular wire	spiral strand rope
	circular and Z-wires	full-locked coil rope
	circular wire and stranded wire	strand rope
C	circular wire	parallel wire strand (PWS)
	circular wire	bundle of parallel wires (air spun)
	seven wire (prestressing) strand	bundle of parallel strands

NOTE 1 Group A products comprising tension rod systems and bars in general have a single solid round cross section connected to end terminations by threads. They are mainly used as

- bracings for roofs, walls, girders
- stays for roof elements, pylons
- inline tensioning for steel-wooden truss and steel structures, space frames

NOTE 2 Group B products comprising spiral strand, ropes, full locked coil ropes and strand ropes are composed of wires which are anchored in sockets or other end terminations.

Spiral strand ropes are mainly used as

- stay cables — for aerials, smoke stacks, masts and bridges
- carrying cables and edge cables — for light weight structures
- hangers or suspenders — for suspension bridges
- stabilizing cables — for cable nets and wood and steel trusses
- hand-rail cables — for banisters, balconies, bridge rails and guardrails

They are fabricated mainly in the diameter range of 5 mm to ~160 mm.

Full locked coil ropes are mainly used as

- stay cables, suspension cables and hangers — for bridge construction
- suspension cables and stabilizing cables — in cable trusses
- edge cables — for cable nets
- stay cables — for pylons, masts, aerials

They are fabricated in the diameter range of 20 to ~180 mm.

Structural wire ropes are mainly used as

- stay cables for masts, aerials
- hangers for suspension bridges
- damper / spacer tie cables between stay cables
- edge cables for fabric membranes
- rail cables for banister, balcony, bridge and guide rails.

NOTE 3 For Group B see EN 12385-2.

NOTE 4 Group C products comprising bundles of parallel wires and bundles of parallel strands need individual or collective anchoring and individual or collective protection.

Bundles of parallel wires are mainly used as stay cables, main cables for suspension bridges and external tendons.

Bundles of parallel strands are mainly used as stay cables or external tendons for concrete, composite and steel bridges.

(4) The types of termination dealt with in this part for Group B and C products are

- metal and resin socketing, see EN 13411-4
- socketing with cement grout
- ferrules and ferrule securing, see EN 13411-3
- swaged sockets and swaged fitting
- U-bolt wire rope grips, see EN 13411-5
- anchoring for bundles with wedges, cold formed button heads for wires and nuts for bars.

NOTE For terminology see 1.3 and Annex C.

1.2 Normative references

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

NOTE The Eurocodes were published as European Prestandards. The following European Standards which are published or in preparation are cited in normative clauses:

EN 10138 Prestressing steels

Part 1 General requirements
Part 2 Wire
Part 3 Strand
Part 4 Bars

EN 10244 Steel wire and wire products – Non-ferrous metallic coatings on steel wire

Part 1 General requirements
Part 2 Zinc and zinc alloy coatings
Part 3 Aluminium coatings

EN 10264 Steel wire and wire products – Steel wire for ropes

Part 1 General requirements
Part 2 Cold drawn non alloyed steel wire for ropes for general applications

Part 3 Cold drawn and cold profiled non alloyed steel wire for high tensile applications

Part 4 Stainless steel wires

EN 12385 Steel wire ropes – safety

Part 1 General requirements

Part 2 Definitions, designation and classification

Part 3 Information for use and maintenance

Part 4 Stranded ropes for general lifting applications

Part 10 Spiral ropes for general structural applications

EN 13411 Terminations for steel wire ropes – safety

Part 3 Ferrules and ferrule-securing

Part 4 Metal and resin socketing

Part 5 U-bolt wire rope grips

1.3 Terms and definitions

(1) For the purpose of this European Standard the following definitions apply.

1.3.1
strand
an element of rope normally consisting of an assembly of wires of appropriate shape and dimensions laid helically in the same or opposite direction in one or more layers around a centre

1.3.2
stranded rope
an assembly of several strands laid helically in one or more layers around a core (single layer rope) or centre (rotation-resistant or parallel-closed rope)

1.3.3
spiral rope
an assembly of at least two layers of wires laid helically over a centre, usually around a wire

1.3.4
spiral strand rope
spiral rope comprising only round wires

1.3.5
full-locked coil rope
spiral rope having an outer layer of full lock (Z-shaped) wires

1.3.6
fill factor f
the ratio of the sum of the nominal metallic cross sectional areas of all the wires in a rope (A) and the circumscribed area (A_u) of the rope based on its nominal diameter (d)

1.3.7
spinning loss factor k
reduction factor for rope construction included in the breaking force factor K

1.3.8

breaking force factor (K)

an empirical factor used in the determination of minimum breaking force of a rope and obtained from the product of fill factor (f) for the rope class or construction, spinning loss factor (k) for the rope class or construction and the constant $\pi / 4$

$$K = \frac{\pi \, f \, k}{4}$$

NOTE K-factors for the more common rope classes and constructions are given in the appropriate part of EN 12385.

1.3.9

minimum breaking force (F_{min})

specified value in kN, below which the measured breaking force (F_{min}) is not allowed to fall in a prescribed breaking force test and normally obtained by calculation from the product of the square of the nominal diameter (d) [mm], the rope grade (R_r) [N/mm²] and the breaking force factor (K)

$$F_{min} = \frac{d^2 \, R_r \, K}{1000}$$

1.3.10

rope grade (R_r)

a level of requirement of breaking force which is designated by a number (e.g. 1770 [N/mm²], 1960 [N/mm²])

NOTE This does not imply that the actual tensile strength grades of the wires in the rope are necessarily of this grade.

1.3.11

unit weight (w)

value of selfweight of rope (w) [kN/(m mm²)] related to the metallic cross section (A_m) [mm²] and the unit length [m] taking account of the weight densities of steel and the corrosion protection system

1.3.12

cable

main tension component in a structure (e.g. a stay cable bridge) which may consist of a rope, strand or bundles of parallel wires or strands

1.4 Symbols

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

Draft note: Will be added later.

2 Basis of Design

2.1 General

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

(2) The supplementary provisions for tensile components given in this chapter should be applied.

(3) As durability is a main concern for the design of tension components the following distinction according to exposure classes may be applied:

Table 2.1: Exposure classes

fatigue action	corrosion action	
	not exposed to external climate	exposed to external climate (rain)
no significant fatigue action	class 1	class 2
mainly axial fatigue action	class 3	class 4
axial and lateral fatigue actions (wind and wind & rain)	–	class 5

(4) It is assumed that the connections of tensile components to the structure are such that the components are replaceable and adjustable.

2.2 Requirements

(1) The following limit states should be considered in choosing tensile components:

1. ULS: Fracture of the component by reaching the design tension resistance taking account of durability, see section 6.

   NOTE The design tension resistance is determined from testing including durability provisions.

2. SLS: Limitation of stress levels and strain levels in the component for controlling the durability behaviour, see section 7.

   NOTE Because of the dominant durability aspect serviceability checks may be relevant and may cover ULS-verifications.

3. Fatigue: Limitation of stress ranges from axial load fluctuations as well as oscillations from wind or wind-rain, see sections 8 and 9.

   NOTE Due to the model uncertainties concerning the excitement mechanisms and the fatigue resistance of cables the fatigue check also presupposes a SLS-check, see section 7.

(2) Depending on the type and system of the structure, and the effects of possible detension of a tensile component below a minimum stress (e.g. uncontrolled stability or fatigue or damages to structural or non structural parts), the tensile components are mostly preloaded by deformations imposed to the structure (prestressing).

As a consequence the permanent actions are composed of actions from gravity loads “G” and prestress “P”, that shall be considered as a single permanent action “G+P” to which the relevant partial factors γ_{Gi} should be applied, see section 5.

NOTE For other materials and ways of construction other rules for combination of "G" and "P" may apply.

(3) Any attachments to prefabricated tensile components as saddles or clamps shall be designed for ultimate limit states and serviceability limit states using the hypothetical occurrence of breaking strength or proof strength of cables as actions, see section 6. For fatigue see EN 1993-1-9

NOTE Fatigue action on the ropes is controlled by the minimum radius in the saddle or anchorage area.

2.3 Actions

2.3.1 Selfweight of tensile components

(1) The characteristic value of selfweight of tensile components and their attachments shall be determined from the cross-sectional make up and the density of the materials unless data are given in the relevant part of EN 12385.

(2) For spiral strands, locked coil strands or structural wire ropes the following approximate expression for the nominal selfweight g_k may be used:

$$g_k = w\ A_m \tag{2.1}$$

where A_m is the metallic cross-section in mm²

w [kN/(m mm²)] is the unit weight that takes the weight densities of steel and of the corrosion protection system into account, see Table 2.2

(3) A_m may be determined from

$$A_m = \frac{\pi\, d^2}{4} f \tag{2.2}$$

where d is the external diameter of rope or strand, including sheathing for corrosion protection if used

f is the fill-factor, see Table 2.2

Table 2.2: Unit weight w and fill-factors f

		Fill factor f							unit weight $w \times 10^{-4}$ $\left[\frac{kN}{m \times mm^2}\right]$
		Core wires + 1 layer z-wires	Core wires + 2 layer z-wires	Core wires + >2 layer z-wires	Number of wire layers around core wire				
					1	2	3-6	>6	
1	Spiral strand ropes				0,77	0,76	0,75	0,73	0,83
2	Full locked coil ropes	0,81	0,84	0,88					0,83
3	Strand wire ropes with CWR				0,56				0,93

(4) For parallel wire ropes or parallel strand ropes the metallic cross section may be determined from

$$A_m = n\ a_m \tag{2.3}$$

where n is the number of identical wires or strands of which the rope is constituted

a_m is the cross section of a wire (derived from its diameter) or a (prestressing) strand (derived from the appropriate standard)

(5) For group C tension components the self weight should be determined from the steel weight of individual wires or strands and the weight of the corrosion protection (HDPE, wax etc.)

2.3.2 Wind actions

(1) The wind effects taken into account shall include:

- the static effects of wind drag on the cables, see EN 1991-1-4, including deflections and possible resulting bending effects near the ends of the cable,
- aerodynamic and other excitation leading to possible oscillation of the cables, see section 8.

2.3.3 Ice loads

(1) For ice loading see Annex B to EN 1993-3-1.

2.3.4 Thermal actions

(1) The thermal actions to be taken into account shall include the effects of differential temperatures between the cables and the rest of the structure.

(2) For a cable in a structure exposed to weather conditions the actions from differential temperature according to EN 1991-1-5 should be used.

2.3.5 Prestressing

(1) The preloads in cables shall be determined such, that when all the permanent actions are applied, the structure adopts the required geometric profile and stress distribution.

(2) To ensure this objective, facilities for prestressing and for adjustment of the cables shall be provided and the characteristic value of the preload shall be taken as required to achieve the objective of (1) at the limit state under consideration.

(3) If adjustment of the cables is not provided allowance shall be made, in calculating the design values of the total effects of the permanent actions and preload for the range of error that may occur in the prestressing together with any errors that may arise in the precamber of the structure.

NOTE From a sensivity check tolerances may be derived.

2.3.6 Rope removal and replacement

(1) The replacement of any one rope should be taken into account in the design in a transient design situation.

NOTE The National Annex may define the transient loading conditions and partial factors for replacement.

(2) A sudden removal of any one rope should be taken into account in the design in an accidental design situation.

NOTE 1 The National Annex may define where such an accidental design situation applies and also give the protection aims and loading conditions.

NOTE 2 In the absence of a more exact analysis the dynamic effect of a sudden removal may conservatively be allowed by using the design effect

$$E_d = k\, E_{d2} - E_{d1} \qquad (2.4)$$

where $k = 1{,}5$ to $2{,}0$

E_{d1} represents the design effects with all cables intact;

E_{d2} represents the design effects with the relevant cable removed.

2.3.7 Fatigue loads

(1) For fatigue loads see EN 1991.

2.4 Design situations and partial factors

2.4.1 Transient design situation during the construction phase

(1) For the construction phase the partial factor for permanent loads (G+P) may be adapted to the particular design situation and limit state model.

NOTE The National Annex may define partial factors γ_G for the construction phase. Recommended values γ_G are

$\gamma_G = 1,10$ for a short time period (only a few hours) for the instalment of first strand in strand by strand installations

$\gamma_G = 1,20$ for the instalment of other strands

$\gamma_G = 1,00$ for favourable effects.

2.4.2 Persistent situations during service

(1) For the assessment of ULS, SLS and fatigue partial factors γ_M may be dependant on

- the severeness of the test conditions for qualification tests
- the measures to suppress bending effects with model uncertainties.

NOTE Indications for γ_M values are given in section 6.

3 Material

3.1 Strength of steels and wires

(1) The characteristic values f_y and f_u for steels and $f_{0,2}$ or $f_{0,1}$ and f_u for wires shall be taken from the relevant technical specifications.

NOTE 1 For steels see EN1993-1-1 and EN1993-1-4.

NOTE 2 For wires see EN 10264, Part 1 to Part 4.

NOTE 3 For ropes see EN 12385, Part 4 and Part 10.

NOTE 4 For terminations see EN 13411-3.

NOTE 5 For strands see EN 10138-3.

NOTE 6 The National Annex may give a maximum value f_u for durability reasons. The following values are recommended:

– steel wires	round wires:	nominal tensile grade:	1770 N/mm²
	Z-wires:	nominal tensile grade:	1570 N/mm²
– stainless steel wires:	round wires:	nominal tensile grade:	1450 N/mm²

3.2 Modulus of elasticity

3.2.1 Tension rod systems (Group A)

(1) The modulus of elasticity for tension rod systems may be taken as E = 210000 N/mm² ; for tension rod systems made of stainless steels see EN 1993-1-4.

3.2.2 Ropes (Group B)

(1) The modulus of elasticity for locked coil strands, bundles of strands, bars and wires should be derived from tests.

NOTE 1 The modulus of elasticity can depend on the stress level and whether the cable is subject to first loading or repeated loading.

NOTE 2 The modulus of elasticity for locked coil strands, strands or bundles of strands, bars and wires is multiplied with the metallic cross section A_m to obtain the tension stiffness of the cable.

(2) The modulus of elasticity used for structural analysis for persistent design situations during service should be obtained for each cable type and diameter by measuring the secant modulus after a sufficient number (at least 5) load cycles between F_{inf} and F_{sup} to get stable values. Herein F_{inf} is the minimum cable force under characteristic permanent and variable actions. F_{sup} is the maximum cable force under characteristic permanent and variable actions.

(3) For short test samples (sample length ≤ 10 x lay length) a smaller creep than for long cables should be expected.

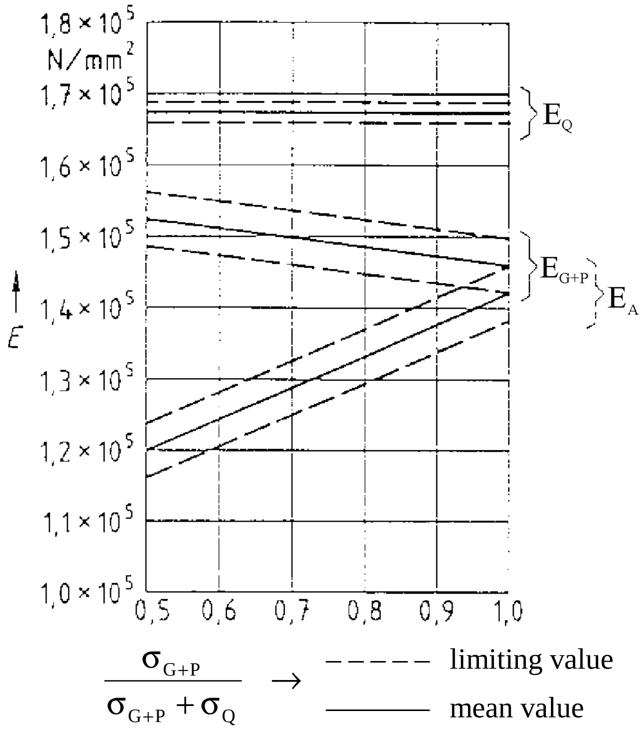
NOTE 1 In the absence of more accurate values this effect may be taken into account in cutting to length by applying an additional shortening of 0,15 mm/m.

NOTE 2 Notional values of moduli of elasticity for first estimations when test results are not available are given in Table 3.1. For further information see EN 10138.

Table 3.1: Notional values for the modulus of elasticity E_Q in the range of variable loads Q

	High strength tension component	E_Q [kN/mm²]	
		steel wires	stainless steel wires
1	Spiral strand ropes	150 ± 10	130 ± 10
2	Full locked coil ropes	160 ± 10	–
3	Strand wire ropes with CWR	100 ± 10	90 ± 10
4	Strand wire ropes with CF	80 ± 10	–
5	Bundle of parallel wires	205 ± 5	–
6	Bundle of parallel strands	195 ± 5	–

NOTE 3 Notional values for the modulus of elasticity E for the design of full locked coil ropes for bridges are given in Figure 3.1. These estimations apply to cyclic loading and unloading between 30 % and 40 % of the calculative breaking strength F_{uk}.



σ_{G+P} stress under characteristic permanent actions

σ_Q maximum stress under characteristic variable actions

E_Q modulus of elasticity for persistent design situations during service

E_{G+P} modulus of elasticity for an appropriate analysis for transient design situations during construction phase up to permanent load G+P

E_A modulus of elasticity for cutting to length

Figure 3.1: Notional values of modulus of elasticity E for the design of full locked coil ropes for bridges

NOTE 4 As non prestretched cables of group B exhibit both elastic and permanent deformations in the first loading it is recommended to prestretch such cables before or after installation by cyclic loading by up to $0{,}45\sigma_{uk}$. For cutting to length cables should be prestreched, with a precision depending on adjustment possiblities.

NOTE 5 For Figure 3.1 the following assumptions apply:

- the lay length is above 10 × the diameter
- the minimum value of stress is 100 N/mm²

The minimum value of stress is the lower bound of the elastic range.

3.2.3 Bundles of parallel wires or strands (Group C)

(1) The modulus of elasticity for bundles of parallel wires and strands may be taken from EN 10138 or Table 3.1.

3.3 Thermal expansion coefficient

(1) The thermal expansion coefficient shall be taken as

$$\alpha_T = 12 \times 10^{-6}\ K^{-1} \quad \text{for steel wires}$$
$$\alpha_T = 16 \times 10^{-6}\ K^{-1} \quad \text{for stainless steel wires} \qquad (3.1)$$

3.4 Cutting to length of tension components Group B

(1) Strands may be marked to length only for cutting at a prescribed cutting load.

(2) For an exact cutting to length the following data should be considered:

– measured values of the elongation between σ_A and σ_{G+P} after cyclic loading according to 3.2.2(2)

– difference between design temperature (normally 10°) and ambient temperature when cutting to length if the length is measured by temperature invariant measurement devices like fixed marks, invar measure tapes etc.

– long term cable creep under loads

– additional elongation of cable after installation of cable clamps

– setting of the pouring cone after cooling of molten metal and after initial load is applied.

NOTE The cable creep and cone setting will take place after a certain time and loading in the structure, so that higher loads may be needed during erection as the cable creep has not finished yet.

3.5 Lengths and fabrication tolerances

(1) The total length and all measuring points for the attachment of saddles and clamps should be marked under defined preload.

NOTE Additional control markings allows for a later check of exact length after parts have been installed.

(2) The fabrication tolerances shall be considered after prestretching and cyclic loading and unloading.

(3) When structures are sensitive to deviations from nominal geometrical values (e.g. by creep), adjusting devices should be provided.

3.6 Friction coefficients

(1) For the friction between full locked coil cables and steel attachments (clamps, saddles, fittings) the friction coefficient should be determined from tests. In the absence of tests $\mu = 0{,}1$ may be used.

(2) For other types of cables the friction coefficient should be determined from tests, see Annex A.

4 Durability for wires and ropes / strands

4.1 General

(1) Because of the crucial importance of corrosion protection for the safety of ropes with exposure classes 2, 4 and 5 according to Table 2.1 the corrosion protection barrier of a cable should be composed of the following measures:

1. Corrosion protection of each individual wire
2. Corrosion protection of the rope interior with inner filler to avoid the ingress of moisture.
3. Corrosion protection of rope surface

(2) The tension components of group C according to Table 1.1 should have two independent corrosion protection barriers with an interface or inner filler between the barriers.

(3) At clamps and anchorages additional corrosion measures should be applied at the structure to prevent water penetration.

(4) Also basic rules for transport, storage and handling should be observed.

NOTE See Annex B.

4.2 Corrosion protection of each individual wire

(1) All steel wires of group B and C should be coated with zinc or zinc alloy.

(2) For group B zinc or zinc alloy coating for round wires should be in accordance with EN 10264-2, class A. Shaped wires should comply with EN 10264-3, class A.

NOTE Z-shaped wires generally are heavy galvanized with a coating thickness up to 300g/m² to allow for thickness reduction on sharp corners.

(3) Zinc-aluminised wires (Zn95Al5) provide much improved corrosion protection than heavy galvanizing with the same coating thickness. Round and Z-shaped wires can be coated with a Zn95Al5 basis weight.

(3) For group C wires should comply with EN 10138.

4.3 Corrosion protection of the rope / strand / cable interior

(1) All interior voids of the cables should be filled with an active or passive inner filling that should not be displaced by water, heat or vibration.

NOTE 1 Active fillers are suspensions of zinc in polyurethane-oil.

NOTE 2 Passive inner fillers can be permanent elastic-plastic wax or aluminium flake in hydrocarbon resin.

NOTE 3 Inner filling applied during stranding of cable can extrude when cable is loaded (bleeding).

NOTE 4 When selecting the appropriate inner filling any possible incompatibility with other corrosion protection components applied to the cable later, should be checked.

4.4 Corrosion protection of the surface of single strands, cables or ropes and components

(1) After the installation of the cables and the erection of the structure in general an additional corrosion protection on ropes and cables need to be applied to compensate for damaging of the initial corrosion protection and for the expense of zinc.

NOTE This protection may consist of polyethylene sheathing or zinc loaded paint. For polyethylene, the minimum thickness is equal to the strand outer diameter divided by 15 and shall not by less than 3 mm.
The following minimum layer thicknesses may be applied to paints:

- 2 prime coats, Polyurethane with zinc dust 50 µm each
- 2 finishing coats. Polyurethane with iron mica, 125 µm each.

(2) The choice of cables with stainless steel wires and stainless steel terminations without additional corrosion protection should comply with the relevant corrosion resistance class.

NOTE 1 The National Annex may specify the corrosion resistance classes for stainless steel.

NOTE 2 The zinc-aluminium eutectoid of Zn95Al5-coated wires provides an up to 3 times better resistance compared with heavy zinc coated wires under equal conditions.

Draft note: To be coordinated with EN 1993-1-4 / EN ISO 12944-2.

4.5 Corrosion protection of bundles of parallel wires or bundles of parallel strands

(1) Cables formed as parallel wire strands should normally be sheathed using steel or polyethylene tube complying to relevant standards with the space between the inside of the sheath and the cable then filled with a suitable corrosion protection compound or cement grout.

(2) Alternatively polyethylene sheathing extruded directly or epoxy coating over the individual strands or cables may be used.

(3) The sheaths used for sheathed strand should be made completely impermeable at the connections to the anchorages. The joints shall be designed so that they do not break, when the sheath is subjected to tension.

(4) Void fillers should be

– continuous hydrophobic material with no detrimental interaction with the main tensile elements.

NOTE 1 Continuous hydrophobic materials are soft fillers as grease, wax or soft resin or hard fillers as cement if their suitability is proved by tests. The choice of materials may be given in the National Annex.

– circulation of dry air or nitrogen.

NOTE 2 Corrosion protection of main cables of suspension bridges requires a special approach. After compacting the main cable into a cross-sectional area as small as possible the cable gets a close wrapping with tensioned galvanized soft wire laid in a suitable paste sufficient to fill completely the voids between the outer cable wires and the wrapping wire. After removal of the surplus paste from outside of the wrapping wire the zinc coated surface is cleaned and subsequently painted. Special treatment is required for suspension bridge cable achorages where the wrapping wire is removed. Dehuminification of the air around the wires is a common method of protection.

4.6 Corrosion protection measures directly at the structure

(1) Provision should be taken to prevent rain water running down the cable from entering at clamps, saddles and anchorings.

(2) Therefore the transitions cable/component shall be sealed carefully with permanent elastic material. Also gaps between clamps should be sealed as well.

5 Structural analysis of cable structures

5.1 General

(1) The analysis should be made for the relevant design situations

1. for the transient construction phase
2. for the persistent service conditions after completion of the construction

for the limit states considered.

5.2 Transient design situations during the construction phase

(1) The confectioning of cables, the geometry of the structure, and the construction process with prestressing shall be planned such, that the conditions for prestress and selfweight satisfy the following conditions:

– attainment of the required geometric form

– attainment of a permanent stress situation that satisfied the serviceability and ultimate limit state conditions for all design situations.

(2) For complying with control measures (e.g. measurements of shape, gradients, deformations frequencies or forces) all calculations should be carried out with characteristic values of permanent loads, imposed deformations and any imposed action step by step to achieve the final required permanent stage.

(3) When nonlinear action effects from deformations are significant during construction these effects shall be taken into account, see 5.4.

(4) Where ultimate limit states during prestressing are controlled by differential effects of the action "G" and "P" (e.g. for concrete parts), the partial factor γ_P = 1,00 should be applied to "P".

5.3 Persistent design situation during service

(1) For any persistent design situation during the service phase the permanent actions "G" from gravity and preloads or prestressing "P" shall be combined in a single permanent action "G + P" corresponding to the permanent shape of the structure.

(2) For the verification of serviceability limit states the action "G + P" shall be included in the relevant combination of action; for the verification of the ultimate limit states EQU or STR (see EN 1990) the permanent actions "G + P" shall be multiplied with the partial factor $\gamma_{G\ sup}$, when the effects of permanent action and of variable actions are unfavourable. In case the permanent actions "G + P" are favourable they should be multiplied with the partial factor $\gamma_{G\ inf}$.

(3) When nonlinear action effects from deformations are significant during service these effects shall be taken into account, see 5.4.

5.4 Nonlinear effects from deformations

5.4.1 General

(1) For structures with tension components the effects of deformations from catenary effects and shortening and lengthening of the components including creep shall be taken into account.

5.4.2 Catenary effects

(1) Catenary effects may be taken into account by applying to each cable or segment of cable the effective modulus

$$E_t = \frac{E}{1 + \dfrac{w^2\, \ell^2\, E}{12\, \sigma^3}} \tag{5.1}$$

E is the modulus of elasticity of the cable

w is the unit weight according to Table 2.2

ℓ is the horizontal span of the cable

σ is the stress in the cable. For situations according to 5.3 it is σ_{G+P} .

5.4.3 Effects of deformations on the structure

(1) For the application of 2nd order analysis deformations due to variable loads should refer to the initial geometrical form of the structure required for the permanent loading corresponding to "G + P" for a given temperature T_0.

(2) For the 2nd order calculations for serviceability limit states and for sublinear behaviour in ultimate limit states the characteristic load combination may be applied to determine the action effects.

(3) For 2nd order calculations for overlinear behaviour of structures in ultimate limit states the required permanent geometrical form of the structure at the reference temperature T_0 may be associated with the stress situation from "γ_G (G + P)" and design values of variable actions $\gamma_Q Q_{k1} + \gamma_Q \psi_2 Q_{k2}$ may be applied together with appropriate assumptions for imperfections of the structure.

6 Ultimate limit states

6.1 Tension rod systems

(1) Tension rod systems should be designed for ULS according to EN 1993-1-1 or EN 1993-1-4 depending on the steel used.

6.2 Ropes and prestressing bars

(1) For the ultimate limit state it shall be verified that

$$\frac{F_{Ed}}{F_{Rd}} \leq 1 \tag{6.1}$$

where F_{Ed} is the design value of the axial rope force

F_{Rd} is the design value of tension resistance.

(2) The design value of the tension resistance F_{Rd} shall be determined from the characteristic value of the breaking strength F_{uk} and the characteristic value of ther proof strength F_k.

$$F_{Rd} = \min\left\{\frac{F_{uk}}{1{,}5\,\gamma_R}; \frac{F_k}{\gamma_R}\right\} \tag{6.2}$$

where F_{uk} is the characteristic value of the breaking strength,

F_k is the characteristic value of the 0,2% proof strength $F_{0,2k}$ or of the 0,1% proof strength $F_{0,1k}$ determined according to the requirement of the standard relevant for the tension component, e.g. by testing for ropes or by calculation for bars,

γ_R is the partial factor.

NOTE 1 F_{uk} corresponds to the characteristic value of the ultimate tensile strength.

NOTE 2 Table 6.1 gives information on the proof strength F_k relevant for the tension component.

Table 6.1: Groups of tension components and relevant proof strength

Group	relevant standard	proof strength F_k
A	EN 10138-1	$F_{0,1k}$ *)
B	EN 10264	$F_{0,2k}$
C	EN 10138-1	$F_{0,1k}$
*) For prestressing bars see EN 1993-1-1 and EN 1993-1-4		

NOTE 3 F_k is not directly related to ULS. By the check against F_k it is verified that the rope will remain elastic even when the actions attain their design value. For ropes (e.g. full locked coil ropes) where $F_k \geq \frac{F_{uk}}{1{,}50}$ this check is not relevant.

NOTE 4 By tests on delivery it is demonstrated that the experimental values F_{uke} and F_{ke} satisfy the requirement

$F_{uke} > F_{uk}$,

$F_{ke} > F_k$,

see EN 12385, Part 1.

NOTE 5 The partial factor γ_R may be determined in the National Annex. It may be dependent on whether or not measures are applied at the rope ends to reduce bending moments from cable rotations, see 7.1(4). The values for γ_R in Table 6.2 are recommended.

Table 6.2: Recommended γ_R – values

Detailing measures to suppress bending stresses ahead of anchorage	γ_R
Yes	0,90
No	1,00

(3) For prestressing bars and group C tension components the characteristic value of the calculative breaking strength should be determined from

$$F_{uk} = A_m f_{uk} \tag{6.3}$$

where A_m is the metallic cross-section, see 2.3.1

f_{uk} is the characteristic value of the tensile strength of rods, wires or (prestressing) strands of which the tension component is constituted according to the relevant standard

(4) For group B tension components F_{uk} should be calculated as

$$F_{uk} = F_{min} k_e \tag{6.4}$$

where F_{min} is determined according to EN 12385-2 as

$$F_{min} = \frac{K\, d^2\, R_r}{1000}\ [KN] \tag{6.5}$$

where K is the minimum breaking force factor taking account of the spinning loss,

d is the nominal diameter of the rope

R_r is the rope grade

k_e is given in Table 6.3 for some types of end terminations

NOTE K, d, R_r are specified for all ropes in the EN 12385-2.

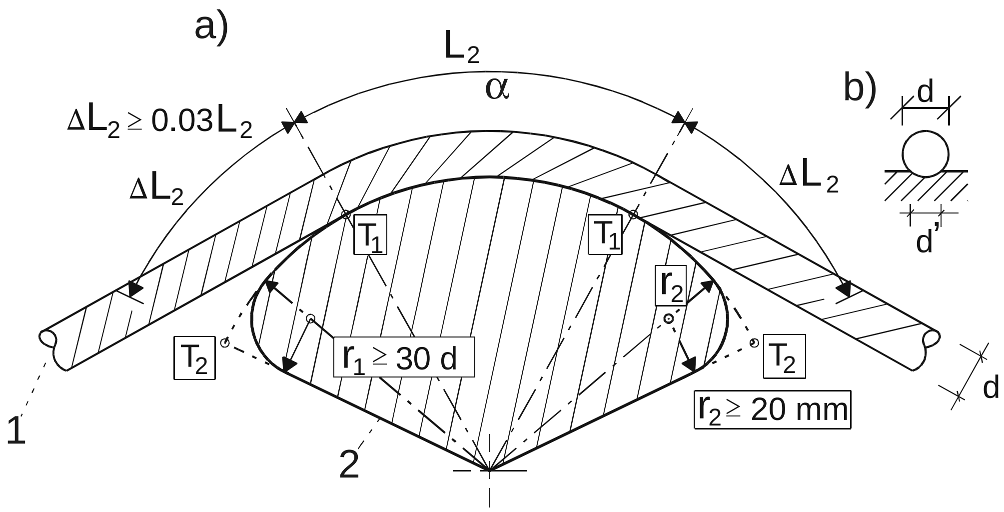
Table 6.3: Loss factors k_e

Type of termination	Loss factor k_e
Metal filled socket	1,0
Resin filled socket	1,0
Ferrule-secured eye	0,9
Swaged socket	0,9
U-bolt grip	0,8 *)
*) For U-bolt grip a reduction of preload is possible.	

6.3 Saddles

6.3.1 Geometrical conditions

(1) In order to reduce the characteristic breaking resistance of strand or rope by no more than 3%, the saddle should be proportioned as shown in Figure 6.1. Where the following conditions are satisfied stresses due to curvature of wires may be neglected in the design.



1 *cable*
2 *saddle*
L_2 *length of strand between the two theoretical points of tangency T_1 under unfavourable characteristic loads including catenary effects*
ΔL_2 *additional length of wrap*

Figure 6.1: Radii of saddle and definition of bedding

(2) The radius of the saddle should be $r_1 \geq 30d$ or $r_1 \geq 400\varnothing$, whichever is greater, where $\varnothing$ is the diameter of wire.

(3) The radius may be reduced to $r_1 \geq 20d$ when the bedding of the rope on at least 60% of the diameter is performed by soft metal or spray zinc coating with a minimum thickness of 1 mm.

(4) Smaller radii may be used for spiral ropes where justified by tests.

NOTE The position of the points T_1 and T_2 should be determined for the relevant load cases taking the movement of bearings and cables (catenary) into account.

6.3.2 Slipping of cables round saddles

(1) To ensure that slip does not occur it shall be verified that for the highest value of the ratio

$$\max\left\{\frac{F_{Ed1}}{F_{Ed2}}\right\} \tag{6.6}$$

where F_{Ed1} and F_{Ed2} are the design values of the greater and smaller force in the cable on either side of the saddle

the following equation is satisfied:

$$\max\left\{\frac{F_{Ed1}}{F_{Ed2}}\right\} \leq e^{\left\{\frac{\mu\alpha}{\gamma_{M,fr}}\right\}} \tag{6.7}$$

where μ is the coefficient of friction between cable and saddle

α is the angle in radians, of the cable passing over the saddle

$\gamma_{M,fr}$ is the partial factor for friction.

NOTE The partial factor γ_{Mfr} may be given in the National Annex. The value γ_{Mfr} = 1,65 is recommended.

(2) If (1) is not satisfied, an additional radial force F_r should be provided by clamps such that

$$\frac{F_{Ed1} - \frac{k\,F_r\,\mu}{\gamma_{Mfr}}}{F_{Ed2}} \leq e^{\left[\frac{\mu\alpha}{\gamma_{M,fr}}\right]} \tag{6.8}$$

where k is normally taken as 1,0 but may be taken as 2, if full friction can be guaranteed at both the saddle grooves and the clamp itself and F_r should not exceed the resistance of the cable to clamping forces, see 6.3.3

$\gamma_{M,fr}$ is the partial factor for friction resistance

(3) In determining F_r from preloaded bolts the following effects should be considered:

a) long term creep

b) reduction of diameter if tension is increased

c) compaction/bedding down of cable or ovalisation

d) reduction of preload in clamp bolts by external loads

e) differential temperature.

6.3.3 Transverse pressure

(1) The transverse pressure q due to the radial clamping force F_r should be limited to

$$\frac{q_{Ed}}{q_{Rd}} \leq 1 \tag{6.9}$$

where $q_{Ed} = \frac{F_r}{d^{/} L_2}$ with $0{,}6d \leq d^{/} \leq d$, see Figure 6.1b)

$q_{Rd} = \frac{q_{Rk}}{\gamma_{M,bed}}$ limit value of transverse pressure determined from tests

$\gamma_{M,bed}$ is the partial factor.

NOTE For calculating q the pressure from F_{Ed1} need not be considered as it is limited by the rules in 6.3.1.

(2) In the absence of tests values for q_R the limit values of transverse pressure q_{Rk} are given in Table 6.4.

NOTE 1 The limit values q_{Rk} in combination with γ_M = 1,00 would lead to a reduction of the breaking strength of the cable by no more than 3%.

Table 6.4: Limit values q_{Rk}

Type of cable	Limit pressure q_{Rk} [N/mm²]	
	Steel clamps and saddles	Cushioned clamps and saddles
Full locked coil rope	40	100
Spiral strand rope	25	60

NOTE 2 Cushioned clamps have a layer of soft metal or spray zinc coating with a minimum thickness of 1 mm.

6.3.4 Design of saddles

(1) Cable saddles should be designed for a cable force of k times the characteristic breaking strength F_{uk} of the cables.

NOTE The factor k may be specified in the National Annex. The value k = 1,05 is recommended.

6.4 Clamps

6.4.1 Slipping of clamps

(1) Where clamps shall transmit longitudinal forces to a cable and the parts are not mechanically keyed together, slipping shall be prevented by verifying

$$F_{Ed_{||}} \leq \frac{\left(F_{Ed_{\perp}} + F_r\right)\mu}{\gamma_{M,fr}} \tag{6.10}$$

where $F_{Ed_{||}}$ is the component of external design load parallel to the cable

$F_{Ed_{\perp}}$ is the component of the external design load perpendicular to the cable

F_r is the clamping force considered that may be reduced by items in 6.3.2(3)

μ is the coefficient of friction

$\gamma_{M,fr}$ is the partial factor for friction

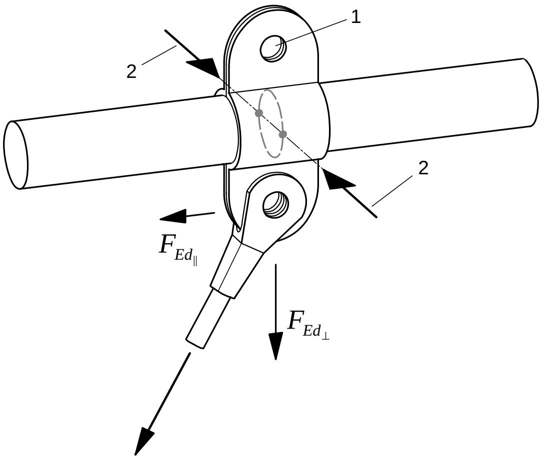
NOTE The partial factor $\gamma_{M,fr}$ may be determined in the National Annex. The partial factor $\gamma_{M,fr}$ = 1,65 is recommended.

6.4.2 Transverse pressure

(1) For $F_{Ed_{\perp}}$ or $F_{Ed_{\perp}} + F_r$ (whichever is greater) the transverse pressure should be limited according to 6.3.3.

6.4.3 Design of clamps

(1) Clamps and their fittings, anchoring secondary elements (e.g. hangers) on a main cable (e.g. a suspension cable) shall be designed as for end terminations for the secondary element for a hypothetical force equivalent to the proof force F_k of the secondary element clamped, see Figure 6.2.



1 preloaded bolts
2 preload F_r from preloaded bolts

Figure 6.2: Clamp

NOTE F_k is not directly related to ULS. By the use of F_k capacity design is applied.

7 Serviceability limit states

7.1 Serviceability criteria

(1) The following serviceability criteria should be considered.

1. Deformations or vibrations of the structure that may influence the design of the structure
2. The behaviour of high strength tension components themselves that are related to their elastic behaviour and durability

(2) Limits for deformations or vibrations may result in stiffness requirement governed by the structural system, the dimensions and the preloading of high strength tension components, and by the slipping resistance of attachments.

(3) Limits to retain elastic behaviours and durability are related to maximum and minimum values of stresses for serviceability load combinations.

(4) Bending stresses in the anchorage zone may be reduced by constructive measures (e.g. noeprene pads for transverse loading).

7.2 Recommendations for stress limits

(1) Stress limits may be introduced for rare load combinations for the following purposes:

- to keep stresses in the elastic range for the relevant design situations during construction and in the service phase,
- to limit strains controlling the durability behaviour and also cater for uncertainty in the fatigue design to sections 8 and 9,
- to cover ULS verifications for linear and sublinear (non linear) structural response to actions.

(2) Stress limits may be related to the breaking strength

$$\sigma_{uk} = \frac{F_{uk}}{A_m} \qquad (7.1)$$

see equation (6.3).

NOTE 1 The National Annex may give values for stress limits f_{const} and f_{SLS}. Recommended values for stress limits f_{const} are given in Table 7.1 for the construction phase and for stress limits f_{SLS} in Table 7.2 for service conditions.

Table 7.1: Stress limits f_{const} for the construction phase

Conditions for erection using strand by strand installation	$f_{const.}$
First strand for only a few hours	0,60 σ_{uk}
After instalment of other strands	0,55 σ_{uk}

NOTE 2 The stress limits follow from

$$f_{const} = \frac{\sigma_{uk}}{1{,}50\gamma_R \gamma_F} = \frac{0{,}66\,\sigma_{uk}}{\gamma_R \gamma_F} \qquad (7.2)$$

with $\gamma_R \times \gamma_F = 1{,}0 \times 1{,}10 = 1{,}10$ for short term situations

$\gamma_R \times \gamma_F = 1{,}0 \times 1{,}20 = 1{,}20$ for long term situations

Table 7.2: Stress limits for service conditions

Model uncertainty for fatigue	f_{SLS}
Fatigue design including bending stresses *)	0,50 σ_{uk}
Fatigue design without bending stresses	0,45 σ_{uk}
*) Bending stresses may be reduced by detailing measures, see 7.1(4).	

NOTE 3 The stress limits follow from

$$f_{SLS} = \frac{\sigma_{uk}}{1{,}50\,\gamma_R \gamma_F} = \frac{0{,}66\,\sigma_{uk}}{\gamma_R \gamma_F} \qquad (7.3)$$

with $\gamma_R \times \gamma_F = 0{,}9 \times 1{,}48 = 1{,}33$ with consideration of bending stresses

$\gamma_R \times \gamma_F = 1{,}0 \times 1{,}48 = 1{,}48$ without consideration of bending stresses

where $\gamma_F \approx \gamma_Q = 1{,}50 \approx 1{,}48$

NOTE 4 The stress limit $f_{SLS} = 0{,}45\ \sigma_{uk}$ is used for testing, see Annex A.

8 Vibrations of cables

8.1 General

(1) For cables exposed to climatic conditions (e.g. for stay cables) the possibility of wind-induced vibrations during and after erection and their significance on the safety should be checked.

(2) Dynamic wind forces acting on the cable may be caused by

a) buffeting (from turbulence in the on-coming air flow)

b) vortex shedding (from von Karman vortexes in the wake behind the cable)

c) galloping (self induction)

d) wake galloping (fluid-elastic interaction of neighbouring cables)

e) interaction of wind, rain and cable

NOTE Gallopping is not possible on a cable with a circular cross section for symmetry reasons. This phenomenon may arise on cables with shapes altered, due to ice, dust, helical shapes of cable etc.

Forces due to c), d) and e) are a function of the motion of the cable (feedback) and due to ensuing aeroelastic instability lead to vibrations of large amplitudes starting at a critical wind speed. As the mechanism of dynamic excitation is not yet sufficiently modelled to make reliable predictions measures should be provided to limit unforeseen vibrations.

(3) Cable vibrations may also be caused by dynamic forces acting on other parts of the structure (girder, pylon).

NOTE This phenomenon is often referred to as "parametric excitation" and is responsible for vibrations of large amplitudes in case of overlapping between stay eigenfrequencies and structure eigenfrequencies.

8.2 Measures to limit vibrations of cables

(1) Cable structures should be monitored for excessive wind induced vibrations either by visual inspection or other methods that allow a more accurate determination of the involved amplitudes, modes and frequencies and the accompanying wind and rain characteristics.

(2) Provisions should be made in the design of a cable structure to enable implementation of vibration-suppressing measures during or after erection if unforeseen vibrations occur.

(3) Such measures are:

a) modification of cable surface (aerodynamic contour)

b) additional damping (e.g. by damping devices)

c) stabilizing cables (e.g. by tie-down cables with appropriate connections)

8.3 Estimation of risks

NOTE The complexity of the physical phenomena involved means it is not always possible to assess the risk of cable stay vibration. Conversely, economic constraints prohibit specifying "unnecessary" preventive measures. The following rules are guides intended to help to reach a trade-off.

(1) Rain-wind instability must systematically be prevented by design precautions; this involves cable stays with texturing.

(2) The risk of vibration increases with cable stay length. Short cable stays (less than about 70 – 80 m) generally involve no risk, other than of parametric resonance in the case of a particularly unstable structure (poorly shaped and flexible deck). There is therefore generally no need to make provisions for dampers on short cable stays.

(3) For long cable stays (more than 80 m), it is recommended that dampers be installed to obtain a damping ratio to critical greater that 0,5 %. It might be possible to dispense with dampers on the backspan cable stays if the spans are so short that there is likely no major displacement of anchorages.

(4) The risk of parametric resonance should be assessed at the design stage by means of a detailed study of the eigenmodes of the structure and cable stays, involving the ratio of angular frequencies and anchorage displacement for each mode.

(5) Everything should be done to avoid overlapping of frequencis, i.e. situations where the cable stay´s frequency of excitation Ω is close to (within 20 % of) the structure´s frequency ω_n or $2\omega_n$. If necessary, stability cables can be used to offset the modal angular frequencies of the cable stays.

(6) To ensure that users feel safe, the amplitude of cable stay vibration should be limited using a response criterion. E.g. with a moderate wind velocity of 15 m/s the amplitude of cable stay vibration shall not exceed L/500, where L is the cord length.

9 Fatigue

9.1 General

(1) The fatigue endurance of tension components according to classes 3, 4 or 5 to Table 2.1 shall be determined using the fatigue actions from EN 1991 and the appropriate category of structural detail.

(2) Fatigue failure of cable systems usually occurs at, or is governed by the effects at anchorages, saddles or clamps. The effective category should preferably be determined from tests representing the actual configuration used and reproducing any flexural effect or transverse stresses likely to occur in practice. The test evaluation should be carried out according to EN 1990 – Annex D.

9.2 Fluctuating axial loads

(1) In the absence of the tests described in 9.1(2) above, fatigue strength curves according to Figure 9.1 may be used and the fatigue category of detail be taken as given in Table 9.1.

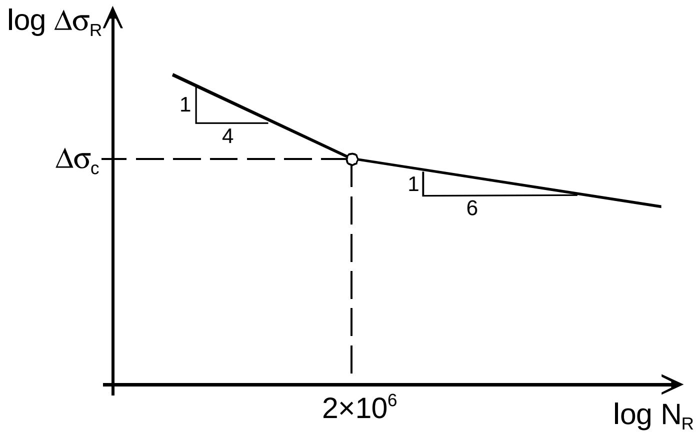


Figure 9.1: Fatigue strength curves for tension components

Table 9.1: Detail categories for fatigue strength according to the standard fatigue strength curves in EN 1993-1-9

Group		Tension element	Detail category $\Delta\sigma_c$ [N/mm²]
A	1	Prestressing bars	105
B	2	Fully locked coil rope with metal or resin socketing	150
	3	Spiral strands with metal or resin socketing	150
C	4	Parallel wire strands with epoxy socketing	160
	5	Bundle of parallel strands	160
	6	Bundle of parallel	160

NOTE The fatigue categories in Table 9.1 refer to exposure classes 3 and 4 according to Table 2.1 and to mainly axial fatigue action. For axial and lateral fatigue actions (exposure class 5 according to Table 2.1) additional constructive measures are required in order to minimise bending stresses in the anchorage zone.

(2) The categories given in (1) are not valid unless the following conditions apply:

a) cables with sockets comply with the basic requirements in Annex A

b) the design of cables, saddles and clamps complies with 6

c) serious aerodynamic oscillations of cables are prevented, see 8

d) adequate protection against corrosion is provided, see 4.

(3) For fatigue assessments see EN 1993-1-9.

Annex A [informative] – Product requirements for tension components

A.1 Scope

(1) This Annex gives the product requirements for tension components and their terminations to be used for buildings and civil engineering works.

(2) The requirements depend on the particular use of the prefabricated tension component (environmental and loading condition).

(3) The following types of prefabricated tension components are included

- Group A: tension rod systems, bars
- Group C: bundles of parallel wires, bundles of bars, bundles of parallel strands

A.2 Basic requirements

(1) Tension components should comply with the following basic points to be considered:

1. strength and ductility of the cable system and its terminations including durability,
2. fatigue resistance to axial load fluctuation plus bending stresses and angular deviations caused by catenary effects, wind forces and erection imperfections,
3. stable condition of axial and flexural stiffness of the cable system,
4. resistance to any corrosion action including environmental effects on corrosion barriers in the cable system and in particular in the region of anchorages,
5. resistance to fretting at any contact between steel parts.

(2) Terminations and anchorages of the tension components shall be designed such that

1. the ultimate resistance of the tension component would be reached before any gross yielding or other permanent deformation of the anchoring or any bearing elements would occur,
2. their fatigue resistance exceeds that of the components,
3. facilities are available for providing adequate adjustment of the component length to meet the requirements for preload, geometrical tolerances etc.,
4. sufficient articulation is provided in the anchorage to cater for manufacturing and erection imperfection,
5. the tension components are replaceable.

(3) These requirements shall be met by

- appropriate choice of materials as wires, strands, steels, protective materials,
- adequate make up and form of construction in view of strength, stiffness, ductility and durability as well as robustness for manufacturing, transport, handling and installation,
- quality control of termination fitting to ensure accurate alignment of cable.

(4) The fulfilment of the requirements shall be verified by initial tests for the system and test during the quality management.

A.3 Materials

(1) All materials used should comply with the relevant European technical specifications.

(2) The suitability of the corrosion protection system including the durability of filler and protection materials should be proved by appropriate testing.

NOTE The testing may prove the following basic functions:

- protection against aggressive agents (chemicals, environmental stress cracking, UV, mechanical impacts)
- watertightness (flexibilty and durability when cable bends)
- durability of colour (if required)

A.4 Requirements for tests

A.4.1 General

(1) The following tests on wire, strands, bars and complete cables shall ensure that they perform as required.

(2) $F_{0,1ke}$ and $F_{uk,e}$ (see 6.2) should be determined in a static tension tests. If necessary for cutting to length (see 3.4) and structural analysis (see 5) the test should follow the expected stress history of the cable in the structure for measuring all relevant data.

(3) To determine the fatigue strength curve (if necessary) a sufficient number of axial tests should be done at $\sigma_{sup} = 0,45\sigma_{uk}$ (see 7.2(2)) with different values of ΔF (force controlled, not $\Delta\ell$), see Table A.4.1.

Table A.4.1: Severity classes for fatigue load

	Type of test	Fatigue loading before fracture test
1	axial test (class 3 and 4)	σ_{sup} = $0,45\sigma_{uk}$ $\Delta\sigma$ according to $\Delta\sigma_c$ given in Table 9.1 $\Delta\alpha$ = 0 n = 2×10^6 cycles
2	axial and flexural test (class 5)	σ_{sup} = $0,45\sigma_{uk}$ $\Delta\sigma$ according to $\Delta\sigma_c$ given in Table 9.1 $\Delta\alpha$ = 0 – 10 milliradians (0 – 0,7 degrees) n = 2×10^6 cycles

(4) If the tension component is used for a structure under fatigue loading and the fatigue resistance is verified according to 9.2(2) at least one test with each diameter should be carried out. It should be checked that in an axial test with $\sigma_{sup} = 0,45\ \sigma_{uk}$ and $\Delta\sigma = 1,25\ \Delta\sigma_c$ (see Table 9.1) after $2\cdot10^6$ cycles the number of broken wires is < 2% of all wires. No failure shall occur in the anchorage material or in any component of the anchorage during the fatigue tests. No failure is acceptable for bars.

(5) If the round out radius at the entrance of the cable in the socket is less than 30d the tests (2) and (3) have to be done as axial and flexural tests with the expected angle $\Delta\alpha$.

(6) After fatigue loading, the test specimen shall be reloaded and shall develop a minimum tensile force equal to 92% of the actual tensile strength of the cable or 95% of the minimum uultimate tensile strength of the cable, whichever is greater. The strain at resistance must be ≥ 1,5%.

(7) Fatigue tests in accordance with EN 10138 should be performed with single strands, wires or bars on samples taken from each manufactured length of prestressing steel.

A.4.2 Main tension elements

A.4.2.1 Wires

(1) Wires after zinc coating if applicable should be tested in an approved testing machine.

A.4.2.2 Strands

(1) Tests should be carried out for tensile strength, 0,1% proof force and elongation according to EN 10138.

(2) Deflective tensile strength: the reduction of tensile strength should be less than 20%.

A.4.2.3 Bars

(1) Tests should be carried out for tensile strength, 0,1% proof force and elongation according to EN 10138.

A.4.3 Strands and complete cables

(1) If different sizes of one type of strand / rope are used at least 3 representative tests are required. Cables shall be tested with all load-bearing appurtenances and the test load be applied in the same way as in the structure.

A.4.4 Coefficient of friction

(1) If the coefficient of friction between strands and surfaces of saddles, clamps etc. is determined by testing

- the effects of axial loads on the diameter of the strands,
- the creeping effects from transverse preloading (on filler material and zinc coating including possible ovalisation)

shall be taken into account.

(2) In the evaluation of the test results due account shall be taken of the fact, that friction can be beneficial or adverse to an effect being considered.

A.4.5 Corrosion protection

A.4.5.1 Waterproofing

(1) To prove the durability of the cable system a test set up with “accelerated ageing” for a complete sample of the lower end of the cable with all anchoring devices stay pipe etc. should be established in which cycles of axial loads and bending and temperature cycles can be simulated.

NOTE For test details see National Annex.

A.4.5.2 Corrosion protection barriers

NOTE For test details, e.g. salt fog tests, see National Annex.

Annex B [informative] – Transport, storage, handling

(1) Spiral strands and full locked coil cables are supplied in either coils or on reels.

(2) The minimum reeling diameter should not be below 30 times the rope diameter of full locked coil ropes, 24 times the rope diameter of spiral strand ropes and 16 times the diameter of stranded ropes to prevent possible tripping of the wire.

> **NOTE** The minimum diameter depends on the protection system, storage time and temperature. Caution for unreeling at temperatures below 5 °C.

(3) If cables are stored in coils each coil should be properly ventilated (no direct ground contact) to prevent any formation of white blister which may be caused by condensation water.

(4) Cables must be handled with utmost care when being installed. Coils require a turn-table for horizontal dereeling.

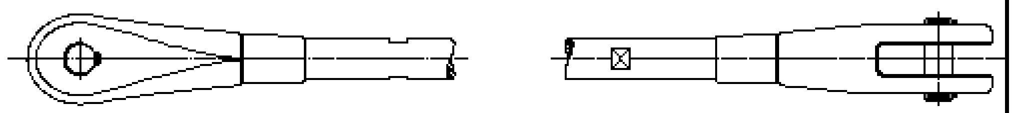
(5) The following general rules shall be observed:

- remove serving not before cable has been installed,
- have a bending radius not smaller than 30 × cable diameter,
- do not bend cables, do not pull across sharp edges,
- neither twist or untwist cables (observe cable marking line).

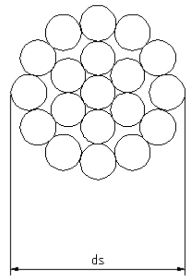
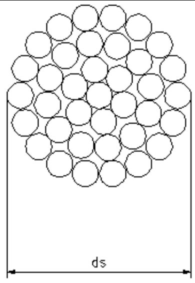
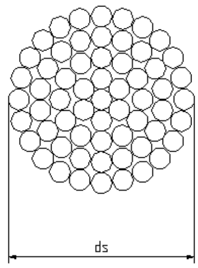
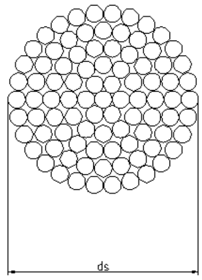
Annex C [informative] – Glossary

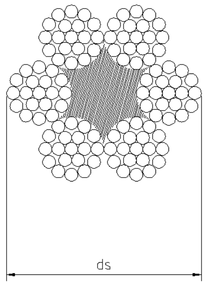
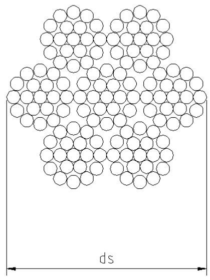
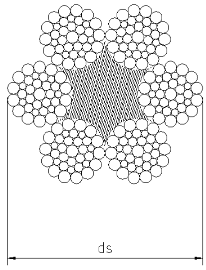
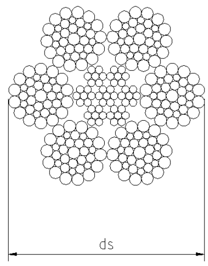
NOTE See EN 12385, Part 2.

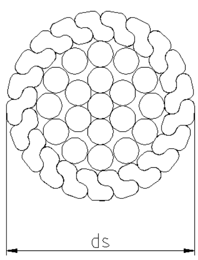
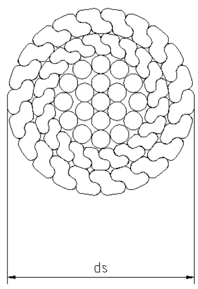
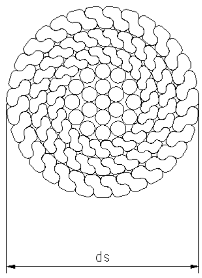
C.1 Products Group A

Tension rod system


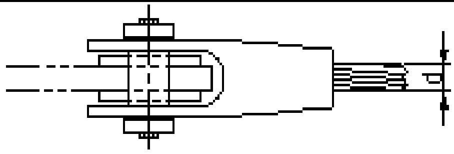
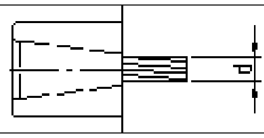
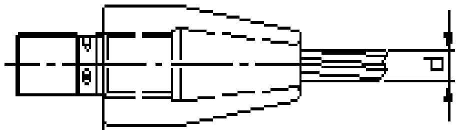
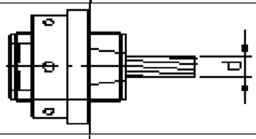
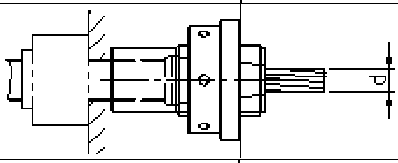
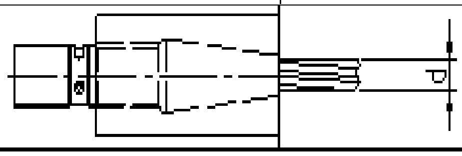
C.2 Products Group B

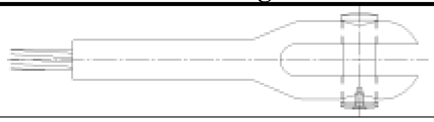
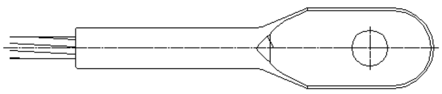
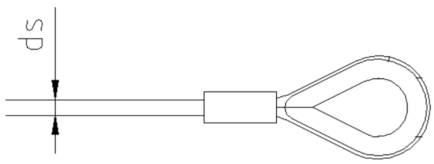
Spiral strand rope				
				
Construction	1 × 19	1 × 37	1 × 61	1 × 91
Diameter d_s [mm]	3 to 14	6 to 36	20 to 40	30 to 52
Strand	1	1	1	1
Wire per strand	19	37	61	91
Outer wire per strand	12	18	24	30
Nominal metallic area factor C	0,6	0,59	0,58	0,58
Breaking force factor K	0,525	0,52	0,51	0,51

Strand rope				
				
Construction	6 × 19 - CF	6 × 19 - CWS	6 × 36WS - CF	6 × 36 WS- CWR
Diameter d_s [mm]	6 to 40	6 to 40	6 to 40	6 to 40
Strand	6	6	6	6
Wire per strand	18	18	36	36
Outer wire per strand	12	12	14	14
Nominal metallic area factor C	0,357	0,414	0,393	0,455
Breaking force factor K	0,307	0,332	0,329	0,355

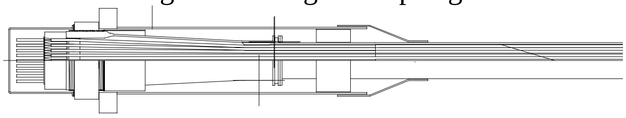
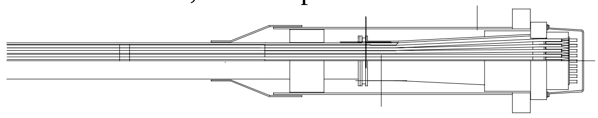
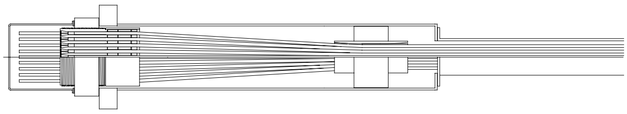
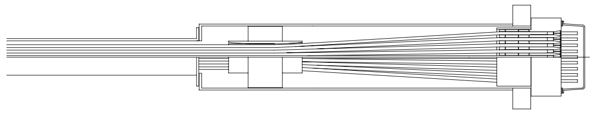
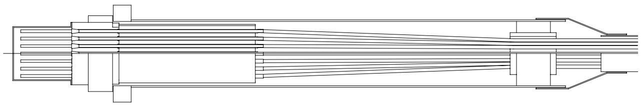
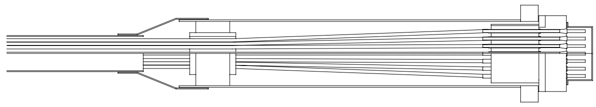
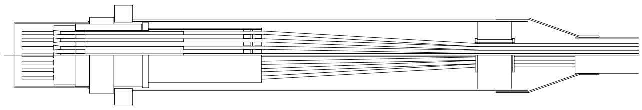
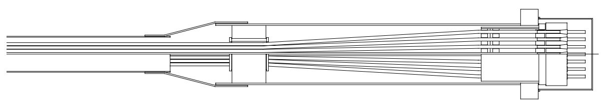
Full locked coil rope			
			
Construction	1 layer Z-wires	2 layer Z-wires	≥ 3 layer Z-wires
Diameter d_s [mm]	20 to 40	25 to 50	40 to 180
Tolerance d	+5%	+5%	+5%
Nominal metallic area factor C	0,636	0,660	0,700
breaking force factor K	0,585	0,607	0,643
NOTE Nominal metallic area factor and breaking force factor acc. EN 12385-2			

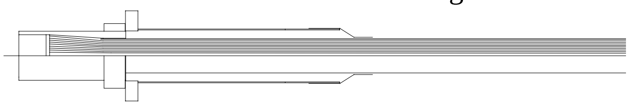
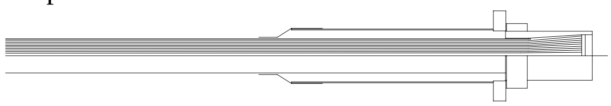
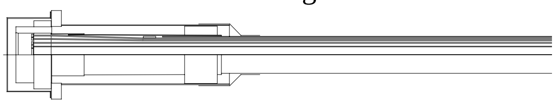
C.3 Wire rope end connectors

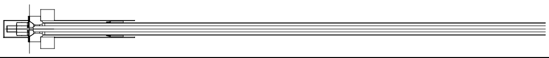
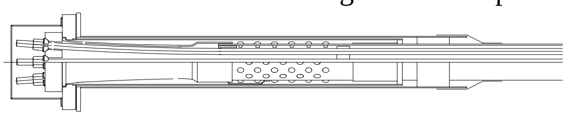
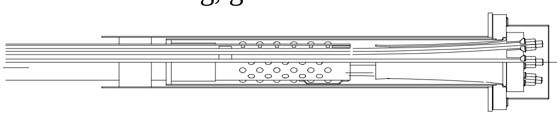
Wire rope end connectors - Metal or resin socketing acc. EN 13411-4	
Open spelter socket	
Cylindrical socket	
Conical socket with internal thread and tension rod	
Cylindrical socket with external thread and nut	
Cylindrical socket with internal and external thread and nut	
Cylindrical socket with internal thread and tension rod	

Wire rope end connectors swaged	
Open swaged socket	
Closed swaged socket	
Swaged fitting with thread	
Thimble with swaged aluminum ferrule acc. EN 13411-3	
U-bolt grip acc. EN 13411-5	*(to be added later ...)*

C.4 Product Group C

Bare strands, PE- or epoxy-coated strands	
Live end anchorage	Live end anchorage
Anchorage with wedges and postgrouted bond socket – bare strands, PE- or epoxi-coated strands	
	
Anchorage with wedges and sealing plates – PE-coated strands	
	
Anchorage with wedges and pregrouted pipe – PE-coated strands	
	
Anchorage with wedges and wax filled transition pipe – PE-coated strands	
	

Wires	
Live end anchorage	Live end anchorage
Anchorage with wires and compound filled socket	
	
Anchorage with wires and button heads filled with epoxy resin	
	

Bars	
Live end anchorage	Live end anchorage
Anchorage with single bar	
	
Anchorage with multiple bars and steel sheathing, grouted	
	

EUROPEAN PRESTANDARD
PRÉNORME EUROPÉENNE
EUROPÄISCHE VORNORM

DRAFT prEN 1993-1-2

February 2002

UDC

Descriptors:

English version

Eurocode 3 : Design of steel structures

Part 1.2 : General rules

Structural fire design

Calcul des structures en acier	Bemessung und Konstruktion von Stahlbauten
Partie 1.2 : Règles générales Calcul du comportement au feu	Teil 1.2 : Allgemeine Regeln Tragwerksbemessung für den Brandfall

Stage 34

CEN

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

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

Ref. No. EN 1993-1.2 : 20xx. E

Content

Foreword

This European Standard EN 1993-1-2, Design of steel structures – General rules – Structural fire design, has been prepared on behalf of Technical Committee CEN/TC250 « Structural Eurocodes », the Secretariat of which is held by BSI. CEN/TC250 is responsible for all Structural Eurocodes.

The text of the draft standard was submitted to the formal vote and was approved by CEN as EN 1993-1-2 on YYYY-MM-DD.

No existing European Standard is superseded.

Background of the Eurocode programme

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

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

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

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

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

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

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

Status and field of application of Eurocodes

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

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

- as a means to prove compliance of building and civil engineering works with the essential requirements of Council Directive 89/106/EEC, particularly Essential Requirement No.1 – Mechanical resistance and stability, and Essential Requirement No 2 – Safety in case of fire

- as a basis for specifying contracts for the execution of construction works and related engineering services

- as a framework for drawing up harmonised technical specifications for construction products (En's and ETA's)

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

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

National standards implementing Eurocodes

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

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

– values and/or classes where alternatives are given in the Eurocode,
– values to be used where a symbol only is given in the Eurocode,
– country specific data (geographical, climatic, etc.), e.g. snow map,
– the procedure to be used where alternative procedures are given in the Eurocode,

it may also contain:

– decisions on the application of informative annexes, and
– references to non-contradictory complementary information to assist the user to apply the Eurocode.

Links between Eurocodes and harmonised technical specifications (EN's and ETA's) for products

There is a need for consistency between the harmonised technical specifications for construction products and the technical rules for works[4]. Furthermore, all the information accompanying the CE Marking of the

---

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

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

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

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

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

[5] see clause 2.2, 3.2(4) and 4.2.3.3

construction products which refer to Eurocodes shall clearly mention which Nationally Determined Parameters have been taken into account.

Additional information specific to EN 1993-1-2

EN 1993-1-2 describes the Principles, requirements and rules for the structural design of buildings exposed to fire, including the following aspects.

Safety requirements

EN 1993-1-2 is intended for clients (e.g. for the formulation of their specific requirements), designers, contractors and relevant authorities.

The general objectives of fire protection are to limit risks with respect to the individual and society, neighbouring property, and where required, environment or directly exposed property, in the case of fire.

Construction Products Directive 89/106/EEC gives the following essential requirement for the limitation of fire risks:

"The construction works must be designed and build in such a way, that in the event of an outbreak of fire

- the load bearing resistance of the construction can be assumed for a specified period of time
- the generation and spread of fire and smoke within the works are limited
- the spread of fire to neighbouring construction works is limited
- the occupants can leave the works or can be rescued by other means
- the safety of rescue teams is taken into consideration".

According to the Interpretative Document N° 2 "Safety in case of fire[5]" the essential requirement may be observed by following various possibilities for fire safety strategies prevailing in the Member States like conventional fire scenarios (nominal fires) or "natural" (parametric) fire scenarios, including passive and/or active fire protection measures.

The fire parts of Structural Eurocodes deal with specific aspects of passive fire protection in terms of designing structures and parts thereof for adequate load bearing resistance and for limiting fire spread as relevant.

Required functions and levels of performance can be specified either in terms of nominal (standard) fire resistance rating, generally given in national fire regulations or by referring to fire safety engineering for assessing passive and active measures.

Supplementary requirements concerning, for example

- the possible installation and maintenance of sprinkler systems,
- conditions on occupancy of building or fire compartment,
- the use of approved insulation and coating materials, including their maintenance,

are not given in this document, because they are subject to specification by the competent authority.

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

Design procedures

A full analytical procedure for structural fire design would take into account the behaviour of the structural system at elevated temperatures, the potential heat exposure and the beneficial effects of active and passive fire protection systems, together with the uncertainties associated with these three features and the importance of the structure (consequences of failure).

At the present time it is possible to undertake a procedure for determining adequate performance which incorporates some, if not all, of these parameters and to demonstrate that the structure, or its components, will give adequate performance in a real building fire. However, where the procedure is based on a nominal (standard) fire the classification system, which call for specific periods of fire resistance, takes into account (though not explicitly), the features and uncertainties described above.

Application of this Part 1-2 is illustrated in Figure 1. The prescriptive approach and the performance-based approach are identified. The prescriptive approach uses nominal fires to generate thermal actions. The performance-based approach, using fire safety engineering, refers to thermal actions based on physical and chemical parameters.

For design according to this part, EN 1991-1-2 is required for the determination of thermal and mechanical actions to the structure.

Design aids

Where simple calculation models are not available, the Eurocode fire parts give design solutions in terms of tabulated data (based on tests or advanced calculation models), which may be used within the specified limits of validity.

It is expected, that design aids based on the calculation models given in EN 1993-1-2, will be prepared by interested external organizations.

The main text of EN 1993-1-2 together with normative Annexes includes most of the principal concepts and rules necessary for structural fire design of steel structures.

National Annex for EN 1993-1-2

This standard gives alternative procedures, values and recommendations for classes with notes indicating where national choices may have to be made. Therefore the National Standard implementing EN 1993-1-2 should have a National annex containing the Eurocode all Nationally Determined Parameters to be used for the design of buildings [and civil engineering works] to be constructed in the relevant country.

National choice is allowed in EN 1993-1-2 through clauses:

[list of clauses to be prepared]
2.3 (1)
2.3 (2)
2.4.2 (3)
4.2.3.6 (1)
4.2.4 (5)
4.3

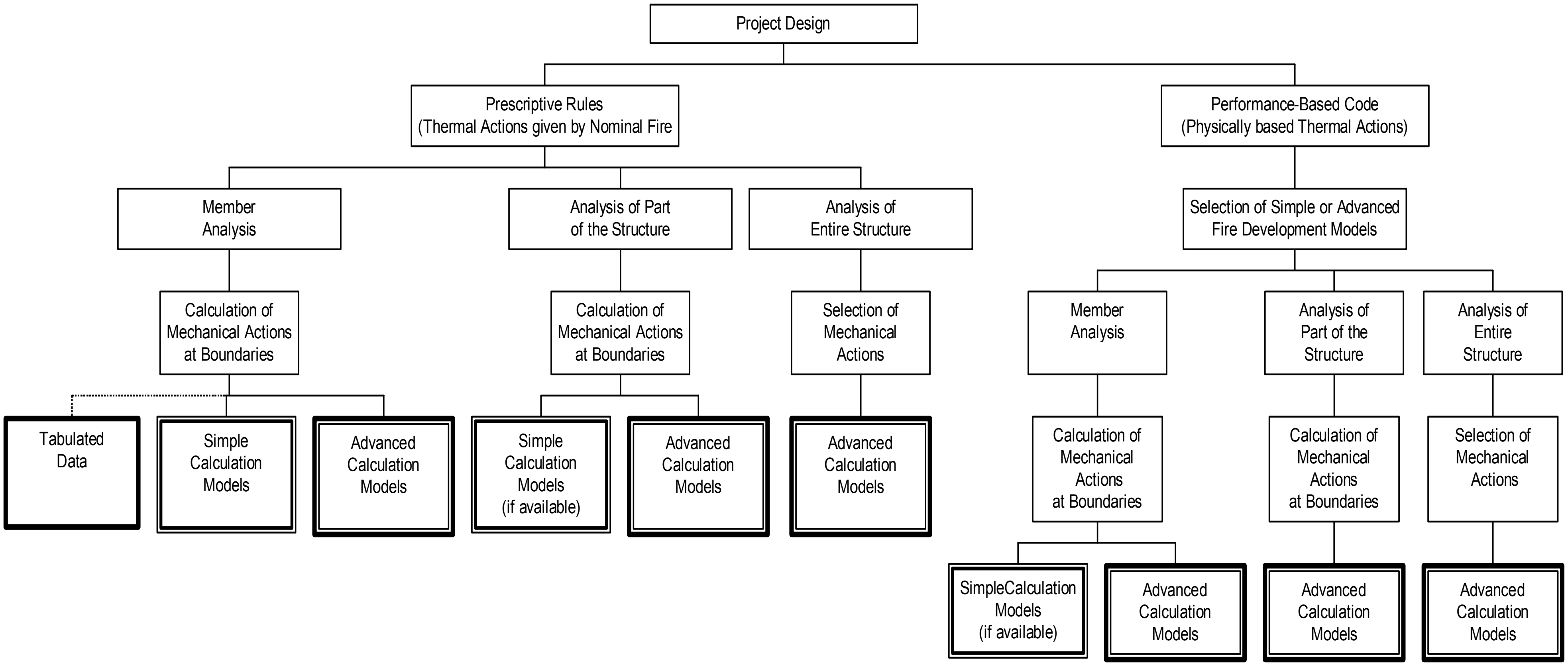


Figure 0.1: Design procedure

1. General

1.1 Scope

1.1.1 Scope of Eurocode 3

(1)P Eurocode 3 applies to the design of buildings and civil engineering works in steel. It complies with the principles and requirements for the safety and serviceability of structures, the basis of their design and verification that are given in EN 1990 – Basis of structural design.

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

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

- EN 1990 "Basis of structural design"
- EN 1991 "Actions on structures"
- hEN´s for construction products relevant for steel structures
- EN xxx[5] "Execution of steel structures"
- EN 1998 "Design of structures for earthquake resistance", when steel structures are built in seismic regions

(4)P Eurocode 3 is subdivided in various parts:

- EN 1993-1 Design of Steel Structures : Generic rules.
- EN 1993-2 Design of Steel Structures : Steel bridges.
- EN 1993-3 Design of Steel Structures : Buildings.
- EN 1993-4 Design of Steel Structures : Silos, tanks and pipelines.
- EN 1993-5 Design of Steel Structures : Piling.
- EN 1993-6 Design of Steel Structures : Crane supporting structures.
- EN 1993-7 Design of Steel Structures : Towers, masts and chimneys.

1.1.2 Scope of Part 1.2 of Eurocode 3

(1) This Part 1-2 of EN 1993 deals with the design of steel structures for the accidental situation of fire exposure and is intended to be used in conjunction with EN 1993-1 and EN 1991-1-2. This part 1.2 only identifies differences from, or supplements to, normal temperature design.

(2) This Part 1-2 of EN 1993 deals only with passive methods of fire protection. Active methods are not covered.

(3) This Part 1-2 of EN 1993 applies to steel structures that are required to fulfil load bearing function when exposed to fire, in terms of avoiding premature collapse of the structure.

NOTE: This part does not include rules for separating elements.

[5] ENxxx is the conversion of EN1090

(4) This Part 1-2 of EN 1993 gives principles and application rules (see EN 1991-1-2) for designing structures for specified requirements in respect of the aforementioned functions and the levels of performance.

(5) This Part 1-2 of EN 1993 applies to structures, or parts of structures, that are within the scope of EN 1993-1 and are designed accordingly.

(6) The methods given in this Part 1-2 of EN 1993 are applicable to structural steel grades S235, S275 and S355 of EN 10025 and to all steel grades of EN 10113, EN 10155, EN 10210-1 and EN 10219-1.

(7) The methods given in this Part 1-2 of EN 1993 are also applicable to cold-formed thin gauge steel members and sheeting within the scope of EN 1993-1-3.

(8) The methods given in this Part 1-2 of EN 1993 are applicable to any steel grade for which material properties at elevated temperatures are available, based on harmonised European standards.

(9) The methods given in this Part 1-2 are also applicable stainless steel members and sheeting within the scope of EN 1993-1-4.

NOTE: For the fire resistance of composite steel and concrete structures, see EN 1994-1-2.

1.2 Normative references

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

EN 10025 Hot rolled products of non-alloy structural steels: Technical delivery conditions;

EN 10113 Hot rolled products in weldable fine grade structural steels:

Part 1: *General delivery conditions*;

Part 2: *Delivery conditions for normalized/normalized rolled steels*;

Part 3: *Delivery conditions for thermo-mechanically rolled steels*;

EN 10155 Structural steels with improved atmospheric corrosion resistance - Technical delivery conditions;

EN 10210 Hot finished structural hollow sections of non-alloy and fine grain structural steels:

Part 1: *Technical delivery conditions*;

EN 10219 Cold formed welded structural hollow sections of non-alloy and fine grain structural steels:

Part 1: *Technical delivery conditions*;

EN ISO 1363 Fire resistance: General requirements;

EN ISO 13501 Fire classification of construction products and building elements

Part 2 Classification using data from fire resistance tests

ENV 13381 Fire tests on elements of building construction:

Part 1: *Test method for determining the contribution to the fire resistance of structural members: by horizontal protective membranes*;

Part 2 *Test method for determining the contribution to the fire resistance of structural members: by vertical protective membranes*;

Part 4: *Test method for determining the contribution to the fire resistance of structural members: by applied protection to steel structural elements*;

EN 1990 Eurocode: Basis of structural design

EN 1991 Eurocode 1. Basis of design and actions on structures:
Part 1-2: *Actions on structures exposed to fire*;
EN 1993 Eurocode 3. Design of steel structures:
Part 1-1: *General rules : General rules and rules for buildings*;
Part 1-3: *General rules : Supplementary rules for cold formed thin gauge steel members and sheeting*;
Part 1-4: *General rules : Supplementary rules for stainless steels*
EN 1994 Eurocode 4. Design of composite steel and concrete structures:
Part 1-2: *General rules : Structural fire design*;
ISO 1000 SI units.

1.3 Assumptions

(1)P In addition to the general assumptions of EN 1990 the following assumption applies:
- Any passive fire protection systems taken into account in the design will be adequately maintained.

1.4 Distinction between principles and application rules

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

1.5 Definitions

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

(2)P The following terms are used in Part 1.2 of Eurocode 1993 with the following meanings:

1.5.1 Special terms relating to design in general

1.5.1.1 Part of structure
Isolated part of an entire structure with appropriate support and boundary conditions.

1.5.1.2 Protected members
Members for which measures are taken to reduce the temperature rise in the member due to fire.

1.5.2 Terms relating to thermal actions

1.5.2.1 Standard temperature-time curve
A nominal curve, defined in EN 13501-2 for representing a model of a fully developed fire in a compartment.

1.5.2.2 Temperature-time curves:
Gas temperature in the environment of member surfaces as a function of time. They may be:

- **nominal:** Conventional curves, adopted for classification or verification of fire resistance, e.g. the standard temperature-time curve, external fire curve, hydrocarbon fire curve;
- **parametric:** Determined on the basis of fire models and the specific physical parameters defining the conditions in the fire compartment.

1.5.3 Terms relating to material and products

1.5.3.1 Carbon steel
In this standard: steel grades referred to in Eurocode 3, except stainless steels

1.5.3.2 Fire protection material
Any material or combination of materials applied to a structural member for the purpose of increasing its fire resistance.

1.5.3.3 Stainless steel
All steels referred to in EN1993-1-4.

1.5.4 Terms relating to heat transfer analysis

1.5.4.1 Configuration factor
The configuration factor for radiative heat transfer from surface A to surface B is defined as the fraction of diffusely radiated energy leaving surface A that is incident on surface B.

1.5.4.2 Convective heat transfer coefficient
Convective heat flux to the member related to the difference between the bulk temperature of gas bordering the relevant surface of the member and the temperature of that surface.

1.5.4.3 Emissivity
Equal to absorptivity of a surface, i.e. the ratio between the radiative heat absorbed by a given surface, and that of a black body surface.

1.5.4.4 Net heat flux
Energy per unit time and surface area definitely absorbed by members.

1.5.4.5 Resulting emissivity
The ratio between the actual radiative heat flux to the member and the net heat flux that would occur if the member and its radiative environment were considered as black bodies.

1.5.4.6 Section factor
For a steel member, the ratio between the exposed surface area and the volume of steel; for an enclosed member, the ratio between the internal surface area of the exposed encasement and the volume of steel.

1.5.4.7 Box value of section factor
Ratio between the exposed surface area of a notional bounding box to the section and the volume of steel.

1.5.5 Terms relating to mechanical behaviour analysis

1.5.5.1 Critical temperature of steel structure
For a given load level, the temperature at which failure is expected to occur in a structural steel element for a uniform temperature distribution.

1.5.5.2 Effective yield strength
For a given temperature, the stress level at which the stress-strain relationship of steel is truncated to provide a yield plateau.

1.5.5.3 External member
Structural member located outside the building that can be exposed to fire through openings in the building enclosure.

1.5.5.4 Maximum stress level
For a given temperature, the stress level at which the stress-strain relationship of steel is truncated to provide a yield plateau.

1.6 Symbols

(1)P For the purpose of this Part 1.2 of EN1993, the following symbols apply:

Latin upper case letters

A_m	the surface area of a member per unit length;
A_p	the area of the inner surface of the fire protection material per unit length of the member;
E_a	the modulus of elasticity of steel for normal temperature design;
$E_{\mathrm{a},\theta}$	the slope of the linear elastic range for steel at elevated temperature θ_a ;
$E_{\mathrm{d,fi}}$	the design effect of actions in the fire situation;
V	the volume of a member per unit length;

Latin lower case letters

c	the specific heat;
d_p	the thickness of fire protection material;
$f_{\mathrm{p},\theta}$	the proportional limit for steel at elevated temperature θ_a ;
$f_{\mathrm{y},\theta}$	the effective yield strength of steel at elevated temperature θ_a ;
$\dot{h}_{\mathrm{net,d}}$	the design value of the net heat flux per unit area;
k_θ	the relative value of a strength or deformation property of steel at elevated temperature θ_a ;
l	the length at 20 °C ;
t	the time in fire exposure;

Greek upper case letters

Δt	the time interval;

Greek lower case letters

η_fi	the reduction factor for design load level in the fire situation;
θ	the temperature;
κ	the adaptation factor;
λ	the thermal conductivity;
μ_0	the degree of utilisation at time $t = 0$.

2 Basis of design

2.1 Requirements

2.1.1 Basic requirements

R (1)P Where mechanical resistance in the case of fire is required, steel structures shall be designed and constructed in such a way that they maintain their load bearing function during the relevant fire exposure.

R (2)P Deformation criteria shall be applied where the means of protection, or the design criteria for separating elements, require consideration of the deformation of the load bearing structure.

PE (3) Except from (2) consideration of the deformation of the load bearing structure is not necessary in the following cases, as relevant:
- the efficiency of the means of protection has been evaluated according to section 3.4.3;
- the separating elements have to fulfil requirements according to a nominal fire exposure.

2.1.2 Nominal fire exposure

(1)P For the standard fire exposure, members shall comply with criteria R as follows:
- load bearing only: mechanical resistance (criterion R).

(2) Criterion "R" is assumed to be satisfied where the load bearing function is maintained during the required time of fire exposure.

(3) With the external fire exposure curve the same criteria should apply, however the reference to this specific curve should be identified by the letters "ef".

(4) With the hydrocarbon fire exposure curve the same criteria should apply, however the reference to this specific curve should be identified by the letters "HC".

2.1.3 Parametric fire exposure

(1) The load-bearing function is ensured when collapse is prevented during the complete duration of the fire including the decay phase or during a required period of time.

2.2 Actions

R (1)P The thermal and mechanical actions shall be taken from EN 1991-1-2.

R (2) In addition to EN 1991-1-2, the emissivity related to the steel surface should be equal to 0,7 for carbon steel and equal to 0,4 for stainless steels according to annex C.

2.3 Design values of material properties

DF (1)P Design values of mechanical (strength and deformation) material properties $X_{d,fi}$ are defined as follows:

$$X_{d,fi} = k_\theta X_k / \gamma_{M,fi} \quad (2.1)$$

where:

X_k is the characteristic value of a strength or deformation property *(generally f_k or E_k)* for normal temperature design to EN 1993-1-1;

k_θ is the reduction factor for a strength or deformation property $(X_{k,\theta} / X_k)$, dependent on the material temperature, see section 3;

$\gamma_{M,fi}$ is the partial safety factor for the relevant material property, for the fire situation.

NOTE: For mechanical properties of steel, the partial safety factor for the fire situation see national annex. The use of $\gamma_{M,fi} = 1.0$ is recommended.

DF (2)P Design values of thermal material properties $X_{d,fi}$ are defined as follows:

- if an increase of the property is favourable for safety:
  $$X_{d,fi} = X_{k,\theta} / \gamma_{M,fi} \tag{2.2a}$$
- if an increase of the property is unfavourable for safety:
  $$X_{d,fi} = \gamma_{M,fi} X_{k,\theta} \tag{2.2b}$$

where:

$X_{k,\theta}$ is the value of a material property in fire design, generally dependent on the material temperature, see section 3;

$\gamma_{M,fi}$ is the partial safety factor for the relevant material property, for the fire situation.

NOTE: For thermal properties of steel, the partial safety factor for the fire situation see national annex. The use of $\gamma_{M,fi} = 1.0$ is recommended.

2.4 Verification methods

2.4.1 General

R (1)P The model of the structural system adopted for design to this Part 1-2 of EN1993 shall reflect the expected performance of the structure in fire.

NOTE: Where rules given in this Part 1-2 of EN1993 are valid only for the standard fire exposure, this is identified in the relevant clauses.

R (2)P It shall be verified that, during the relevant duration of fire exposure t:

$$E_{fi,d} \leq R_{fi,d,t} \tag{2.3}$$

where:

$E_{fi,d}$ is the design effect of actions for the fire situation, determined in accordance with EN 1991-1-2, including the effects of thermal expansions and deformations;

$R_{fi,d,t}$ is the corresponding design resistance in the fire situation.

R (3)P The structural analysis for the fire situation should be carried out according to EN 1990 5.1.4 (2).

NOTE 1: For member analysis, see 2.4.2;
For analysis of parts of the structure, see 2.4.3;
For global structural analysis, see 2.4.4.

NOTE 2: For verifying standard fire resistance requirements, a member analysis is sufficient.

PE (4) As an alternative to design by calculation, fire design may be based on the results of fire tests, or on fire tests in combination with calculations.

2.4.2 Member analysis

R

(1) The effect of actions should be determined for time t=0 using combination factors $\psi_{1,1}$ or $\psi_{2,1}$ according to EN 1991-1-2 clause 4.3.1.

PE

(2) As a simplification to (1), the effect of actions $E_{\mathrm{d,fi}}$ may be obtained from a structural analysis for normal temperature design as:

$$E_{\mathrm{d,fi}} = \eta_{\mathrm{fi}} E_{\mathrm{d}} \qquad (2.4)$$

where:

E_{d} is the design value of the corresponding force or moment for normal temperature design, for a fundamental combination of actions (see EN 1990);

η_{fi} is the reduction factor for the design load level for the fire situation.

R

(3) The reduction factor η_{fi} for load combination (6.10) in EN 1990 should be taken as:

$$\eta_{\mathrm{fi}} = \frac{G_{\mathrm{k}} + \psi_{fi} Q_{\mathrm{k,1}}}{\gamma_{\mathrm{G}} G_{\mathrm{k}} + \gamma_{\mathrm{Q,1}} Q_{\mathrm{k,1}}} \qquad (2.5)$$

or for load combination (6.10a) and (6.10b) in EN 1990 as the smaller value given by the two following expressions:

$$\eta_{\mathrm{fi}} = \frac{G_{\mathrm{k}} + \psi_{fi} Q_{\mathrm{k,1}}}{\gamma_{\mathrm{G}} G_{\mathrm{k}} + \gamma_{\mathrm{Q,1}} Q_{\mathrm{k,1}}} \qquad (2.5a)$$

$$\eta_{\mathrm{fi}} = \frac{G_{\mathrm{k}} + \psi_{fi} Q_{\mathrm{k,1}}}{\xi \gamma_{\mathrm{G}} G_{\mathrm{k}} + \gamma_{\mathrm{Q,1}} Q_{\mathrm{k,1}}} \qquad (2.5b)$$

where:

$Q_{\mathrm{k,1}}$ is the principal variable load;
G_{k} is the characteristic value of a permanent action;
γ_{G} is the partial factor for permanent actions;

$\gamma_{\mathrm{Q,1}}$ is the partial factor for variable action 1;
ψ_{fi} is the combination factor for frequent values, given either by $\psi_{1,1}$ or $\psi_{2,1}$,see EN1991-1-2;
ξ is a reduction factor for unfavourable permanent actions G.

NOTE 1: An example of the variation of the reduction factor η_{fi} versus the load ratio $Q_{\mathrm{k,1}}/G_{\mathrm{k}}$ for different values of the combination factor $\psi_{\mathrm{fi}} = \psi_{1,1}$ according to expression (2.5), is shown in figure 2.1 with the following assumptions: γ_{GA} = 1,0, γ_{G} = 1,35 and γ_{Q} = 1,5. Partial factors are specified in the relevant National annexes of EN 1990. Equations (2.5a) and (2.5b) give slightly higher values.

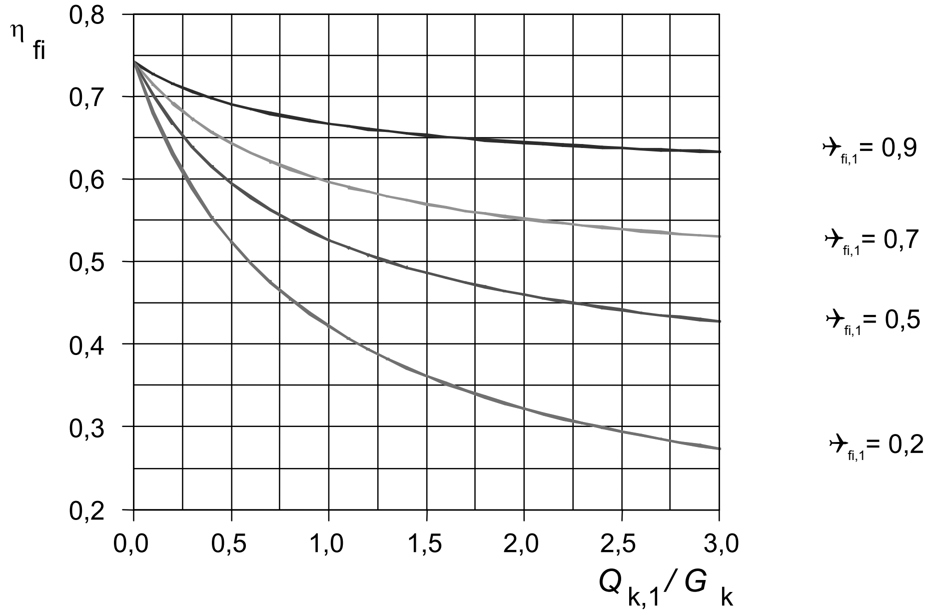


Figure 2.1: Variation of the reduction factor η_{fi} with the load ratio $Q_{k,1} / G_k$

NOTE 2: As a simplification the recommended value of η_{fi} = 0,65 may be used, except for imposed load according to load category E as given in EN 1991-1-1 (areas susceptible to accumulation of goods, including access areas) where the recommended value is 0,7.

PE (4) Only the effects of thermal deformations resulting from thermal gradients across the cross-section need to be considered. The effects of axial or in-plain thermal expansions may be neglected.

PE (5) The boundary conditions at supports and ends of member may be assumed to remain unchanged throughout the fire exposure.

PE (6) Simplified or advanced calculation methods given in clauses 4.2 and 4.3 respectively are suitable for verifying members under fire conditions.

2.4.3 Analysis of part of the structure

ST (1) 2.4.2 (1) applies

PE (2) As an alternative to carrying out a structural analysis for the fire situation at time $t = 0$, the reactions at supports and internal forces and moments at boundaries of part of the structure may be obtained from a structural analysis for normal temperature as given in clause 2.4.2.

R (3) The part of the structure to be analysed should be specified on the basis of the potential thermal expansions and deformations such, that their interaction with other parts of the structure can be approximated by time-independent support and boundary conditions during fire exposure.

R (4)P Within the part of the structure to be analyzed, the relevant failure mode in fire exposure, the temperature-dependent material properties and member stiffness, effects of thermal expansions and deformations (indirect fire actions) shall be taken into account

PE (5) The boundary conditions at supports and forces and moments at boundaries of part of the structure may be assumed to remain unchanged throughout the fire exposure.

2.4.4 Global structural analysis

R (1)P When a global structural analysis for the fire situation is carried out, the relevant failure mode in fire exposure, the temperature-dependent material properties and member stiffness , effects of thermal expansions and deformations (indirect fire actions) shall be taken into account.

3 Material properties

3.1 General

RC (1) Unless given as design values, the values of material properties given in this section should be treated as characteristic values.

RC (2)P The mechanical properties of steel at 20 °C shall be taken as those given in EN 1993-1-1 for normal temperature design.

3.2 Mechanical properties of carbon steels

3.2.1 Strength and deformation properties

RC (1) For heating rates between 2 and 50 K/min, the strength and deformation properties of steel at elevated temperatures should be obtained from the stress-strain relationship given in figure 3.1.

NOTE: For the rules of this standard it is assumed that the heating rates fall within the specified limits.

RC (2) The relationship given in figure 3.1 should be used to determine the resistances to tension, compression, moment or shear.

PE (3) Table 3.1 gives the reduction factors for the stress-strain relationship for steel at elevated temperatures given in figure 3.1. These reduction factors are defined as follows:

- effective yield strength, relative to yield strength at 20 °C: $k_{y,\theta} = f_{y,\theta}/f_y$
- proportional limit, relative to yield strength at 20 °C: $k_{p,\theta} = f_{p,\theta}/f_y$
- slope of linear elastic range, relative to slope at 20 °C: $k_{E,\theta} = E_{a,\theta}/E_a$

NOTE: The variation of these reduction factors with temperature is illustrated in figure 3.2.

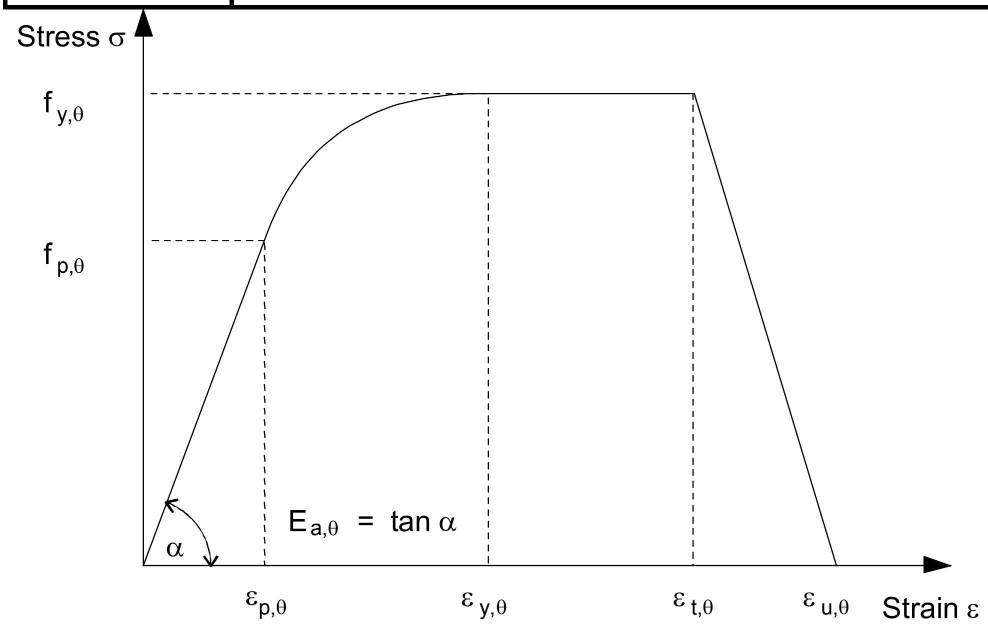
PE (4) Alternatively, for temperatures below 400 °C, the stress-strain relationship specified in (1) may be extended by the strain-hardening option given in annex A, provided local or overall buckling does not lead to premature collapse.

3.2.2 Unit mass

PE (1) The unit mass of steel ρ_a may be considered to be independent of the steel temperature. The following value may be taken:

$$\rho_a = 7850 \text{ kg/m}^3$$

Strain range	Stress σ	Tangent modulus
$\varepsilon \le \varepsilon_{p,\theta}$	$\varepsilon E_{a,\theta}$	$E_{a,\theta}$
$\varepsilon_{p,\theta} < \varepsilon < \varepsilon_{y,\theta}$	$f_{p,\theta} - c + (b/a)\left[a^2 - (\varepsilon_{y,\theta} - \varepsilon)^2\right]^{0,5}$	$\dfrac{b(\varepsilon_{y,\theta} - \varepsilon)}{a\left[a^2 - (\varepsilon_{y,\theta} - \varepsilon)^2\right]^{0,5}}$
$\varepsilon_{y,\theta} \le \varepsilon \le \varepsilon_{t,\theta}$	$f_{y,\theta}$	0
$\varepsilon_{t,\theta} < \varepsilon < \varepsilon_{u,\theta}$	$f_{y,\theta}\left[1 - (\varepsilon - \varepsilon_{t,\theta})/(\varepsilon_{u,\theta} - \varepsilon_{t,\theta})\right]$	-
$\varepsilon = \varepsilon_{u,\theta}$	0,00	-
Parameters	$\varepsilon_{p,\theta} = f_{p,\theta}/E_{a,\theta}$ $\quad \varepsilon_{y,\theta} = 0,02$ $\quad \varepsilon_{t,\theta} = 0,15$ $\quad \varepsilon_{u,\theta} = 0,20$	
Functions	$a^2 = (\varepsilon_{y,\theta} - \varepsilon_{p,\theta})(\varepsilon_{y,\theta} - \varepsilon_{p,\theta} + c/E_{a,\theta})$ $b^2 = c\,(\varepsilon_{y,\theta} - \varepsilon_{p,\theta})E_{a,\theta} + c^2$ $c = \dfrac{(f_{y,\theta} - f_{p,\theta})^2}{(\varepsilon_{y,\theta} - \varepsilon_{p,\theta})E_{a,\theta} - 2(f_{y,\theta} - f_{p,\theta})}$	



Key:

$f_{y,\theta}$	effective yield strength;
$f_{p,\theta}$	proportional limit;
$E_{a,\theta}$	slope of the linear elastic range;
$\varepsilon_{p,\theta}$	strain at the proportional limit;
$\varepsilon_{y,\theta}$	yield strain;
$\varepsilon_{t,\theta}$	limiting strain for yield strength;
$\varepsilon_{u,\theta}$	ultimate strain.

Figure 3.1: Stress-strain relationship for carbon steel at elevated temperatures.

Table 3.1: Reduction factors for stress-strain relationship of carbon steel at elevated temperatures

Steel Temperature θ_a	Reduction factors at temperature θ_a relative to the value of f_y or E_a at 20°C		
	Reduction factor (relative to f_y) for effective yield strength $k_{y,\theta} = f_{y,\theta}/f_y$	Reduction factor (relative to f_y) for proportional limit $k_{p,\theta} = f_{p,\theta}/f_y$	Reduction factor (relative to E_a) for the slope of the linear elastic range $k_{E,\theta} = E_{a,\theta}/E_a$
20°C	1,000	1,000	1,000
100°C	1,000	1,000	1,000
200°C	1,000	0,807	0,900
300°C	1,000	0,613	0,800
400°C	1,000	0,420	0,700
500°C	0,780	0,360	0,600
600°C	0,470	0,180	0,310
700°C	0,230	0,075	0,130
800°C	0,110	0,050	0,090
900°C	0,060	0,0375	0,0675
1000°C	0,040	0,0250	0,0450
1100°C	0,020	0,0125	0,0225
1200°C	0,000	0,0000	0,0000

NOTE: For intermediate values of the steel temperature, linear interpolation may be used.

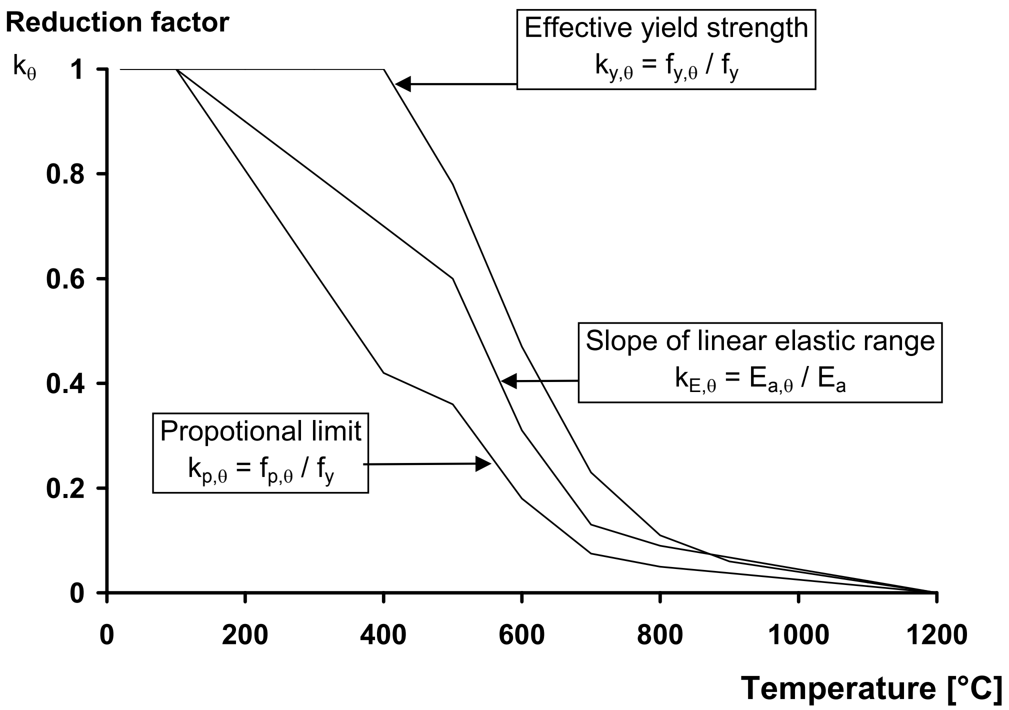


Figure 3.2: Reduction factors for the stress-strain relationship of carbon steel at elevated temperatures

3.3 Mechanical properties of stainless steels

PE (1) The mechanical properties of stainless steel may be taken from annex C.

3.4 Thermal properties

3.4.1 Carbon steels

3.4.1.1 Thermal elongation

RC (1) The thermal elongation of steel $\Delta l / l$ should be determined from the following:

- for $20°C \leq \theta_a < 750°C$:

$$\Delta l / l = 1{,}2 \times 10^{-5}\, \theta_a + 0{,}4 \times 10^{-8}\, \theta_a^2 - 2{,}416 \times 10^{-4} \qquad (3.1a)$$

- for $750°C \leq \theta_a \leq 860°C$:

$$\Delta l / l = 1{,}1 \times 10^{-2} \qquad (3.1b)$$

- for $860°C < \theta_a \leq 1200°C$:

$$\Delta l / l = 2 \times 10^{-5}\, \theta_a - 6{,}2 \times 10^{-3} \qquad (3.1c)$$

where:

l	is	the length at 20°C;
Δl	is	the temperature induced expansion;
θ_a	is	the steel temperature [°C].

NOTE: The variation of the thermal elongation with temperature is illustrated in figure 3.3.

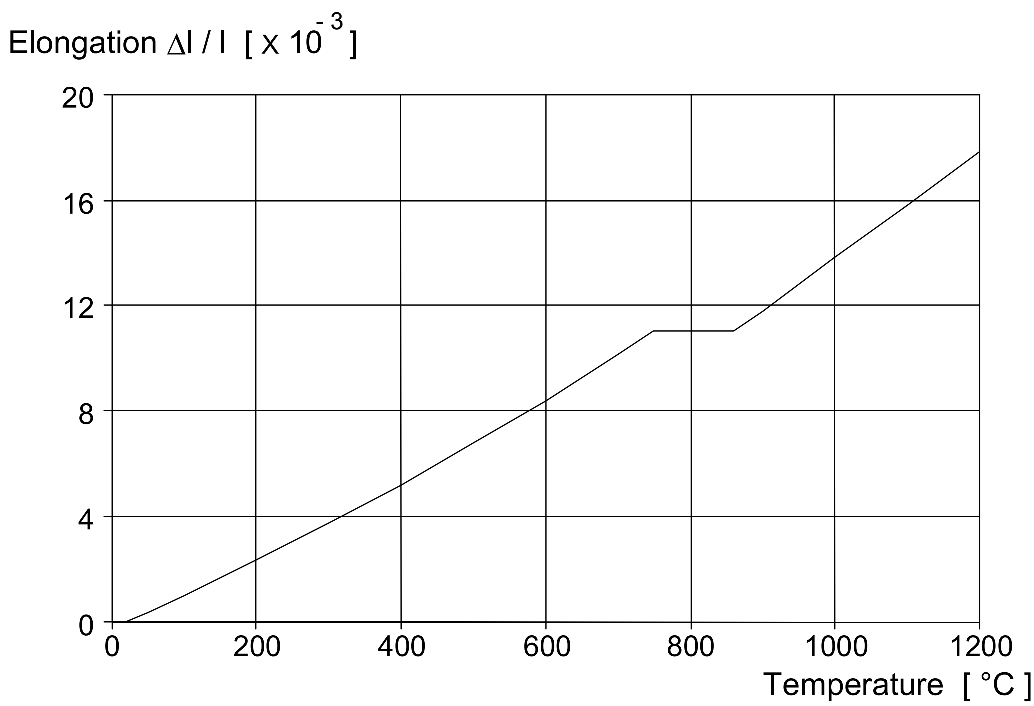


Figure 3.3: Thermal elongation of carbon steel as a function of the temperature

3.4.1.2 Specific heat

RC (1) The specific heat of steel c_a should be determined from the following:

- for $20°C \leq \theta_a < 600°C$:

$$c_a = 425 + 7{,}73 \times 10^{-1}\,\theta_a - 1{,}69 \times 10^{-3}\,\theta_a^2 + 2{,}22 \times 10^{-6}\,\theta_a^3 \text{ J/kgK} \qquad (3.2a)$$

- for $600°C \leq \theta_a < 735°C$:

$$c_a = 666 + \frac{13002}{738 - \theta_a} \text{ J/kgK} \qquad (3.2b)$$

- for $735°C \leq \theta_a < 900°C$:

$$c_a = 545 + \frac{17820}{\theta_a - 731} \text{ J/kgK} \qquad (3.2c)$$

- for $900°C \leq \theta_a \leq 1200°C$:

$$c_a = 650 \text{ J/kgK} \qquad (3.2d)$$

where:

θ_a is the steel temperature [°C].

NOTE: The variation of the specific heat with temperature is illustrated in figure 3.4.

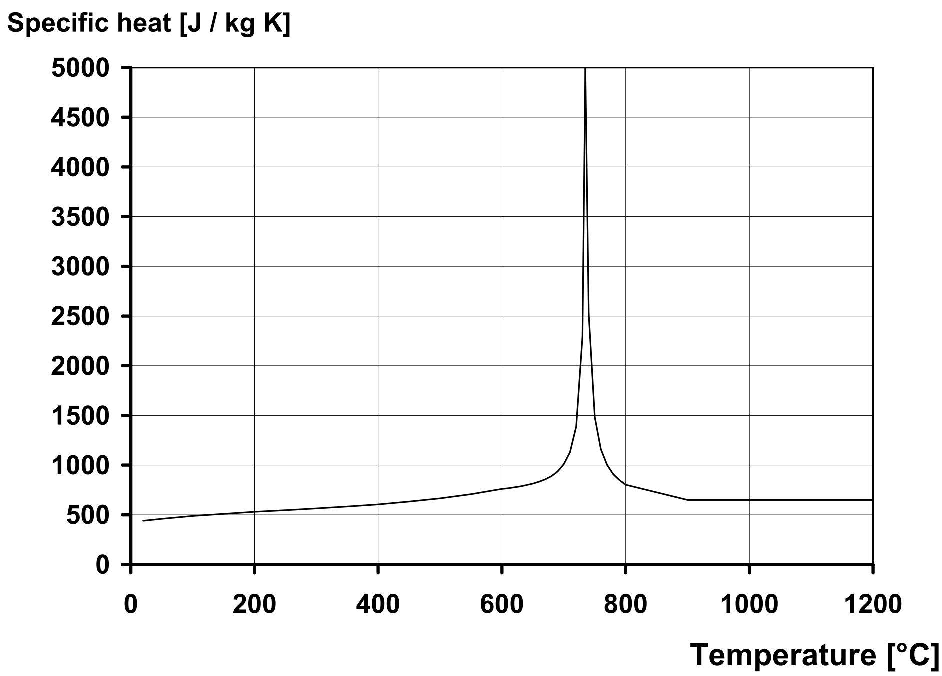


Figure 3.4: Specific heat of carbon steel as a function of the temperature

3.4.1.3 Thermal conductivity

RC (1) The thermal conductivity of steel λ_a should be determined from the following:

- for $20°C \leq \theta_a < 800°C$:

$$\lambda_a = 54 - 3{,}33 \times 10^{-2}\, \theta_a \text{ W/mK} \quad (3.3a)$$

- for $800°C \leq \theta_a \leq 1200°C$:

$$\lambda_a = 27{,}3 \text{ W/mK} \quad (3.3b)$$

where:

θ_a is the steel temperature [°C].

NOTE: The variation of the thermal conductivity with temperature is illustrated in figure 3.5.

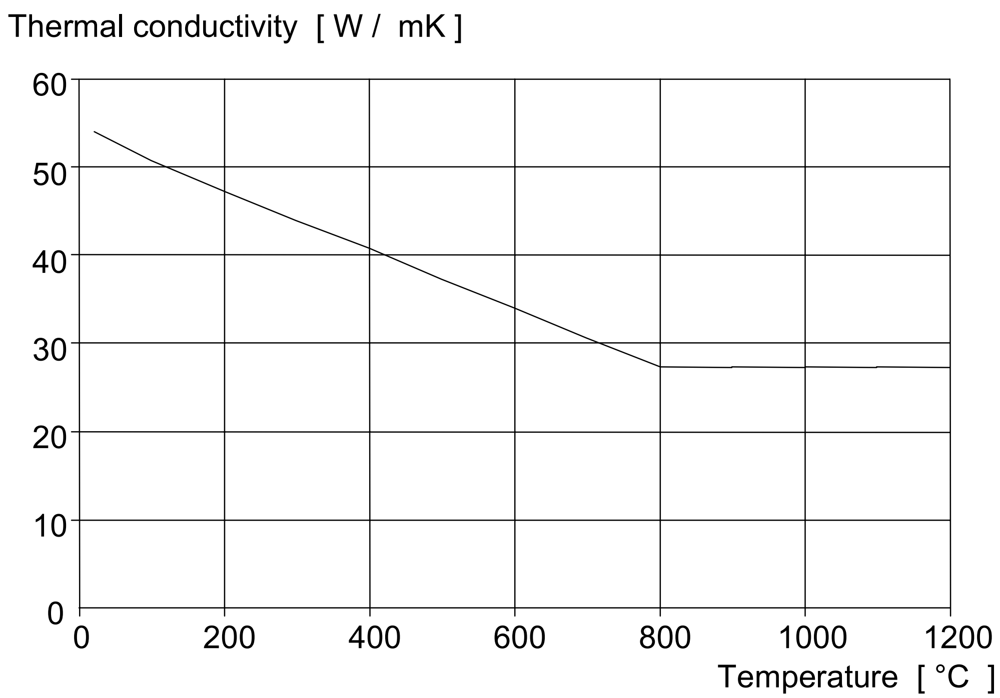


Figure 3.5: Thermal conductivity of carbon steel as a function of the temperature

3.4.2 Stainless steels

PE (1) The thermal properties of stainless steels may be taken from annex C.

3.4.3 Fire protection materials

RC (1) The properties and performance of fire protection materials used in design should have been assessed using the test procedures given in ENV 13381-1, ENV 13381-2 or ENV 13381-4 as appropriate.

NOTE: These standards include a requirement that the fire protection materials shall remain coherent and cohesive to their supports throughout the relevant fire exposure.

4 Structural fire design

4.1 General

ST (1) This section gives rules for steelwork that can be either:

- unprotected;
- insulated by fire protection material;
- protected by heat screens.

NOTE: Examples of other protection methods are water filling or partial protection in walls and floors.

PE (2) Fire resistance may be determined by one or more of the following approaches:

- simple calculation models;
- advanced calculation models;
- testing.

DF (3) Simple calculation models are simplified design methods for individual members, which are based on conservative assumptions.

DF (4) Advanced calculation models are design methods in which engineering principles are applied in a realistic manner to specific applications.

4.2 Simple calculation models

4.2.1 General

RQ (1)P The load-bearing function of a steel member shall be assumed to be maintained after a time t in a given fire if:

$$E_{\mathrm{fi,d}} \leq R_{\mathrm{fi,d,t}} \tag{4.1}$$

where:

$E_{\mathrm{fi,d}}$ is the design effect of actions for the fire design situation, according to EN 1991-1-2;

$R_{\mathrm{fi,d,t}}$ is the corresponding design resistance of the steel member, for the fire design situation, at time t.

RQ (2)P The design resistance $R_{\mathrm{fi,d,t}}$ at time t shall be determined, usually in the hypothesis of a uniform temperature in the cross-section, by modifying the design resistance for normal temperature design to EN 1993-1-1, to take account of the mechanical properties of steel at elevated temperatures, see 4.2.3.

NOTE: In 4.2.3 $R_{\mathrm{fi,d,t}}$ becomes $M_{\mathrm{fi,t,Rd}}$, $N_{\mathrm{fi,t,Rd}}$ etc (separately or in combination) and the corresponding values of $M_{\mathrm{fi,Ed}}$, $N_{\mathrm{fi,Ed}}$ etc represent $E_{\mathrm{fi,d}}$.

PE (3) For steel section the hypothesis of a uniform temperature in the cross section may be used. If a non uniform temperature distribution is used, the design resistance for normal temperature design to EN1993-1-1 is modified on the base of this temperature distribution.

PE (4) Alternatively to (1), by using a uniform temperature distribution, the verification may be carried out in the temperature domain, see 4.2.4.

PE

(5) Net-section failure at fastener holes need not be considered, provided that there is a fastener in each hole, because the steel temperature is lower at connections due to the presence of additional material.

PE

(6) The fire resistance of a bolted or a welded connection may be assumed to be sufficient provided that the following conditions are satisfied:

1. The thermal resistance $(d_f/\lambda_f)_c$ of the connection's fire protection should be greater than the minimum value of thermal resistance $(d_f/\lambda_f)_m$ of fire protection applied to any of the jointed members.

   Where:
   d_f is the thickness of the fire protection material. ($d_f = 0$ for unprotected members.)
   λ_f is the effective thermal conductivity of the fire protection material.

2. The utilisation of the connection should be less than the maximum value of utilisation of any of the connected members.

3. The resistance of the connection at ambient temperature should satisfy the recommendations given in EN1993-1.8.

PE

(7) As an alternative to the method given in clause 4.2.1 (6) the fire resistance of a connection may be determined using the method given in Annex D.

NOTE: As a simplification the comparison of the level of utilisation within the connections and joined members may be performed for room temperature.

4.2.2 Classification of cross-sections

RC

(1) For the purpose of these simplified rules the cross-sections may be classified as for normal temperature design without considering any change by increasing temperature.

NOTE: See EN1993-1-1

4.2.3 Resistance

4.2.3.1 Tension members

RC

(1) The design resistance $N_{fi,\theta,Rd}$ of a tension member with a uniform temperature θ_a should be determined from:

$$N_{fi,\theta,Rd} = k_{y,\theta} N_{Rd} [\gamma_{M,1} / \gamma_{M,fi}] \tag{4.2}$$

where:

$k_{y,\theta}$ is the reduction factor for the yield strength of steel at temperature θ_a,reached at time t see section 3;

N_{Rd} is the design resistance of the cross-section $N_{pl,Rd}$ for normal temperature design, according to EN 1993-1-1.

PE

(2) The design resistance $N_{fi,t,Rd}$ at time t of a tension member with a non-uniform temperature distribution across the cross-section may be determined from:

$$N_{fi,t,Rd} = \sum_{i=1}^{n} A_i \, k_{y,\theta,i} \, f_y \, / \, \gamma_{M,fi} \tag{4.3}$$

where:

A_i is an elemental area of the cross-section with a temperature θ_i;

$k_{y,\theta,i}$ is the reduction factor for the yield strength of steel at temperature θ_i, see section 3;

θ_i is the temperature in the elemental area A_i.

PE

(3) The design resistance $N_{fi,t,Rd}$ at time t of a tension member with a non-uniform temperature distribution may conservatively be taken as equal to the design resistance $N_{fi,\theta,Rd}$ of a tension member with a uniform steel temperature θ_a equal to the maximum steel temperature $\theta_{a,max}$ reached at time t.

4.2.3.2 Compression members with Class 1, Class 2 or Class 3 cross-sections

RC

(1) The design buckling resistance $N_{b,fi,t,Rd}$ at time t of a compression member with a Class 1, Class 2 or Class 3 cross-section with a uniform temperature θ_a should be determined from:

$$N_{b,fi,t,Rd} = \chi_{fi} A\, k_{y,\theta} f_y / \gamma_{M,fi} \tag{4.4}$$

where:

χ_{fi} is the reduction factor for flexural buckling in the fire design situation;

$k_{y,\theta}$ is the reduction factor from section 3 for the yield strength of steel at the steel temperature θ_a reached at time t.

RC

(2) The value of χ_{fi} should be taken as the lesser of the values of $\chi_{y,fi}$ and $\chi_{z,fi}$ determined according to:

$$\chi_{fi} = \frac{1}{\varphi_\theta + \sqrt{\varphi_\theta^2 - \bar{\lambda}_\theta^2}} \tag{4.5}$$

with

$$\varphi_\theta = \frac{1}{2}[1 + \alpha\, \bar{\lambda}_\theta + \bar{\lambda}_\theta^2]$$

and

$$\alpha = 0{,}65\sqrt{235 / f_y}$$

The non-dimensional slenderness $\bar{\lambda}_\theta$ for the temperature θ_a, is given by:

$$\bar{\lambda}_\theta = \bar{\lambda}\,[k_{y,\theta} / k_{E,\theta}]^{0,5} \tag{4.6}$$

where:

$k_{y,\theta}$ is the reduction factor from section 3 for the yield strength of steel at the steel temperature θ_a reached at time t;

$k_{E,\theta}$ is the reduction factor from section 3 for the slope of the linear elastic range at the steel temperature θ_a reached at time t.

RC

(3) The buckling length l_{fi} of a column for the fire design situation should generally be determined as for normal temperature design. However, in a braced frame the buckling length l_{fi} of a column length may be determined by considering it as fixed in direction at continuous or semi-continuous connections to the column lengths in the fire compartments above and below, provided that the fire resistance of the building components that separate these fire compartments is not less than the fire resistance of the column.

PE (4) In the case of a braced frame in which each storey comprises a separate fire compartment with sufficient fire resistance, in an intermediate storey the buckling length l_{fi} of a continuos column may be taken as $l_{fi} = 0,5L$ and in the top storey the buckling length may be taken as $l_{fi} = 0,7L$, where L is the system length in the relevant storey, see figure 4.1.

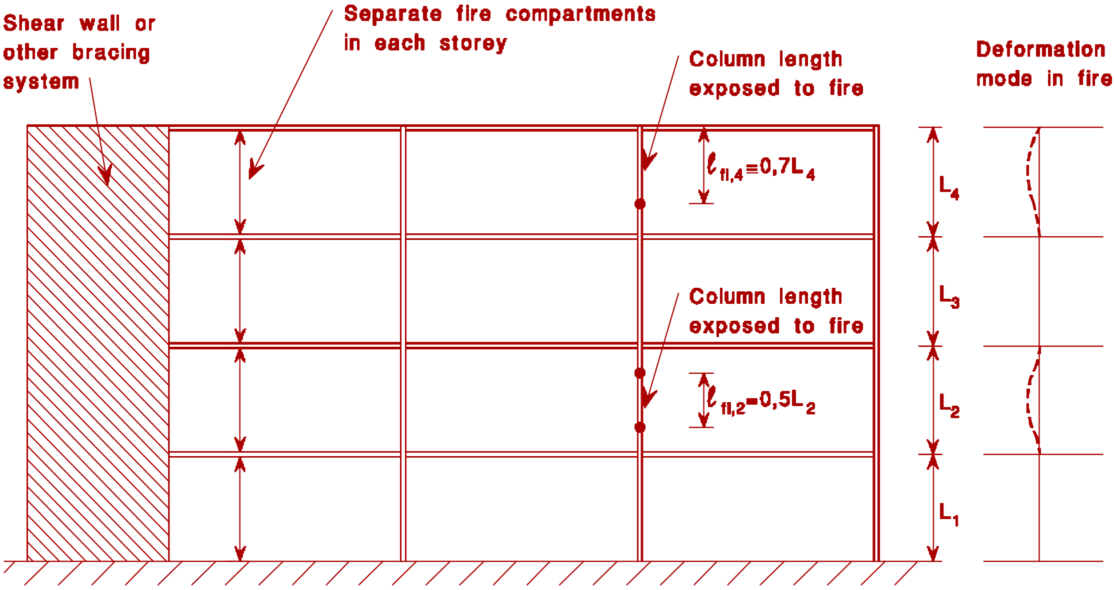


Figure 4.1: Buckling lengths l_{fi} of columns in braced frames

PE (5) The design resistance $N_{b,fi,t,Rd}$ at time t of a compression member with a non-uniform temperature distribution may be taken as equal to the design resistance $N_{b,fi,\theta,Rd}$ of a compression member with a uniform steel temperature θ_a equal to the maximum steel temperature $\theta_{a,max}$ reached at time t.

4.2.3.3 Beams with Class 1 or Class 2 cross-sections

RC (1) The design moment resistance $M_{fi,\theta,Rd}$ of a Class 1 or Class 2 cross-section with a uniform temperature θ_a should be determined from:

$$M_{fi,\theta,Rd} = k_{y,\theta}[\gamma_{M,1}/\gamma_{M,fi}]M_{Rd} \tag{4.7}$$

where:

M_{Rd} is the plastic moment resistance of the gross cross-section $M_{pl,Rd}$ for normal temperature design, according to EN 1993-1-1 or the reduced moment resistance for normal temperature design, allowing for the effects of shear if necessary, according to EN 1993-1-1;

$k_{y,\theta}$ is the reduction factor for the yield strength of steel at temperature θ_a, see section 3

PE (2) The design moment resistance $M_{fi,t,Rd}$ at time t of a Class 1 or Class 2 cross-section with a non-uniform temperature distribution across the cross-section may be determined from:

$$M_{fi,t,Rd} = \sum_{i=1}^{n} A_i\, z_i\, k_{y,\theta,i}\, f_{y,i} / \gamma_{M,fi} \tag{4.8}$$

where:

z_i is the distance from the plastic neutral axis to the centroid of the elemental area A_i;

$f_{y,i}$ is the nominal yield strength f_y for the elemental area A_i taken as positive on the compression side of the plastic neutral axis and negative on the tension side;

A_i and $k_{y,\theta,i}$ are as defined in 4.2.3.1 (2).

PE (3) Alternatively, the design moment resistance $M_{fi,t,Rd}$ at time t of a Class 1 or Class 2 cross-section in a member with a non-uniform temperature distribution, may be determined from:

$$M_{fi,t,Rd} = M_{fi,\theta,Rd}/\kappa_1\,\kappa_2 \tag{4.9}$$

where:

$M_{fi,\theta,Rd}$	is	the design moment resistance of the cross-section for a uniform temperature θ_a which is equal to the uniform temperature θ_a at time t in a cross-section which is not thermally influenced by the supports.;
κ_1	is	an adaptation factor for non-uniform temperature across the cross-section, see (7);
κ_2	is	an adaptation factor for non-uniform temperature along the beam, see (8).

RC (4) The design lateral torsional buckling resistance moment $M_{b,fi,t,Rd}$ at time t of a laterally unrestrained beam with a Class 1 or Class 2 cross-section should be determined from:

$$M_{b,fi,t,Rd} = \chi_{LT,fi}\, W_{pl,y}\, k_{y,\theta,com} f_y \,/\, \gamma_{M,fi} \tag{4.10}$$

where:

$\chi_{LT,fi}$	is	the reduction factor for lateral-torsional buckling in the fire design situation;
$k_{y,\theta,com}$	is	the reduction factor from section 3 for the yield strength of steel at the maximum temperature in the compression flange $\theta_{a,com}$ reached at time t.

NOTE : Conservatively $\theta_{a,com}$ can be assumed to be equal to the uniform temperature θ_a.

RC (5) The value of $\chi_{LT,fi}$ should be determined according to the following equations:

$$\chi_{LT,fi} = \frac{1}{\phi_{LT,\theta,com} + \sqrt{[\phi_{LT,\theta,com}]^2 - [\bar{\lambda}_{LT,\theta,com}]^2}} \tag{4.11}$$

with

$$\phi_{LT,\theta,com} = \frac{1}{2}\left[1 + \alpha\bar{\lambda}_{LT,\theta,com} + (\bar{\lambda}_{LT,\theta,com})^2\right] \tag{4.12}$$

and

$$\alpha = 0.65\sqrt{235/f_y} \tag{4.13}$$

$$\bar{\lambda}_{LT,\theta,com} = \bar{\lambda}_{LT}\,[k_{y,\theta,com}/k_{E,\theta,com}]^{0,5} \tag{4.14}$$

where:

$k_{E,\theta,com}$	is	the reduction factor from section 3 for the slope of the linear elastic range at the maximum steel temperature in the compression flange $\theta_{a,com}$ reached at time t.

RC (6) The design shear resistance $V_{fi,t,Rd}$ at time t of a Class 1 or Class 2 cross-section should be determined from:

$$V_{fi,t,Rd} = k_{y,\theta,\,web,}\, V_{Rd}\,[\gamma_{M,1}/\gamma_{M,fi}] \tag{4.15}$$

where:

$V_{\rm Rd}$	is	the shear resistance of the gross cross-section for normal temperature design, according to EN 1993-1-1;
θ_{web}	is	the average temperature in the web of the section;
$k_{\rm y,\theta,\omega\varepsilon\beta}$	is	the reduction factor for the yield strength of steel at the steel temperature $\theta_{\rm web}$, see section 3.

RC (7) The value of the adaptation factor κ_1 for non-uniform temperature distribution across a cross-section should be taken as follows:

- for a beam exposed on all four sides: $\kappa_1 = 1{,}0$
- for an unprotected beam exposed on three sides, with a composite or concrete slab on side four: $\kappa_1 = 0{,}70$
- for an protected beam exposed on three sides, with a composite or concrete slab on side four: $\kappa_1 = 0{,}85$

RC (8) For a non-uniform temperature distribution along a beam the adaptation factor κ_2 should be taken as follows:

- at the supports of a statically indeterminate beam: $\kappa_2 = 0{,}85$
- in all other cases: $\kappa_2 = 1{,}0.$

4.2.3.4 Beams with Class 3 cross-sections

RC (1) The design moment resistance $M_{\rm fi,t,Rd}$ at time t of a Class 3 cross-section with a uniform temperature should be determined from:

$$M_{\rm fi,t,Rd} = k_{\rm y,\theta} M_{\rm Rd} [\gamma_{\rm M,1} / \gamma_{\rm M,fi}] \tag{4.16}$$

where:

$M_{\rm Rd}$	is	the elastic moment resistance of the gross cross-section $M_{\rm el,Rd}$ for normal temperature design, according to EN 1993-1-1 or the reduced moment resistance allowing for the effects of shear if necessary according to EN 1993-1-1;
$k_{\rm y,\theta}$	is	the reduction factor for the yield strength of steel at the steel temperature $\theta_{\rm a}$, see section 3.

PE (2) The design moment resistance $M_{\rm fi,t,Rd}$ at time t of a Class 3 cross-section with a non-uniform temperature distribution may be determined from:

$$M_{\rm fi,t,Rd} = k_{\rm y,\theta,max} M_{\rm Rd} [\gamma_{\rm M,1} / \gamma_{\rm M,fi}] / \kappa_1 \kappa_2 \tag{4.17}$$

where:

$M_{\rm Rd}$	is	the elastic moment resistance of the gross cross-section $M_{\rm el,Rd}$ for normal temperature design or the reduced moment resistance allowing for the effects of shear if necessary according to EN 1993-1-1;
$k_{\rm y,\theta,max}$	is	the reduction factor for the yield strength of steel at the maximum steel temperature $\theta_{\rm a,max}$ reached at time t, see 3;
κ_1	is	an adaptation factor for non-uniform temperature in a cross-section, see 4.2.3.3 (7);
κ_2	is	an adaptation factor for non-uniform temperature along the beam, see 4.2.3.3 (8).

RC (3) The design buckling resistance moment $M_{\rm b,fi,t,Rd}$ at time t of a laterally unrestrained beam with a Class 3 cross-section should be determined from:

$$M_{\rm b,fi,t,Rd} = \chi_{\rm LT,fi} W_{\rm el,y} k_{\rm y,\theta,com} f_{\rm y} / \gamma_{\rm M,fi} \tag{4.18}$$

where:

$\chi_{LT,fi}$ is as given in 4.2.3.3 (5).

NOTE: Conservatively $\theta_{a,com}$ can be assumed to be equal to the maximum temperature $\theta_{a,max}$.

RC

(4) The design shear resistance $V_{fi,t,Rd}$ at time t of a Class 3 cross-section should be determined from:

$$V_{fi,t,Rd} = k_{y,\theta,web} V_{Rd} [\gamma_{M,1} / \gamma_{M,fi}] \tag{4.19}$$

where:

V_{Rd} is the shear resistance of the gross cross-section for normal temperature design, according to EN 1993-1-1.

4.2.3.5 Members with Class 1, 2 or 3 cross-sections, subject to combined bending and axial compression

RC

(1) The design buckling resistance $R_{fi,t,d}$ at time t of a member subject to combined bending and axial compression should be verified by satisfying expressions (4.20a) and (4.20b) for a member with a Class 1 or Class 2 cross-section, or expressions (4.20c) and (4.20d) for a member with a Class 3 cross-section.

$$\frac{N_{fi,Ed}}{\chi_{\min,fi}\, A\, k_{y,\theta} \dfrac{f_y}{\gamma_{M,fi}}} + \frac{k_y\, M_{y,fi,Ed}}{W_{pl,y}\, k_{y,\theta} \dfrac{f_y}{\gamma_{M,fi}}} + \frac{k_z\, M_{z,fi,Ed}}{W_{pl,z}\, k_{y,\theta} \dfrac{f_y}{\gamma_{M,fi}}} \leq 1 \tag{4.20a}$$

$$\frac{N_{fi,Ed}}{\chi_{z,fi}\, A\, k_{y,\theta} \dfrac{f_y}{\gamma_{M,fi}}} + \frac{k_{LT}\, M_{y,fi,Ed}}{\chi_{LT,fi} W_{pl,y}\, k_{y,\theta} \dfrac{f_y}{\gamma_{M,fi}}} + \frac{k_z\, M_{z,fi,Ed}}{W_{pl,z}\, k_{y,\theta} \dfrac{f_y}{\gamma_{M,fi}}} \leq 1 \tag{4.20b}$$

$$\frac{N_{fi,Ed}}{\chi_{\min,fi}\, A\, k_{y,\theta} \dfrac{f_y}{\gamma_{M,fi}}} + \frac{k_y\, M_{y,fi,Ed}}{W_{el,y}\, k_{y,\theta} \dfrac{f_y}{\gamma_{M,fi}}} + \frac{k_z\, M_{z,fi,Ed}}{W_{el,z}\, k_{y,\theta} \dfrac{f_y}{\gamma_{M,fi}}} \leq 1 \tag{4.20c}$$

$$\frac{N_{fi,Ed}}{\chi_{z,fi}\, A\, k_{y,\theta} \dfrac{f_y}{\gamma_{M,fi}}} + \frac{k_{LT}\, M_{y,fi,Ed}}{\chi_{LT,fi}\, W_{el,y}\, k_{y,\theta} \dfrac{f_y}{\gamma_{M,fi}}} + \frac{k_z\, M_{z,fi,Ed}}{W_{el,z}\, k_{y,\theta} \dfrac{f_y}{\gamma_{M,fi}}} \leq 1 \tag{4.20d}$$

where:

$\chi_{min,fi}$ is as defined in 4.2.3.2;

$\chi_{z,fi}$ is as defined in 4.2.3.2;

$\chi_{LT,fi}$ is as defined in 4.2.3.3 (5);

$$k_{LT} = 1 - \frac{\mu_{LT}\, N_{fi,Ed}}{\chi_{z,fi}\, A\, k_{y,\theta} \dfrac{f_y}{\gamma_{M,fi}}} \leq 1$$

with: $\mu_{LT} = 0{,}15 \bar{\lambda}_{z,\theta}\ \beta_{M.LT} - 0{,}15 \leq 0{,}9$

$$k_y = 1 - \frac{N_{fi,Ed}}{\chi_{y,fi}\, A\, k_{y,\theta}\, \dfrac{f_y}{\gamma_{M,fi}}} \leq 3$$

with: $\mu_y = \left(1{,}2\,\beta_{M.y} - 3\right)\overline{\lambda}_{y,\theta} + 0{,}44\,\beta_{M,y} - 0{,}29 \leq 0{,}8$

$$k_z = 1 - \frac{N_{fi,Ed}}{\chi_{z,fi}\, A\, k_{y,\theta}\, \dfrac{f_y}{\gamma_{M,fi}}} \leq 3$$

with: $\mu_z = \left(2\,\beta_{M.z} - 5\right)\overline{\lambda}_{z,\theta} + 0{,}44\,\beta_{M,z} - 0{,}29 \leq 0{,}8$ and $\overline{\lambda}_{z,\theta} \leq 1{,}1$

NOTE: For the equivalent uniform moment factors β_M see figure 4.2.

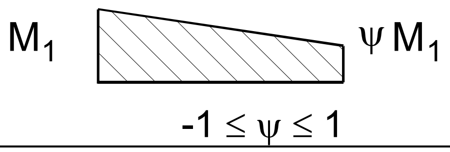
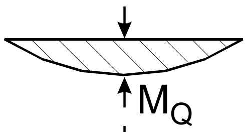
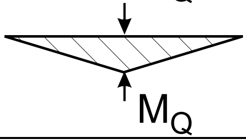
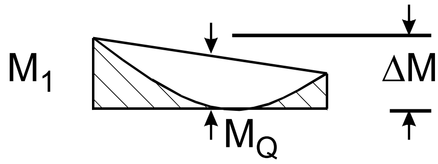
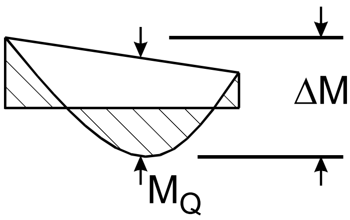
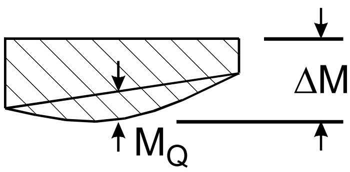
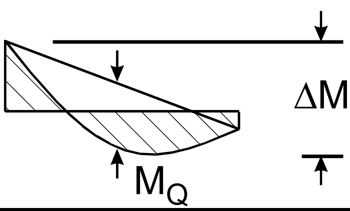
Moment diagram	**Equivalent uniform moment factor** β_M
End moments 	$\beta_{M,\psi} = 1{,}8 - 0{,}7\ \psi$
Moments due to in-plane lateral loads  	$\beta_{M,Q} = 1{,}3$ $\beta_{M,Q} = 1{,}4$
Moments due to in-plane lateral loads plus end moments    	$\beta_M = \beta_{M,\psi} + \frac{M_Q}{\Delta M}(\beta_{M,Q} - \beta_{M,\psi})$ $M_Q = \mid \max M \mid$ due to lateral load only $\Delta M = \begin{cases} \lvert \max M \rvert & \text{for moment diagram without change of sign} \\ \lvert \max M \rvert + \lvert \min M \rvert & \text{for moment diagram with change of sign} \end{cases}$

Figure 4.2: Equivalent uniform moment factors.

4.2.3.6 Members with Class 4 cross-sections

PE (1) For members with class 4 cross-sections other than tension members it may be assumed that 4.2.1(1) is satisfied if at time t the steel temperature θ_a at all cross-sections is not more than 350° C.

NOTE 1 : For further information see annex E.
NOTE 2 : Other values than 350°C may be given in the national annex.

4.2.4 Critical temperature

PE (1) As an alternative to 4.2.3, verification may be carried out in the temperature domain.

PE (2) Except when considering deformation criteria or when stability phenomena have to be taken into account, the critical temperature $\theta_{a,cr}$ of carbon steel according to 1.1.1 (6) at time t for a uniform temperature distribution in a member may be determined for any degree of utilisation μ_0 at time $t = 0$ using:

$$\theta_{a,cr} = 39{,}19 \ln\left[\frac{1}{0{,}9674\,\mu_0^{\,3{,}833}} - 1\right] + 482 \qquad (4.21)$$

NOTE : Examples for values of $\theta_{a,cr}$ for values of μ_0 from 0,135 to 0,80 are given in table 4.1.

PE (4) For members with Class 1, Class 2 or Class 3 cross-sections and for all tension members, the degree of utilisation μ_0 at time $t = 0$ may be obtained from:

$$\mu_0 = E_{fi,d}/R_{fi,d,0} \qquad (4.22)$$

where:

$R_{fi,d,0}$ is the value of $R_{fi,d,t}$ for time $t = 0$, from 4.2.3;
$E_{fi,d}$ and $R_{fi,d,t}$ are as defined in 4.2.1(1).

PE (5) Alternatively for tension members, and for beams where lateral-torsional buckling is not a potential failure mode, μ_0 may conservatively be obtained from:

$$\mu_0 = \eta_{fi}\left[\gamma_{M,fi}/\gamma_{M1}\right] \qquad (4.23)$$

where:

η_{fi} is the reduction factor defined in 2.4.3(3).

Table 4.1: Critical temperature $\theta_{a,cr}$ for values of the utilisation factor μ_0

μ_0	$\theta_{a,cr}$	μ_0	$\theta_{a,cr}$	μ_0	$\theta_{a,cr}$
0,22	711	0,42	612	0,62	549
0,24	698	0,44	605	0,64	543
0,26	685	0,46	598	0,66	537
0,28	674	0,48	591	0,68	531
0,30	664	0,50	585	0,70	526
0,32	654	0,52	578	0,72	520
0,34	645	0,54	572	0,74	514
0,36	636	0,56	566	0,76	508
0,38	628	0,58	560	0,78	502
0,40	620	0,60	554	0,80	496

NOTE: The national annex may give default values for critical temperatures.

4.2.5 Steel temperature development

4.2.5.1 Unprotected internal steelwork

RC (1) For an equivalent uniform temperature distribution in the cross-section, the increase of temperature $\Delta\theta_{a,t}$ in an unprotected steel member during a time interval Δt should be determined from:

$$\Delta\theta_{a,t} = 0{,}9\, k_{shadow} \frac{A_m / V}{c_a \rho_a} \dot{h}_{net,d} \Delta t \tag{4.24}$$

where:

k_{shadow}	is	correction factor for the shadow effect, from 5.2.5.1(2)
A_m/V	is	the section factor for unprotected steel members;
A_m	is	the surface area of the member per unit length [m²];
V	is	the volume of the member per unit length [m³];
c_a	is	the specific heat of steel, from section 3 [J/kgK];
$\dot{h}_{net,d}$	is	the design value of the net heat flux per unit area [W/m^2];
Δt	is	the time interval [seconds];
ρ_a	is	the unit mass of steel, from section 3 [kg/m^3].

RC (2) The correction factor for the shadow effect may be determined from:

$$k_{shadow} = [A_m/V]_{box}/[A_m/V] \tag{4.25}$$

where:

$[A_m/V]_{box}$ is box value of the section factor.

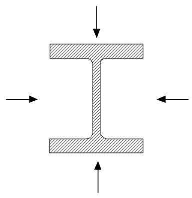
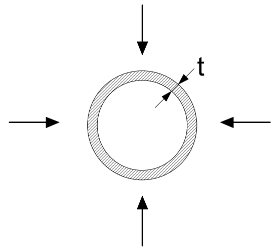
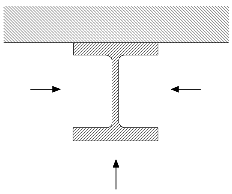
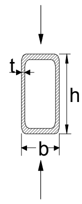
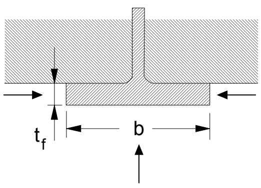
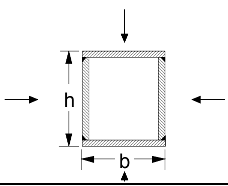
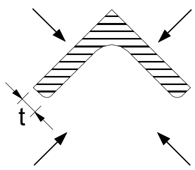
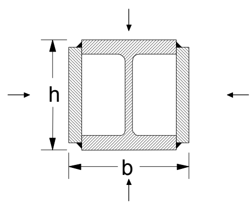
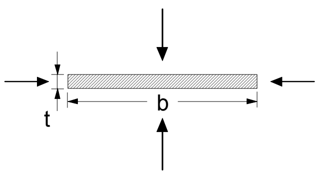
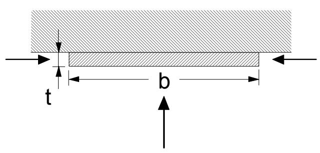
RC (3) The value of $\dot{h}_{net,d}$ should be obtained from EN 1991-1-2 using $\varepsilon_f = 1{,}0$ and ε_m according to 2.2(2), where ε_f, ε_m are as defined in EN 1991-1-2.

RC (4) The value of Δt should not be taken as more than 5 seconds.

RC (5) In expression (4.24) the value of the section factor A_m/V should not be taken as less than $10\,m^{-1}$.

NOTE: Some expressions for calculating design values of the section factor A_m/V for unprotected steel members are given in table 4.2.

Table 4.2: Section factor A_m/V for unprotected steel members.

Open section exposed to fire on all sides: $\frac{A_m}{V} = \frac{\text{perimeter}}{\text{cross- section area}}$ 	Tube exposed to fire on all sides: $A_m/V = 1/t$ 
Open section exposed to fire on three sides: $\frac{A_m}{V} = \frac{\text{surface exposed to fire}}{\text{cross- section area}}$ 	Hollow section (or welded box section of uniform thickness) exposed to fire on all sides: If $t \ll b$: $A_m/V \approx 1/t$ 
I-section flange exposed to fire on three sides: $A_m/V = (b + 2t_f)/(bt_f)$ If $t \ll b$: $A_m/V \approx 1/t_f$  	Welded box section exposed to fire on all sides: $\frac{A_m}{V} = \frac{2(b + h)}{\text{cross- section area}}$ If $t \ll b$: $A_m/V \approx 1/t$ 
Angle exposed to fire on all sides: $A_m/V = 2/t$  	I-section with box reinforcement, exposed to fire on all sides: $\frac{A_m}{V} = \frac{2(b + h)}{\text{cross- section area}}$ 
Flat bar exposed to fire on all sides: $A_m/V = 2(b + t)/(bt)$ If $t \ll b$: $A_m/V \approx 2/t$  	Flat bar exposed to fire on three sides: $A_m/V = (b + 2t)/(bt)$ If $t \ll b$: $A_m/V \approx 1/t$ 

4.2.5.2 Internal steelwork insulated by fire protection material

RC (1) For a uniform temperature distribution in a cross-section, the temperature increase $\Delta\theta_{a,t}$ of an insulated steel member during a time interval Δt should be obtained from:

$$\Delta\theta_{a,t} = \frac{\lambda_p A_p / V}{d_p c_a \rho_a} \frac{(\theta_{g,t} - \theta_{a,t})}{(1 + \phi/3)} \Delta t - (e^{\phi/10} - 1)\Delta\theta_{g,t} \qquad (\text{ but } \Delta\theta_{a,t} \geq 0 \text{ if } \Delta\theta_{g,t} > 0) \qquad (4.26)$$

with:

$$\phi = \frac{c_p \rho_p}{c_a \rho_a} d_p A_p / V$$

where:

A_p/V	is	the section factor for steel members insulated by fire protection material;
A_p	is	the appropriate area of fire protection material per unit length of the member [m²];
V	is	the volume of the member per unit length [m³];
c_a	is	the temperature dependant specific heat of steel, from section 3 [J/kgK];
c_p	is	the temperature independent specific heat of the fire protection material [J/kgK];
d_p	is	the thickness of the fire protection material [m];
Δt	is	the time interval [seconds];
$\theta_{a,t}$	is	the steel temperature at time t [°C];
$\theta_{g,t}$	is	the ambient gas temperature at time t [°C];
$\Delta\theta_{g,t}$	is	the increase of the ambient gas temperature during the time interval Δt [K];
λ_p	is	the thermal conductivity of the fire protection system [W/mK];
ρ_a	is	the unit mass of steel, from section 3 [kg/m³];
ρ_p	is	the unit mass of the fire protection material [kg/m³].

RC (2) The values of c_p, λ_p and ρ_p should be determined as specified in section 3.

RC (3) The value of Δt should not be taken as more than 30 seconds.

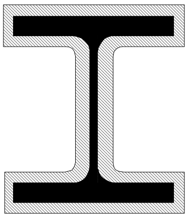
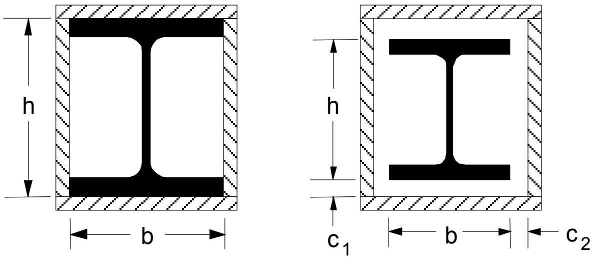
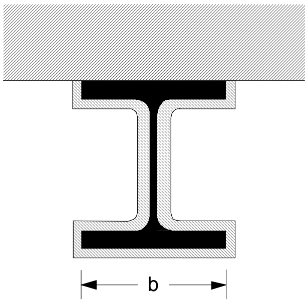
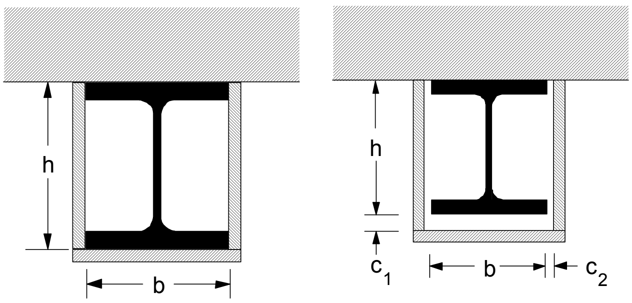
RC (4) The area A_p of the fire protection material should generally be taken as the area of its inner surface, but for hollow encasement with a clearance around the steel member the same value as for hollow encasement without a clearance may be adopted.

NOTE : Some design values of the section factor A_p/V for insulated steel members are given in table 4.3.

PE (5) For moist fire protection materials the calculation of the steel temperature increase $\Delta\theta_a$ may be modified to allow for a time delay in the rise of the steel temperature when it reaches 100 °C. This delay time should be determined by a method conforming with ENV 13381-4.

PE (6) As an alternative to 4.2.5.2 (1), the uniform temperature of an insulated steel member after a given time duration of standard fire exposure may be obtained using design flow charts derived in conformity with ENV 13381-4.

Table 4.3: Section factor A_p/V for steel members insulated by fire protection material

Sketch	Description	Section factor (A_p/V)
	Contour encasement of uniform thickness	$\dfrac{\text{steel perimeter}}{\text{steel cross-section area}}$
	Hollow encasement of uniform thickness)[1]	$\dfrac{2(b+h)}{\text{steel cross-section area}}$
	Contour encasement of uniform thickness, exposed to fire on three sides	$\dfrac{\text{steel perimeter - } b}{\text{steel cross-section area}}$
	Hollow encasement of uniform thickness, exposed to fire on three sides)[1]	$\dfrac{2h+b}{\text{steel cross-section area}}$

)[1] The clearance dimensions c_1 and c_2 should not normally exceed $h/4$

4.2.5.3 Internal steelwork in a void that is protected by heat screens

ST

(1) The provisions given below apply to both of the following cases:
- steel members in a void that have a floor on top and by a horizontal heat screen below, and
- steel members in a void that have vertical heat screens on both sides,

provided in both cases that there is a gap between the heat screen and the member. They do not apply if the heat screen is in direct contact with the member.

RC

(2) For internal steelwork protected by heat screens, the calculation of the steel temperature increase $\Delta\theta_a$ should be based on the methods given in 4.2.5.1 or 4.2.5.2 as appropriate, taking the ambient gas temperature $\theta_{g,t}$ as equal to the gas temperature in the void.

RC

(3) The properties and performance of the heat screens used in design should have been determined using a test procedure conforming with ENV13381-1 or ENV13381-2 as appropriate.

RC

(4) The temperature development in the void in which the steel members are situated should be determined from measurement according to ENV13381-1 or ENV13381-2 as appropriate.

4.2.5.4 External steelwork

RQ

(1)P The temperature of external steelwork shall be determined taking into account:
- the radiative heat flux from the fire compartment;
- the radiative heat flux and the convective heat flux from the flames emanating from openings;
- the radiative and convective heat loss from the steelwork to the ambient atmosphere;
- the sizes and locations of the structural members.

PE

(2)P Heat screens may be provided on one, two or three sides of an external steel member in order to protect it from radiative heat transfer.

RC

(3) Heat screens should be either:
- directly attached to that side of the steel member that it is intended to protect, or
- large enough to fully screen that side from the expected radiative heat flux.

RC

(4) Heat screens may be used according to annex B if they are be non-combustible and have a fire resistance of at least EI 30 according to EN ISO 13501-2.

RC

(5) The temperature in external steelwork protected by heat screens should be determined as required in 4.2.5.4(1), assuming that there is no radiative heat transfer to those sides that are protected by heat screens.

PE

(6) Calculations may be based on steady state conditions resulting from a stationary heat balance using the methods given in annex B.

RC

(7) Design using annex B of this Part 1-2 of EN 1993 should be based on the model given in annex G of EN 1991-1-2 describing the compartment fire conditions and the flames emanating from openings, on which the calculation of the radiative and convective heat fluxes should be based.

4.3 Advanced calculation models

NOTE: The conditions for using advanced calculation models may be given in the National Annex.

4.3.1 General

RQ (1)P Advanced calculation methods shall provide a realistic analysis of structures exposed to fire. They shall be based on fundamental physical behaviour in such a way as to lead to a reliable approximation of the expected behaviour of the relevant structural component under fire conditions.

RQ (2)P Any potential failure modes not covered by the advanced calculation method (including local buckling and failure in shear) shall be eliminated by appropriate means.

RC (3) Advanced calculation methods should include separate calculation models for the determination of:
- the development and distribution of the temperature within structural members (thermal response model);
- the mechanical behaviour of the structure or of any part of it (mechanical response model).

PE (4) Advanced calculation methods may be used in association with any heating curve, provided that the material properties are known for the relevant temperature range.

PE (5) Advanced calculation methods may be used with any type of cross-section.

4.3.2 Thermal response

RQ (1)P Advanced calculation methods for thermal response shall be based on the acknowledged principles and assumptions of the theory of heat transfer.

RQ (2)P The thermal response model shall consider:
- the relevant thermal actions specified in EN 1991-1-2;
- the variation of the thermal properties of the material with the temperature, see section 3.

PE (3) The effects of non-uniform thermal exposure and of heat transfer to adjacent building components may be included where appropriate.

PE (4) The influence of any moisture content and of any migration of the moisture within the fire protection material may conservatively be neglected.

4.3.3 Mechanical response

RQ (1)P Advanced calculation methods for mechanical response shall be based on the acknowledged principles and assumptions of the theory of structural mechanics, taking into account the changes of mechanical properties with temperature.

RQ (2)P The effects of thermally induced strains and stresses both due to temperature rise and due to temperature differentials, shall be considered.

RQ (3)P The mechanical response of the model shall also take account of:
- the combined effects of mechanical actions, geometrical imperfections and thermal actions;
- the temperature dependent mechanical properties of the material, see section 3;
- geometrical non-linear effects;
- the effects of non-linear material properties, including the unfavourable effects of loading and unloading on the structural stiffness.

PE (4) Provided that the stress-strain relationships given in section 3 are used, the effects of transient thermal creep need not be given explicit consideration.

RQ (5)P The deformations at ultimate limit state implied by the calculation method shall be limited to ensure that compatibility is maintained between all parts of the structure.

RC (6) The design should take into account the ultimate limit state beyond which the calculated deformations of the structure would cause failure due to the loss of adequate support to one of the members.

RC (7) For the analysis of isolated vertical members a sinusoidal initial imperfection with a maximum value of h/1000 at mid-height should be used, when not specified by relevant product standards.

4.3.4 Validation of advanced calculation models

(1)P A verification of the accuracy of the calculation models shall be made on basis of relevant test results.

(2) Calculation results may refer to temperatures, deformations and fire resistance times.

(3)P The critical parameters shall be checked to ensure that the model complies with sound engineering principles, by means of a sensitivity analysis.

(4) Critical parameters may refer, for example to the buckling length, the size of the elements, the load level.

Annex A [normative] Strain-hardening of carbon steel at elevated temperatures

PE

(1) For temperatures below $400\,°C$, the alternative strain-hardening option mentioned in 3.2 may be used as follows:

- for $0{,}02 < \varepsilon < 0{,}04$:

$$\sigma_a = 50(f_{u,\theta} - f_{y,\theta})\varepsilon + 2f_{y,\theta} - f_{u,\theta} \tag{A.1a}$$

- for $0{,}04 \leq \varepsilon \leq 0{,}15$:

$$\sigma_a = f_{u,\theta} \tag{A.1b}$$

- for $0{,}15 < \varepsilon < 0{,}20$:

$$\sigma = f_{u,\theta}[1 - 20(\varepsilon - 0{,}15)] \tag{A.1c}$$

- for $\varepsilon \geq 0{,}20$:

$$\sigma_a = 0{,}00 \tag{A.1d}$$

where:

$f_{u,\theta}$ is the ultimate strength at elevated temperature, allowing for strain-hardening.

NOTE: The alternative stress-strain relationship for steel, allowing for strain hardening, is illustrated in figure A.1.

PE

(2) The ultimate strength at elevated temperature, allowing for strain hardening, should be determined as follows:

- for $\theta_a < 300\,°C$:

$$f_{u,\theta} = 1{,}25 f_{y,\theta} \tag{A.2a}$$

- for $300\,°C \leq \theta_a < 400\,°C$:

$$f_{u,\theta} = f_{y,\theta}(2 - 0{,}0025\,\theta_a) \tag{A.2b}$$

- for $\theta_a \geq 400\,°C$:

$$f_{u,\theta} = f_{y,\theta} \tag{A.2c}$$

NOTE: The variation of the alternative stress-strain relationship with temperature is illustrated in figure A.2.

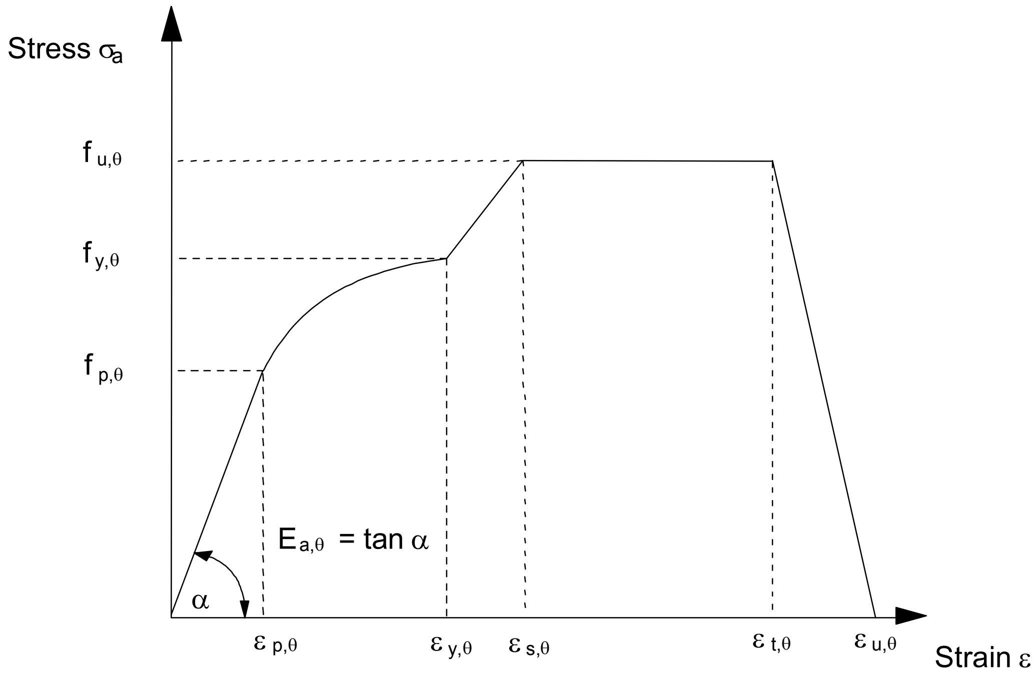


Figure A.1: Alternative stress-strain relationship for steel allowing for strain-hardening

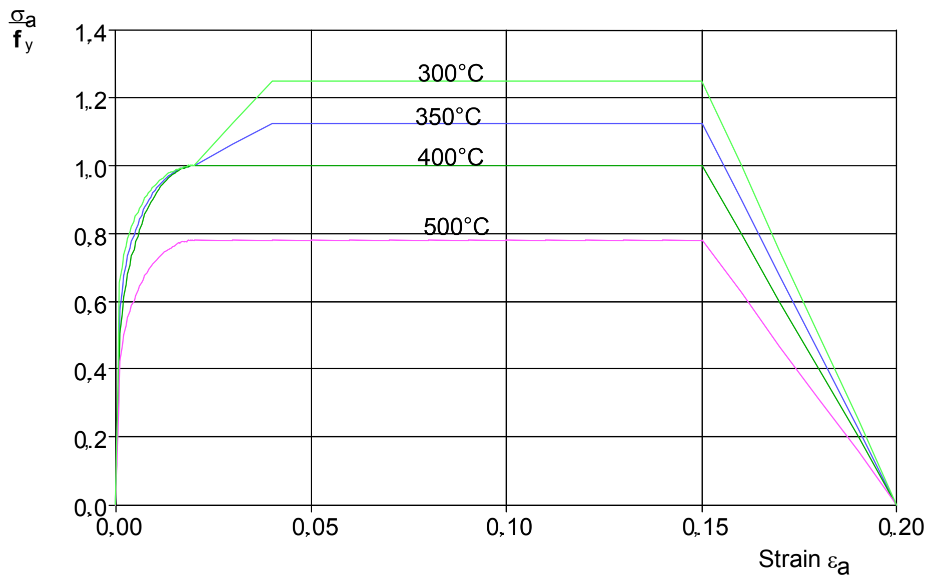


Figure A.2: Alternative stress-strain relationships for steel at elevated temperatures, allowing for strain hardening

Annex B [normative] Heat transfer to external steelwork

B.1 General

B.1.1 Basis

ST

(1) In this annex B, the fire compartment is assumed to be confined to one storey only. All windows or other similar openings in the fire compartment are assumed to be rectangular.

RC

(2) The determination of the temperature of the compartment fire, the dimensions and temperatures of the flames projecting from the openings, and the radiation and convection parameters should be performed according to annex B of EN 1991-1-2.

RC

(3) A distinction should be made between members not engulfed in flame and members engulfed in flame, depending on their locations relative to the openings in the walls of the fire compartment.

RC

(4) A member that is not engulfed in flame should be assumed to receive radiative heat transfer from all the openings in that side of the fire compartment and from the flames projecting from all these openings.

RC

(5) A member that is engulfed in flame should be assumed to receive convective heat transfer from the engulfing flame, plus radiative heat transfer from the engulfing flame and from the fire compartment opening from which it projects. The radiative heat transfer from other flames and from other openings may be neglected.

B.1.2 Conventions for dimensions

PE

(1) The convention for geometrical data may be taken from figure B.1.

B.1.3 Heat balance

RC

(1) For a member not engulfed in flame, the average temperature of the steel member T_m [K] should be determined from the solution of the following heat balance:

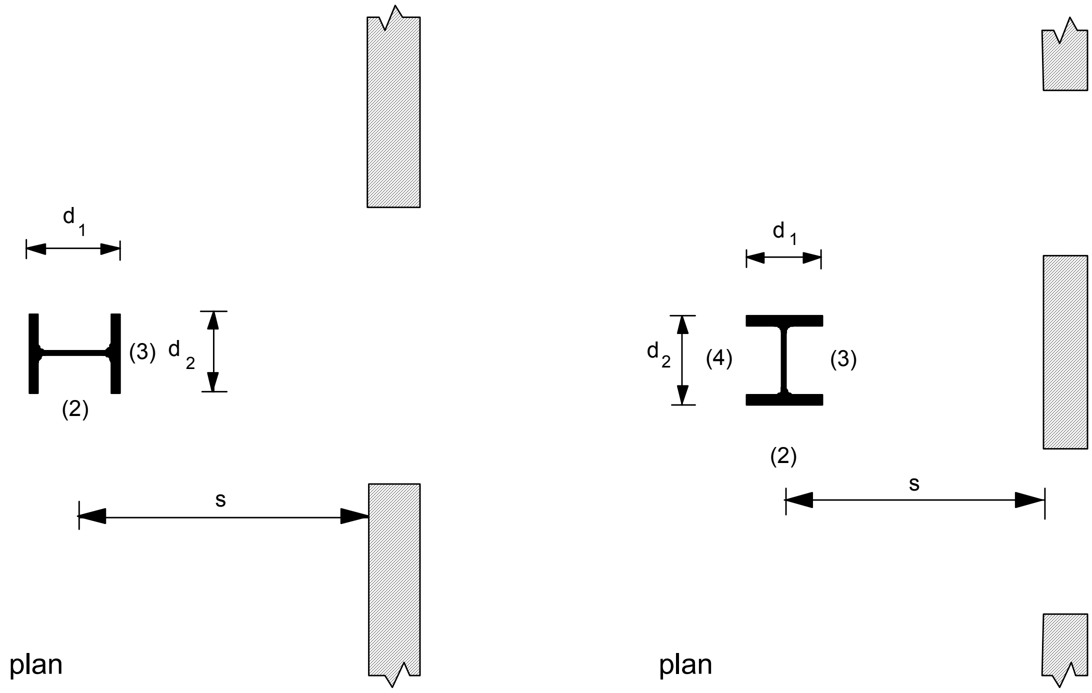
$$\sigma T_m^4 + \alpha T_m = \Sigma I_z + \Sigma I_f + 293\alpha \qquad \text{(B.1)}$$

where:

σ	is	the Stefan Boltzmann constant [$56,7 \times 10^{-12}$ kW/m^{2}K^4];
α	is	the convective heat transfer coefficient [kW/m^{2}K];
I_z	is	the radiative heat flux from a flame [kW/m^2];
I_f	is	the radiative heat flux from an opening [kW/m^2].

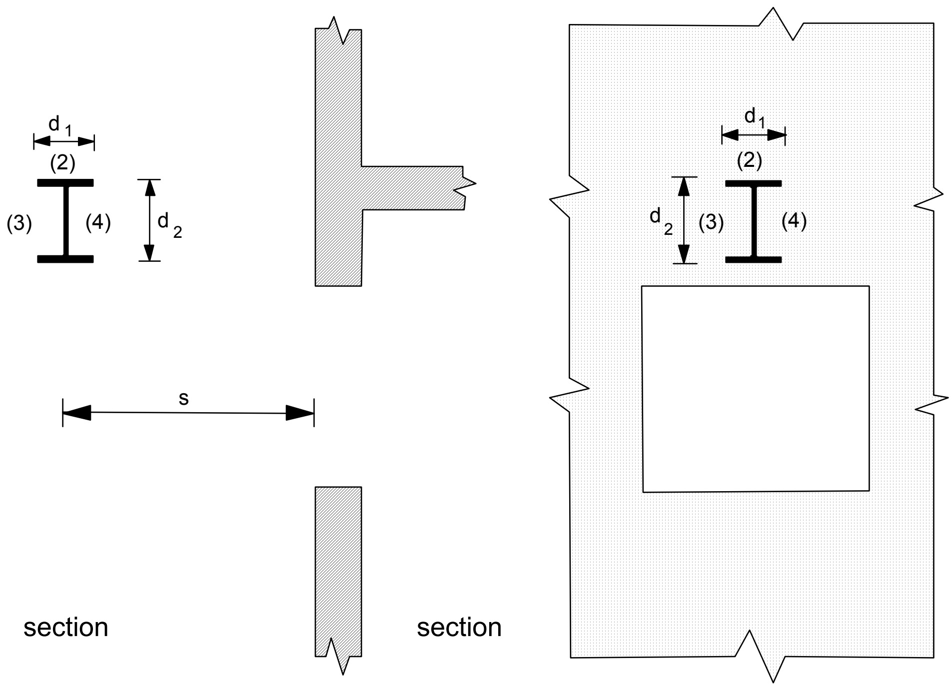
RC

(2) The convective heat transfer coefficient α should be obtained from annex B of EN 1991-1-2 for the `no forced draught' or the `forced draught' condition as appropriate, using an effective cross-sectional dimension $d = (d_1 + d_2)/2$.



1) Column opposite opening 2) Column between openings

a) Columns



1) Beam parallel to wall 2) Beam perpendicular to wall

b) Beams

Figure B.1: Member dimensions and faces

RC (3) For a member engulfed in flame, the average temperature of the steel member T_m [K] should be determined from the solution of the following heat balance:

$$\sigma T_m^4 + \alpha T_m = I_z + I_f + \alpha T_z \tag{B.2}$$

where:

T_z	is	the flame temperature [K];
I_z	is	the radiative heat flux from the flame [kW/m^2];
I_f	is	the radiative heat flux from the corresponding opening [kW/m^2].

RC (4) The radiative heat flux I_z from flames should be determined according to the situation and type of member as follows:

- Columns not engulfed in flame: see B.2;
- Beams not engulfed in flame: see B.3;
- Columns engulfed in flame: see B.4;
- Beams fully or partially engulfed in flame: see B.5.

Other cases may be treated analogously, using appropriate adaptations of the treatments given in B.2 to B.5.

RC (5) The radiative heat flux I_f from an opening should be determined from:

$$I_f = \phi_f \varepsilon_f (1 - a_z) \sigma T_f^4 \tag{B.3}$$

where:

ϕ_f	is	the overall configuration factor of the member for radiative heat transfer from that opening;
ε_f	is	the emissivity of the opening;
a_z	is	the absorptivity of the flames;
T_f	is	the temperature of the fire [K] from annex B of EN 1991-1-2.

RC (6) The emissivity ε_f of an opening should be taken as unity, see annex B of EN 1991-1-2.

RC (7) The absorptivity a_z of the flames should be determined from B.2 to B.5 as appropriate.

B.1.4 Overall configuration factors

RC

(1) The overall configuration factor ϕ_f of a member for radiative heat transfer from an opening should be determined from:

$$\phi_f = \frac{(C_1\varphi_{f,1} + C_2\varphi_{f,2})d_1 + (C_3\varphi_{f,3} + C_4\varphi_{f,4})d_2}{(C_1 + C_2)d_1 + (C_3 + C_4)d_2} \tag{B.4}$$

where:

$\phi_{f,i}$	is	the configuration factor of member face i for that opening, see annex G of EN 1991-1-2;
d_i	is	the cross-sectional dimension of member face i ;
C_i	is	the protection coefficient of member face i as follows:
		- for a protected face: $C_i = 0$
		- for an unprotected face: $C_i = 1$

RC

(2) The configuration factor $\phi_{f,i}$ for a member face from which the opening is not visible should be taken as zero.

RC

(3) The overall configuration factor ϕ_z of a member for radiative heat transfer from a flame should be determined from:

$$\phi_z = \frac{(C_1\varphi_{z,1} + C_2\varphi_{z,2})d_1 + (C_3\varphi_{z,3} + C_4\varphi_{z,4})d_2}{(C_1 + C_2)d_1 + (C_3 + C_4)d_2} \tag{B.5}$$

where:

$\phi_{z,i}$	is	the configuration factor of member face i for that flame, see annex G of EN 1991-1-2.

RC

(4) The configuration factors $\phi_{z,i}$ of individual member faces for radiative heat transfer from flames may be based on equivalent rectangular flame dimensions. The dimensions and locations of equivalent rectangles representing the front and sides of a flame for this purpose should be determined as given in B.2 for columns and B.3 for beams. For all other purposes, the flame dimensions from annex B of EN 1991-1-2 should be used.

RC

(5) The configuration factor $\phi_{z,i}$ for a member face from which the flame is not visible should be taken as zero.

PE

(6) A member face may be protected by a heat screen, see 4.2.5.4. A member face that is immediately adjacent to the compartment wall may also be treated as protected, provided that there are no openings in that part of the wall. All other member faces should be treated as unprotected.

B.2 Column not engulfed in flame

B.2.1 Radiative heat transfer

RC (1) A distinction should be made between a column located opposite an opening and a column located between openings.

NOTE: Illustration are given in figure B.2

RC (2) If the column is opposite an opening the radiative heat flux I_z from the flame should be determined from:

$$I_z = \phi_z \varepsilon_z \sigma T_z^4 \tag{B.6}$$

where:

ϕ_z	is	the overall configuration factor of the column for heat from the flame, see B.1.4;
ε_z	is	the emissivity of the flame, see B.2.2;
T_z	is	the flame temperature [K] from B.2.3.

NOTE: Illustration are given in figure B.3.

RC (3) If the column is between openings the total radiative heat flux I_z from the flames on each side should be determined from:

$$I_z = (\phi_{z,m} \varepsilon_{z,m} + \phi_{z,n} \varepsilon_{z,n}) \sigma T_z^4 \tag{B.7}$$

where:

$\phi_{z,m}$	is	the overall configuration factor of the column for heat from flames on side m, see B.1.4;
$\phi_{z,n}$	is	the overall configuration factor of the column for heat from flames on side n, see B.1.4;
$\varepsilon_{z,m}$	is	the total emissivity of the flames on side m, see B.2.2;
$\varepsilon_{z,n}$	is	the total emissivity of the flames on side n, see B.2.2.

NOTE: Illustration are given in figure B.4.

B.2.2 Flame emissivity

RC (1) If the column is opposite an opening, the flame emissivity ε_z should be determined from the expression for ε given in annex B of EN 1991-1-2, using the flame thickness λ at the level of the top of the openings. Provided that there is no awning or balcony above the opening λ may be taken as follows:

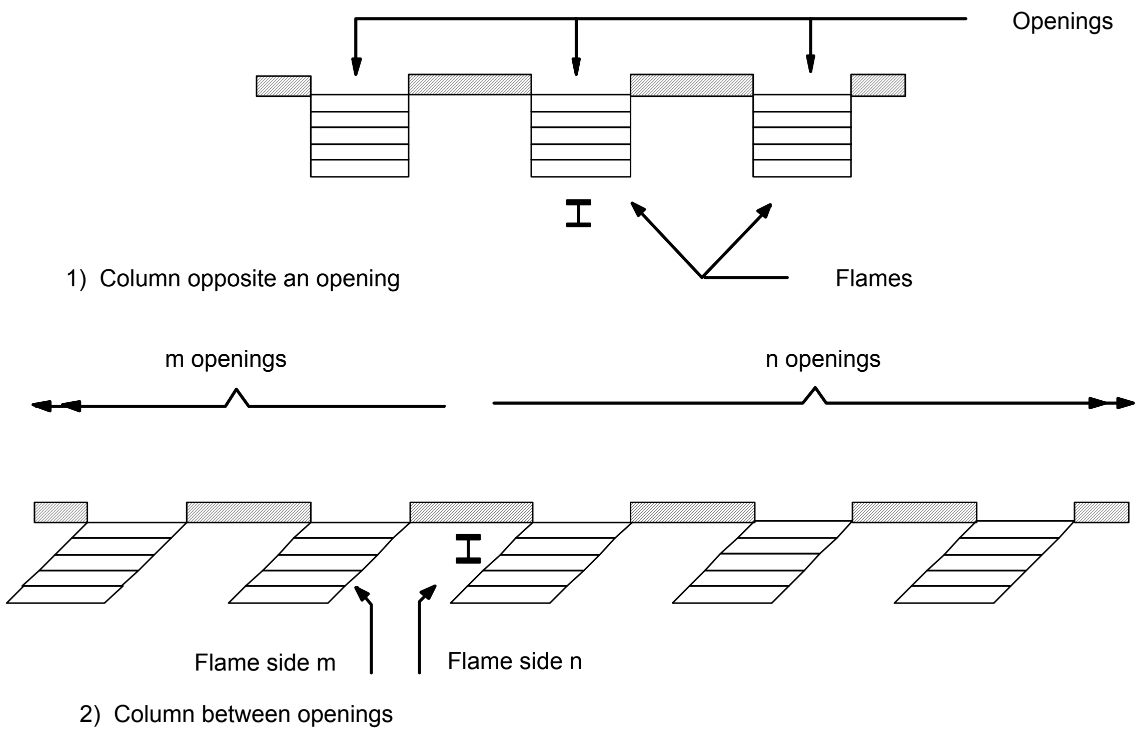
- for the `no forced draught' condition:

$$\lambda = 2h/3 \tag{B.8a}$$

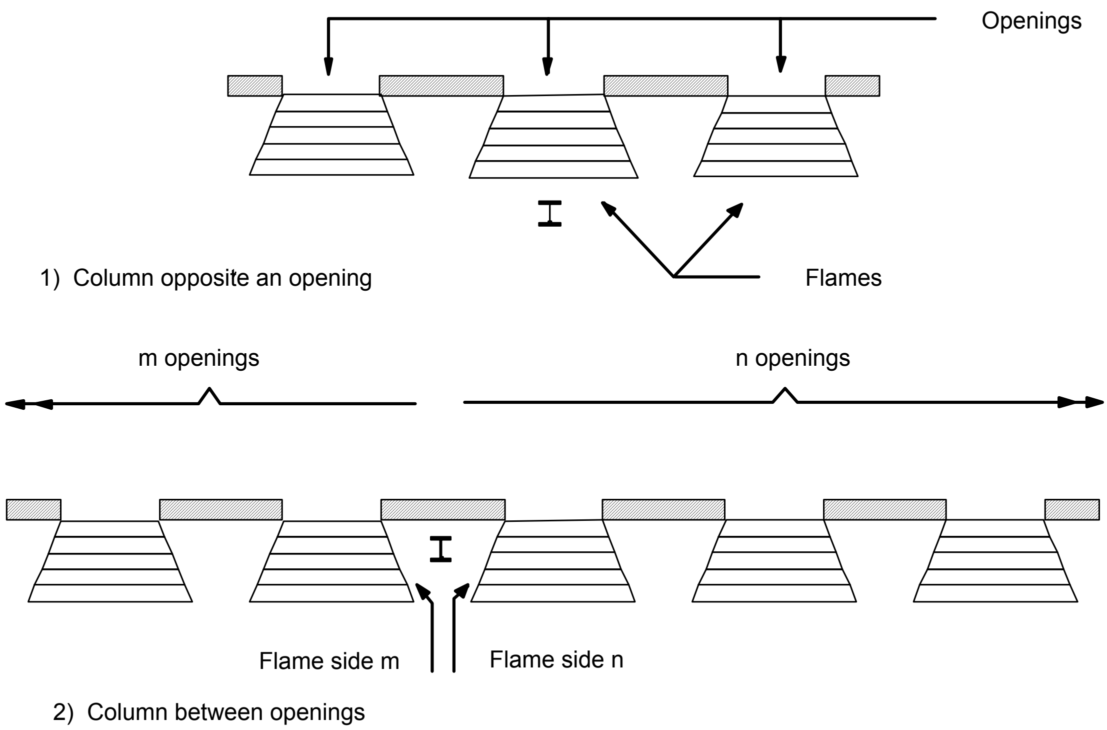
- for the `forced draught' condition:

$$\lambda = x \text{ but } \lambda \leq hx/z \tag{B.8b}$$

where h, x and z are as given in annex B of EN 1991-1-2.

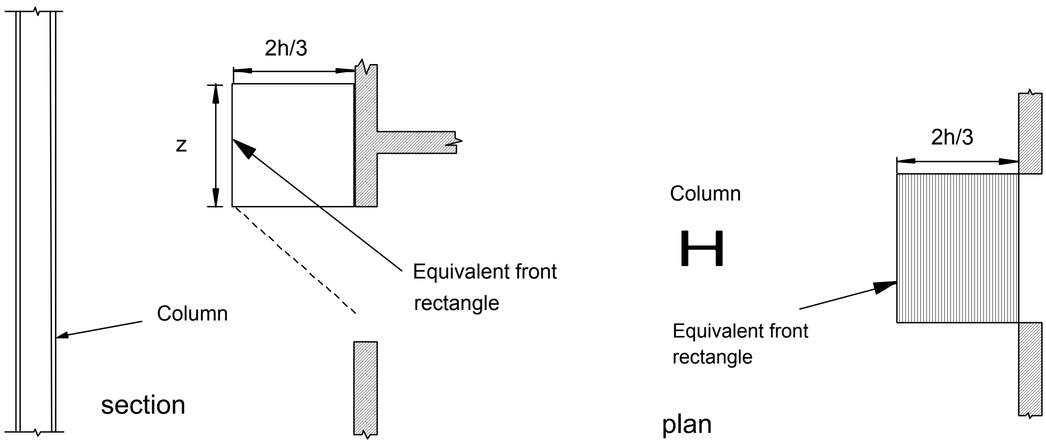


a) `No forced draught' condition

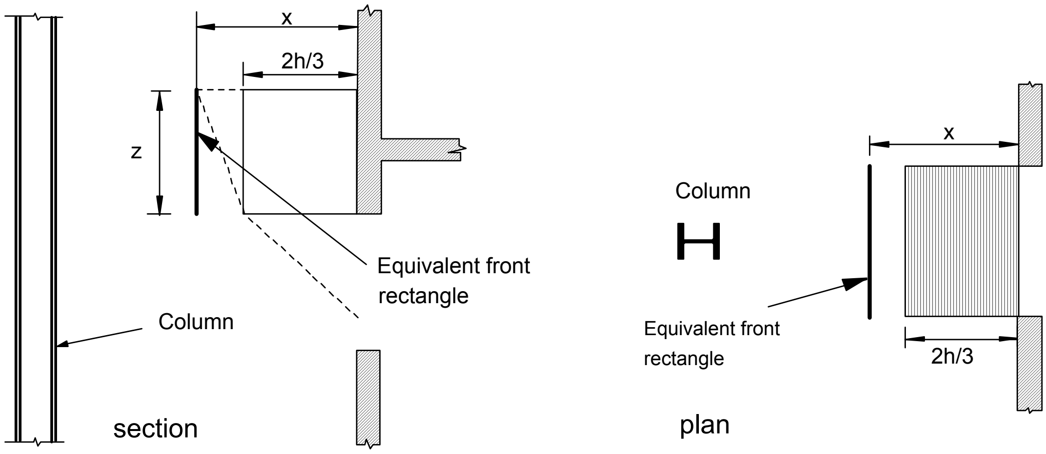


b) `Forced draught' condition

Figure B.2: Column positions

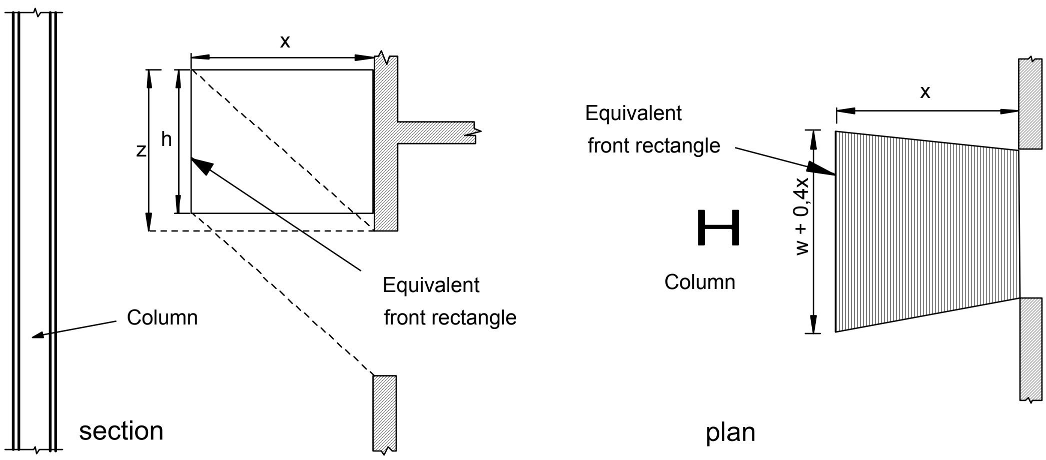


1) wall above and h < 1,25w



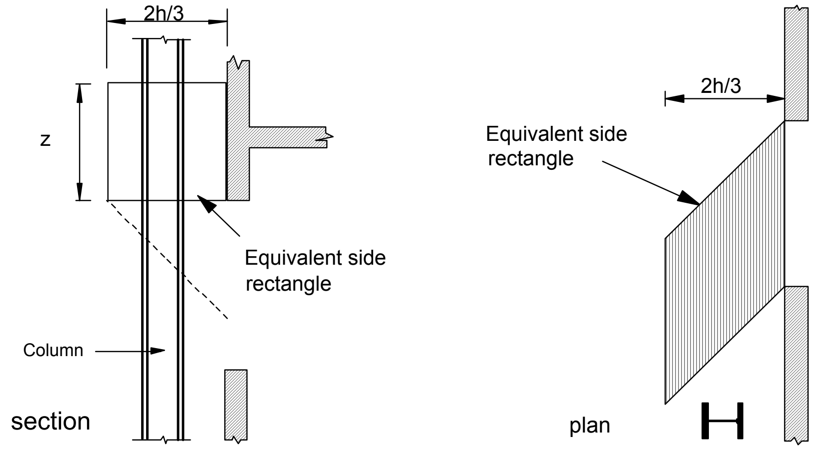
2) wall above and h > 1,25w or no wall above

a) `No forced draught'

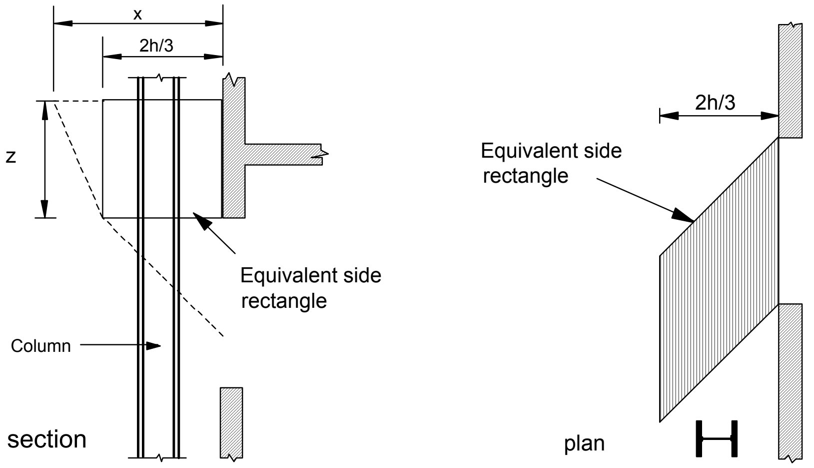


b) **`Forced draught'**

Figure B.3: Column opposite opening

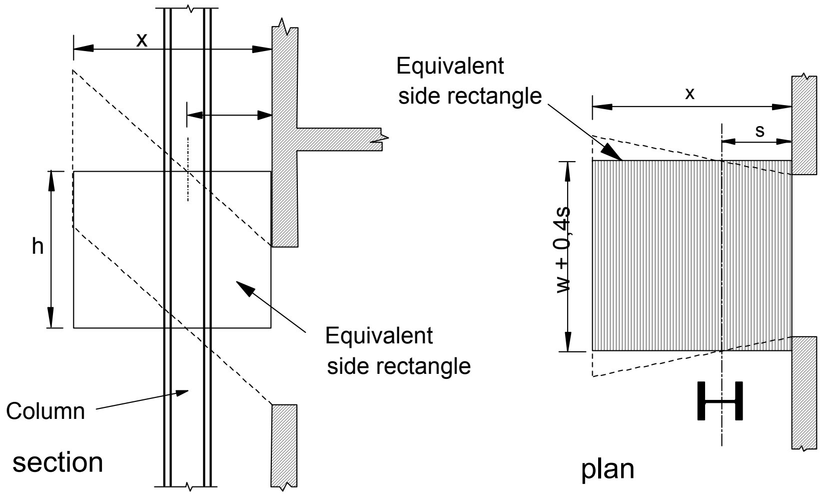


1) wall above and h < 1,25w



2) wall above and h > 1,25w or no wall above

a) `No forced draught'



b) `Forced draught'

Figure B.4: Column between openings

RC (2) If the column is between two openings, the total emissivities $\varepsilon_{z,m}$ and $\varepsilon_{z,n}$ of the flames on sides m and n should be determined from the expression for ε given in annex B of EN 1991-1-2 using a value for the total flame thickness λ as follows:

- for side m: $\lambda = \sum_{i=1}^{m} \lambda_i$ (B.9a)

- for side n: $\lambda = \sum_{i=1}^{n} \lambda_i$ (B.9b)

where:

m is the number of openings on side m;

n is the number of openings on side n;

λ_i is the flame thickness for opening i.

RC (3) The flame thickness λ_i should be taken as follows:

- for the `no forced draught' condition:

$$\lambda_i = w_i \qquad \text{(B.10a)}$$

- for the `forced draught' condition:

$$\lambda_i = w_i + 0{,}4s \qquad \text{(B.10b)}$$

where:

w_i is the width of the opening;

s is the horizontal distance from the centreline of the column to the wall of the fire compartment, see figure B.1.

B.2.3 Flame temperature

RC (1) The flame temperature T_z should be taken as the temperature at the flame axis obtained from the expression for T_z given in annex B of EN 1991-1-2, for the `no forced draught' condition or the `forced draught' condition as appropriate, at a distance l from the opening, measured along the flame axis, as follows:

- for the `no forced draught' condition:

$$l = h/2 \qquad \text{(B.11a)}$$

- for the `forced draught' condition:
  - for a column opposite an opening:

$$l = 0 \qquad \text{(B.11b)}$$

  - for a column between openings l is the distance along the flame axis to a point at a horizontal distance s from the wall of the fire compartment. Provided that there is no awning or balcony above the opening:

$$l = sX/x \qquad \text{(B.11c)}$$

where X and x are as given in annex B of EN 1991-1-2.

B.2.4 Flame absorptivity

RC (1) For the `no forced draught' condition, the flame absorptivity a_z should be taken as zero.

RC (2) For the `forced draught' condition, the flame absorptivity a_z should be taken as equal to the emissivity ε_z of the relevant flame, see B.2.2.

B.3 Beam not engulfed in flame

B.3.1 Radiative heat transfer

ST (1) Throughout B.3 it is assumed that the level of the bottom of the beam is not below the level of the top of the openings in the fire compartment.

RC (2) A distinction should be made between a beam that is parallel to the external wall of the fire compartment and a beam that is perpendicular to the external wall of the fire compartment, see figure B.5.

RC (3) If the beam is parallel to the external wall of the fire compartment, the average temperature of the steel member T_m should be determined for a point in the length of the beam directly above the centre of the opening. For this case the radiative heat flux I_z from the flame should be determined from:

$$I_z = \phi_z \varepsilon_z \sigma T_z^4 \tag{B.12}$$

where:

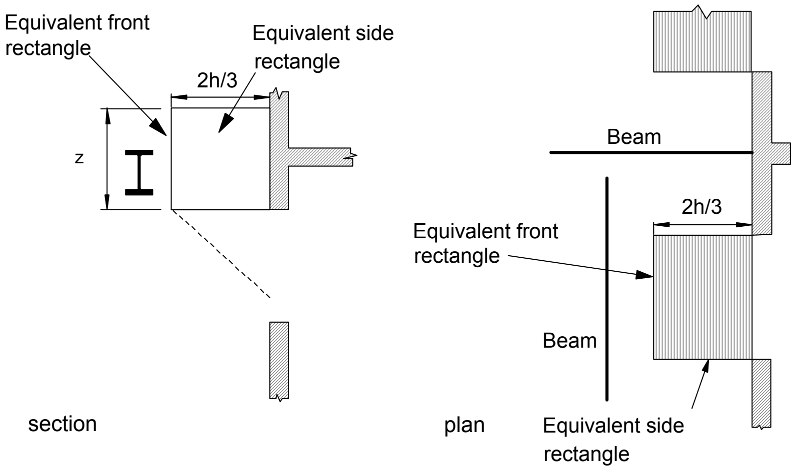
ϕ_z is the overall configuration factor for the flame directly opposite the beam, see B.1.4;
ε_z is the flame emissivity, see B.3.2;
T_z is the flame temperature from B.3.3 [K].

RC (4) If the beam is perpendicular to the external wall of the fire compartment, the average temperature in the beam should be determined at a series of points every 100 mm along the length of the beam. The average temperature of the steel member T_m should then be taken as the maximum of these values. For this case the radiative heat flux I_z from the flames should be determined from:

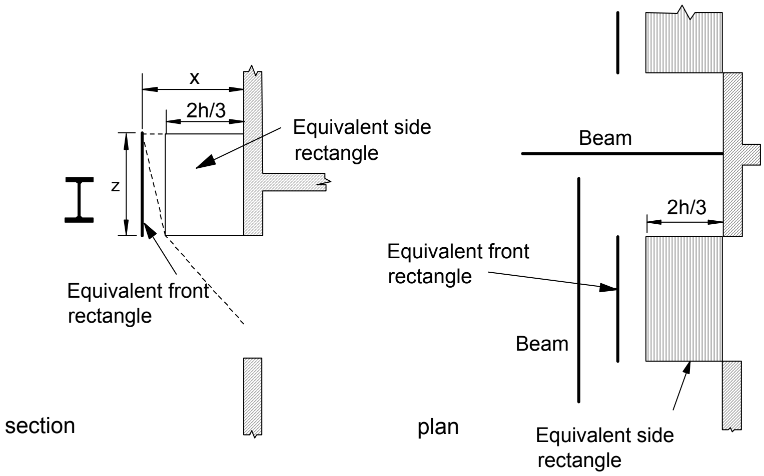
$$I_z = (\phi_{z,m} \varepsilon_{z,m} + \phi_{z,n} \varepsilon_{z,n}) \sigma T_z^4 \tag{B.13}$$

where:

$\phi_{z,m}$ is the overall configuration factor of the beam for heat from flames on side m, see B.3.2;
$\phi_{z,n}$ is the overall configuration factor of the beam for heat from flames on side n, see B.3.2;
$\varepsilon_{z,m}$ is the total emissivity of the flames on side m, see B.3.3;
$\varepsilon_{z,n}$ is the total emissivity of the flames on side n, see B.3.3;
T_z is the flame temperature [K], see B.3.4.

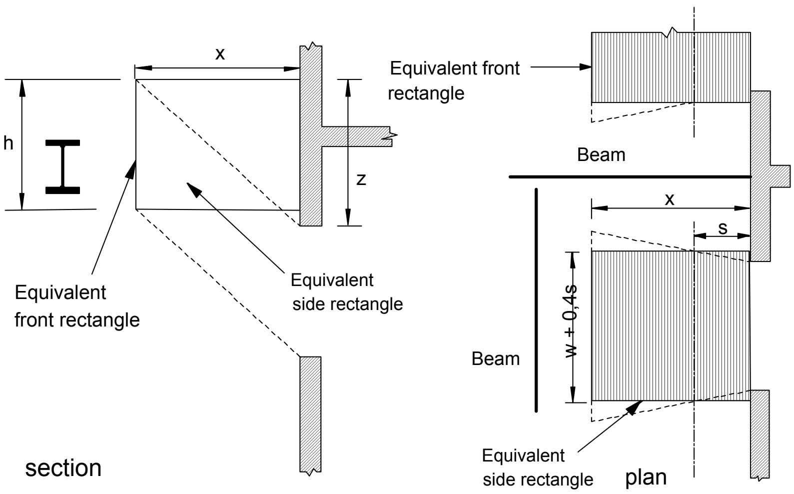


1) wall above and h < 1,25w



2) wall above and h > 1,25w or no wall above

a) `No forced draught'



b) `Forced draught'

Figure B.5: Beam not engulfed in flame

B.3.2 Flame emissivity

RC

(1) If the beam is parallel to the external wall of the fire compartment, above an opening, the flame emissivity ε_z should be determined from the expression for ε given in annex B of EN 1991-1-2, using a value for the flame thickness λ at the level of the top of the openings. Provided that there is no awning or balcony above the opening λ may be taken as follows:

- for the `no forced draught' condition:

$$\lambda = 2h/3 \tag{B.14a}$$

- for the `forced draught' condition:

$$\lambda = x \text{ but } \lambda \leq hx/z \tag{B.14b}$$

where h, x and z are as given in annex B of EN 1991-1-2

RC

(2) If the beam is perpendicular to the external wall of the fire compartment, between two openings, the total emissivities $\varepsilon_{z,m}$ and $\varepsilon_{z,n}$ of the flames on sides m and n should be determined from the expression for ε given in annex B of EN 1991-1-2 using a value for the flame thickness λ as follows:

- for side m: $\lambda = \sum_{i=1}^{m} \lambda_i$ (B.15a)

- for side n: $\lambda = \sum_{i=1}^{n} \lambda_i$ (B.15b)

where:

m	is	the number of openings on side m;
n	is	the number of openings on side n;
λ_i	is	the width of opening i.

RC

(3) The flame thickness λ_i should be taken as follows:

- for the `no forced draught' condition:

$$\lambda_i = w_i \tag{B.16a}$$

- for the `forced draught' condition:

$$\lambda_i = w_i + 0,4s \tag{B.16b}$$

where:

w_i	is	the width of the opening;
s	is	the horizontal distance from the wall of the fire compartment to the point under consideration on the beam, see figure B.5.

B.3.3 Flame temperature

RC

(1) The flame temperature T_z should be taken as the temperature at the flame axis obtained from the expression for T_z given in annex B of EN 1991-1-2, for the `no forced draught' or `forced draught' condition as appropriate, at a distance l from the opening, measured along the flame axis, as follows:

- for the `no forced draught' condition:

$$l = h/2 \qquad \text{(B.17a)}$$

- for the `forced draught' condition:
  - for a beam parallel to the external wall of the fire compartment, above an opening:

$$l = 0 \qquad \text{(B.17b)}$$

  - for a beam perpendicular to the external wall of the fire compartment, between openings l is the distance along the flame axis to a point at a horizontal distance s from the wall of the fire compartment. Provided that there is no awning or balcony above the opening:

$$l = sX/x \qquad \text{(B.17c)}$$

where X and x are as given in annex B of EN 1991-1-2.

B.3.4 Flame absorptivity

RC

(1) For the `no forced draught' condition, the flame absorptivity a_z should be taken as zero.

RC

(2) For the `forced draught' condition, the flame absorptivity a_z should be taken as equal to the emissivity ε_z of the relevant flame, see B.3.2.

B.4 Column engulfed in flame

RC

(1) The radiative heat flux I_z from the flames should be determined from:

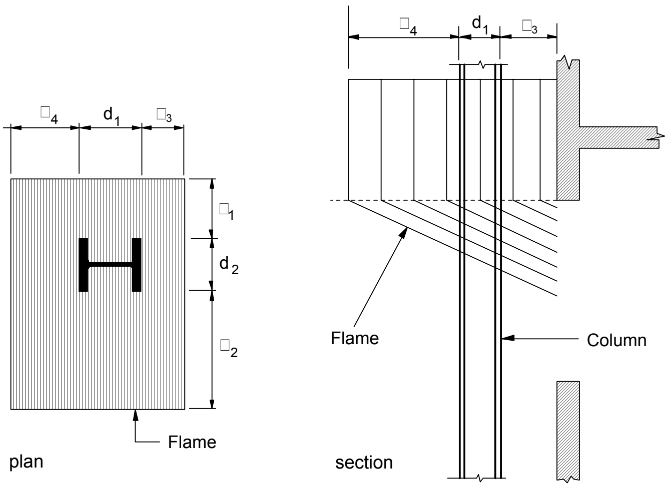
$$I_z = \frac{(I_{z,1} + I_{z,2})d_1 + (I_{z,3} + I_{z,4})d_2}{2(d_1 + d_2)} \qquad \text{(B.18)}$$

with:

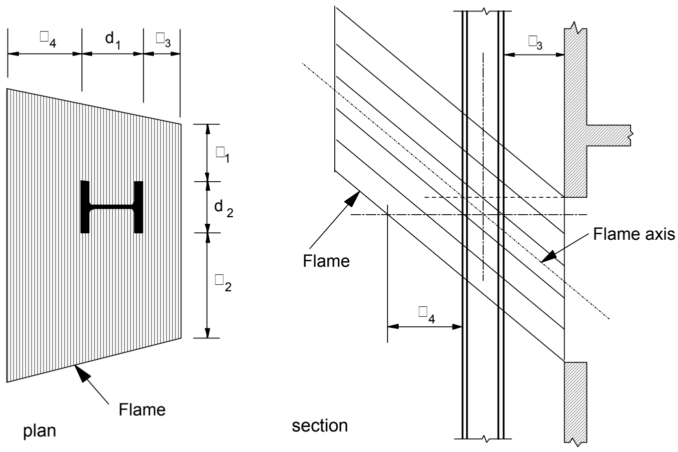
$$I_{z,1} = C_1 \varepsilon_{z,1} \sigma T_z^4$$
$$I_{z,2} = C_2 \varepsilon_{z,2} \sigma T_z^4$$
$$I_{z,3} = C_3 \varepsilon_{z,3} \sigma T_o^4$$
$$I_{z,4} = C_4 \varepsilon_{z,4} \sigma T_z^4$$

where:

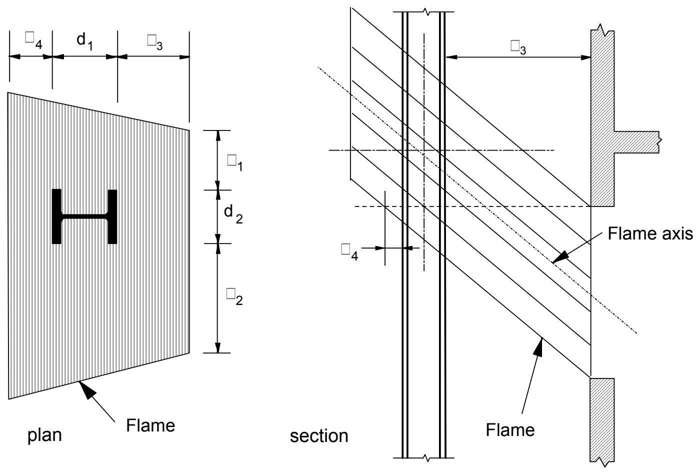
$I_{z,i}$	is	the radiative heat flux from the flame to column face i;
$\varepsilon_{z,i}$	is	the emissivity of the flames with respect to face i of the column;
i	is	the column face indicator (1), (2), (3) or (4);
C_i	is	the protection coefficient of member face i, see B.1.4;
T_z	is	the flame temperature [K];
T_o	is	the flame temperature at the opening [K] from annex B of EN 1991-1-2.



a) `No forced draught' condition



1) Flame axis intersects column axis below top of opening



2) Flame axis intersects column axis above top of opening

b) 'Forced draught' condition

Figure B.6: Column engulfed in flame

RC (2) The emissivity of the flames $\varepsilon_{z,i}$ for each of the faces 1, 2, 3 and 4 of the column should be determined from the expression for ε given in annex B of EN 1991-1-2, using a flame thickness λ equal to the dimension λ_i indicated in figure B.6 corresponding to face i of the column.

RC (3) For the `no forced draught' condition the values of λ_i at the level of the top of the opening should be used, see figure B.6(a).

RC (4) For the `forced draught' condition, if the level of the intersection of the flame axis and the column centreline is below the level of the top of the opening, the values of λ_i at the level of the intersection should be used, see figure B.6(b)(1). Otherwise the values of λ_i at the level of the top of the opening should be used, see figure B.6(b)(2), except that if $\lambda_4 < 0$ at this level, the values at the level where $\lambda_4 = 0$ should be used.

RC (5) The flame temperature T_z should be taken as the temperature at the flame axis obtained from the expression for T_z given in annex B of EN 1991-1-2 for the `no forced draught' or `forced draught' condition as appropriate, at a distance l from the opening, measured along the flame axis, as follows:

- for the `no forced draught' condition:

$$l = h/2 \tag{B.19a}$$

- for the `forced draught' condition, l is the distance along the flame axis to the level where λ_i is measured. Provided that there is no balcony or awning above the opening:

$$l = (\lambda_3 + 0{,}5 d_1)X/x \text{ but } l \leq 0{,}5 hX/z \tag{B.19b}$$

where h, X, x and z are as given in annex B of EN 1991-1-2.

RC (6) The absorptivity a_z of the flames should be determined from:

$$a_z = \frac{\varepsilon_{z,1} + \varepsilon_{z,2} + \varepsilon_{z,3}}{3} \tag{B.20}$$

where $\varepsilon_{z,1}$, $\varepsilon_{z,2}$ and $\varepsilon_{z,3}$ are the emissivities of the flame for column faces 1, 2, and 3.

B.5 Beam fully or partially engulfed in flame

B.5.1 Radiative heat transfer

B.5.1.1 General

ST (1) Throughout B.5 it is assumed that the level of the bottom of the beam is not below the level of the top of the adjacent openings in the fire compartment.

RC (2) A distinction should be made between a beam that is parallel to the external wall of the fire compartment and a beam that is perpendicular to the external wall of the fire compartment, see figure B.7.

RC (3) If the beam is parallel to the external wall of the fire compartment, its average temperature T_m should be determined for a point in the length of the beam directly above the centre of the opening.

RC (4) If the beam is perpendicular to the external wall of the fire compartment, the value of the average temperature should be determined at a series of points every 100 mm along the length of the beam. The maximum of these values should then be adopted as the average temperature of the steel member T_m.

RC (5) The radiative heat flux I_z from the flame should be determined from:

$$I_z = \frac{(I_{z1} + I_{z2})\, d_1 + (I_{z3} + I_{z4})\, d_2}{2\,(d_1 + d_2)} \tag{B.21}$$

where:

$I_{z,i}$	is	the radiative heat flux from the flame to beam face i;
i	is	the beam face indicator (1), (2), (3) or (4).

B.5.1.2 `No forced draught' condition

RC (1) For the `no forced draught' condition, a distinction should be made between those cases where the top of the flame is above the level of the top of the beam and those where it is below this level.

RC (2) If the top of the flame is above the level of the top of the beam the following equations should be applied:

$$I_{z,1} = C_1\, \varepsilon_{z,1}\, \sigma T_o^4 \tag{B.22a}$$

$$I_{z,2} = C_2\, \varepsilon_{z,2}\, \sigma T_{z,2}^4 \tag{B.22b}$$

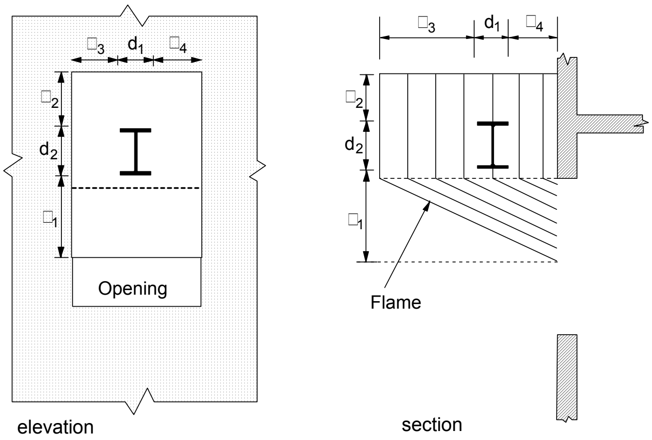
$$I_{z,3} = C_3\, \varepsilon_{z,3}\, \sigma (T_{z,1}^4 + T_{z,2}^4)/2 \tag{B.22c}$$

$$I_{z,4} = C_4\, \varepsilon_{z,4}\, \sigma (T_{z,1}^4 + T_{z,2}^4)/2 \tag{B.22d}$$

where:

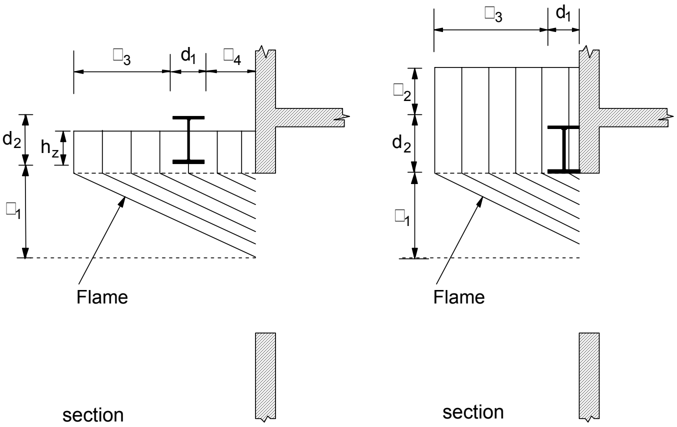
$\varepsilon_{z,i}$	is	the emissivity of the flame with respect to face i of the beam, see B.5.2;
T_o	is	the temperature at the opening [K] from annex B of EN 1991-1-2;
$T_{z,1}$	is	the flame temperature [K] from annex B of EN 1991-1-2, level with the bottom of the beam;
$T_{z,2}$	is	the flame temperature [K] from annex B of EN 1991-1-2, level with the top of the beam.

PE (3) In the case of a beam parallel to the external wall of the fire compartment C_4 may be taken as zero if the beam is immediately adjacent to the wall, see figure B.7.



1) Beam perpendicular to wall

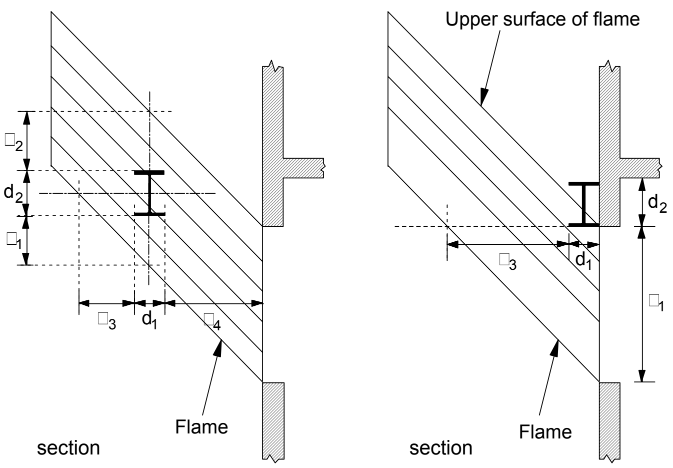
2) Beam parallel to wall



3) Top of flame below top of beam

4) Beam immediately adjacent to wall

a) `No forced draught' condition



1) Beam not adjacent to wall

2) Beam immediately adjacent to wall

b) `Forced draught' condition

Figure B.7: Beam engulfed in flame

RC (4) If the top of the flame is below the level of the top of the beam the following equations should be applied:

$$I_{z,1} = C_1 \varepsilon_{z,1} \sigma T_o^4 \tag{B.23a}$$

$$I_{z,2} = 0 \tag{B.23b}$$

$$I_{z,3} = (h_z/d_2) C_3 \varepsilon_{z,3} \sigma (T_{z,1}^4 + T_x^4)/2 \tag{B.23c}$$

$$I_{z,4} = (h_z/d_2) C_4 \varepsilon_{z,4} \sigma (T_{z,1}^4 + T_x^4)/2 \tag{B.23d}$$

where:

T_x	is	the flame temperature at the flame tip [813 K];
h_z	is	the height of the top of the flame above the bottom of the beam.

B.5.1.3 `Forced draught' condition

RC (1) For the `forced draught' condition, in the case of beams parallel to the external wall of the fire compartment a distinction should be made between those immediately adjacent to the wall and those not immediately adjacent to it.

NOTE: Illustrations are given in figure B.7.

RC (2) For a beam parallel to the wall, but not immediately adjacent to it, or for a beam perpendicular to the wall the following equations should be applied:

$$I_{z,1} = C_1 \varepsilon_{z,1} \sigma T_o^4 \tag{B.24a}$$

$$I_{z,2} = C_2 \varepsilon_{z,2} \sigma T_{z,2}^4 \tag{B.24b}$$

$$I_{z,3} = C_3 \varepsilon_{z,3} \sigma (T_{z,1}^4 + T_{z,2}^4)/2 \tag{B.24c}$$

$$I_{z,4} = C_4 \varepsilon_{z,4} \sigma (T_{z,1}^4 + T_{z,2}^4)/2 \tag{B.24d}$$

RC (3) If the beam is parallel to the wall and immediately adjacent to it, only the bottom face should be taken as engulfed in flame but one side and the top should be taken as exposed to radiative heat transfer from the upper surface of the flame, see figure B.7(b)(2). Thus:

$$I_{z,1} = C_1 \varepsilon_{z,1} \sigma T_o^4 \tag{B.25a}$$

$$I_{z,2} = \phi_{z,2} C_2 \varepsilon_{z,2} \sigma T_{z,2}^4 \tag{B.25b}$$

$$I_{z,3} = \phi_{z,3} C_3 \varepsilon_{z,3} \sigma (T_{z,1}^4 + T_{z,2}^4)/2 \tag{B.25c}$$

$$I_{z,4} = 0 \tag{B.25d}$$

where $\phi_{z,i}$ is the configuration factor relative to the upper surface of the flame, for face i of the beam, from annex G of EN 1991-1-2.

B.5.2 Flame emissivity

RC (1) The emissivity of the flame ε_{zi} for each of the faces 1, 2, 3 and 4 of the beam should be determined from the expression for ε given in annex B of EN 1991-1-2, using a flame thickness λ equal to the dimension λ_i indicated in figure B.7 corresponding to face i of the beam.

B.5.3 Flame absorptivity

(1) The absorptivity of the flame a_z should be determined from:

$$a_z = 1 - e^{-0,3h} \tag{B.26}$$

Annex C [informative] Stainless steel

C.1 General

ST (1) The thermal and mechanical properties of following stainless are given in this annex: 1.4301, 1.4401, 1.4571, 1.4003 and 1.4462.

Note: For other stainless steels according to EN 1993-1-4 the mechanical properties given in 3.2 may be used. The thermal properties may be taken from this annex.

RC (2) The values of material properties given in this annex should be treated as characteristic.

RC (3) The mechanical properties of steel at 20 °C should be taken as those given in EN 1993-1-4 for normal temperature design.

C.2 Mechanical properties of steel

C.2.1 Strength and deformation properties

RC (1) For heating rates between 2 and 50 K/min, the strength and deformation properties of stainless steel at elevated temperatures should be obtained from the stress-strain relationship given in figure C.1.

NOTE: For the rules of this standard it is assumed that the heating rates fall within the specified limits.

RC (2) This relationship should be used to determine the resistances to tension, compression, moment or shear.

ST (3) Table C.1 gives reduction factors, relative to the appropriate value at 20 °C, for the stress-strain relationship of several stainless steels at elevated temperatures as follows:

- slope of linear elastic range, relative to slope at 20 °C: $k_{E,\theta} = E_{a,\theta}/E_a$
- proof strength, relative to yield strength at 20 °C: $k_{0.2p,\theta} = f_{0,2p,\theta}/f_y$
- tensile strength, relative to tensile strength at 20 °C: $k_{u,\theta} = f_{u,\theta}/f_u$

ST (4) For the use of simple calculation methods table C.1 gives the correction factor $k_{2\%,\theta}$ for the determination of the yield strength using:

$$k_{y,\theta} = f_{0,2p,\theta} + k_{2\%,\theta}\,(f_{u,\theta} - f_{0,2p,\theta}) \tag{C.1}$$

ST (5) For the use of advanced calculation methods table C.2 gives additional values for the stress-strain relationship of several stainless steels at elevated temperatures as follows:

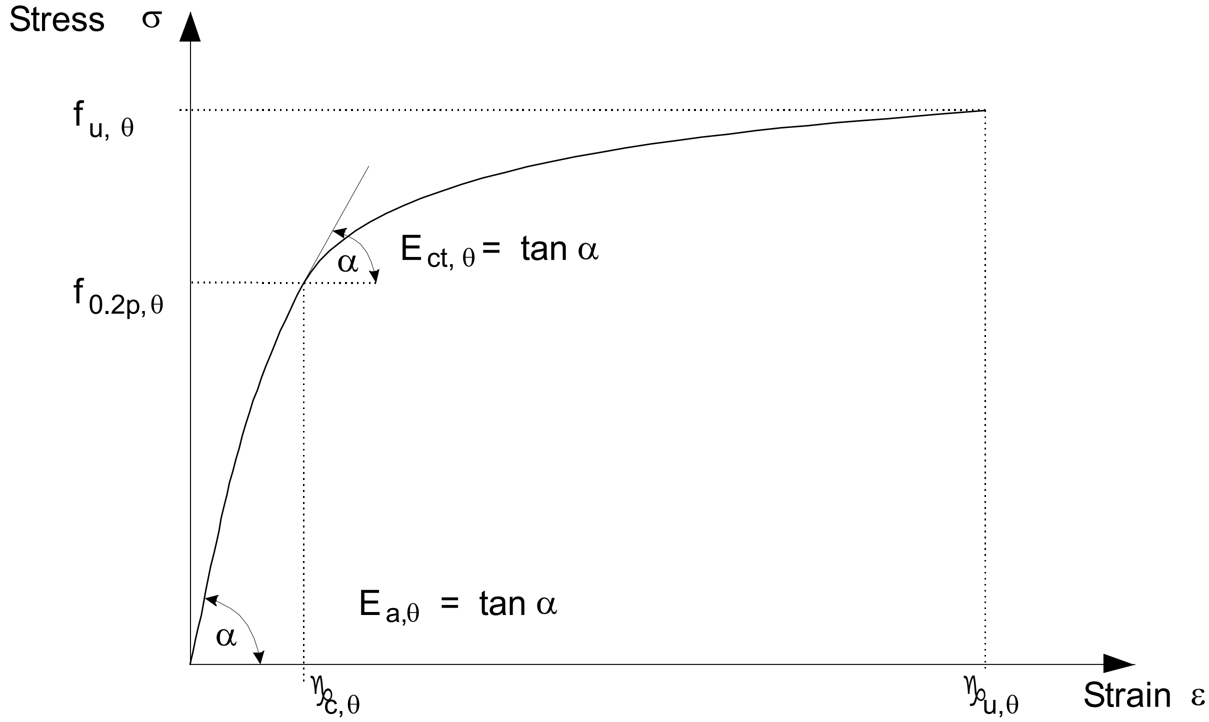
- slope at proof strength, relative to slope at 20 °C: $k_{Ect,\theta} = E_{ct,\theta}/E_a$
- ultimate strain: $\varepsilon_{u,\theta}$

C.2.2 Unit mass

PE (1) The unit mass of steel ρ_a may be considered to be independent of the steel temperature. The following value may be taken:

$$\rho_a = 7850\,\text{kg/m}^3$$

Strain range	Stress σ	Tangent modulus E_t
$\varepsilon \leq \varepsilon_{c,\theta}$	$\dfrac{E \cdot \varepsilon}{1 + a \cdot \varepsilon^b}$	$\dfrac{E(1 + a \cdot \varepsilon^b - a \cdot b \cdot \varepsilon^b)}{(1 + a \cdot \varepsilon^b)^2}$
$\varepsilon_{c,\theta} < \varepsilon \leq \varepsilon_{u,\theta}$	$f_{0.2p,\theta} - e + (d/c)\sqrt{c^2 - (\varepsilon_{u,\theta} - \varepsilon)^2}$	$\dfrac{d + (\varepsilon_{u,\theta} - \varepsilon)}{c\sqrt{c^2 - (\varepsilon_{u,\theta} - \varepsilon)^2}}$
Parameters	$\varepsilon_{c,\theta} = f_{0.2p,\theta}/E_{a,\theta} + 0.002$	
Functions	$a = \dfrac{E_{a,\theta}\,\varepsilon_{c,\theta} - f_{0.2p,\theta}}{f_{0.2p,\theta}\,\varepsilon_{c,\theta}^{\;b}}$ $c^2 = (\varepsilon_{u,\theta} - \varepsilon_{c,\theta})\left(\varepsilon_{u,\theta} - \varepsilon_{c,\theta} + \dfrac{e}{E_{ct,\theta}}\right)$ $e = \dfrac{(f_{u,\theta} - f_{0.2p,\theta})^2}{(\varepsilon_{u,\theta} - \varepsilon_{c,\theta})E_{ct,\theta} - 2(f_{u,\theta} - f_{0.2p,\theta})}$	$b = \dfrac{(1 - \varepsilon_{c,\theta}\,E_{ct,\theta}/f_{0.2p,\theta})\,E_{a,\theta}\,\varepsilon_{c,\theta}}{(E_{a,\theta}\,\varepsilon_{c,\theta}/f_{0.2p,\theta} - 1)\,f_{0.2p,\theta}}$ $d^2 = e\,(\varepsilon_{u,\theta} - \varepsilon_{c,\theta})\,E_{ct,\theta} + e^2$



Key:

$f_{u,\theta}$	is	tensile strength;
$f_{0.2p,\theta}$	is	the proof strength at 0.2% plastic strain;
$E_{a,\theta}$	is	the slope of the linear elastic range;
$E_{ct,\theta}$	is	the slope at proof strength;
$\varepsilon_{c,\theta}$	is	the total strain at proof strenght;
$\varepsilon_{u,\theta}$	is	the ultimate strain.

Figure C.1: Stress-strain relationship for stainless steel at elevated temperatures.

Table C.1: Factors for determination of strain and stiffness of stainless steel at elevated temperatures

Steel Temperature θ_a	Reduction factor (relative to E_a) for the slope of the linear elastic range $k_{E,\theta} = E_{a,\theta}/E_a$	Reduction factor (relative to f_y) for proof strength $k_{0.2p,\theta} = f_{0.2p,\theta}/f_y$	Reduction factor (relative to f_u) for tensile strength $k_{u,\theta} = f_{u,\theta}/f_u$	Factor for determination of the yield strength $f_{y,\theta}$ $k_{2\%,\theta}$
Grade 1.4301				
20	1,00	1,00	1,00	0,26
100	0,96	0,82	0,87	0,24
200	0,92	0,68	0,77	0,19
300	0,88	0,64	0,73	0,19
400	0,84	0,60	0,72	0,19
500	0,80	0,54	0,67	0,19
600	0,76	0,49	0,58	0,22
700	0,71	0,40	0,43	0,26
800	0,63	0,27	0,27	0,35
900	0,45	0,14	0,15	0,38
1000	0,20	0,06	0,07	0,40
1100	0,10	0,03	0,03	0,40
1200	0,00	0,00	0,00	0,40
Grade 1.4401 / 1.4404				
20	1,00	1,00	1,00	0,24
100	0,96	0,88	0,93	0,24
200	0,92	0,76	0,87	0,24
300	0,88	0,71	0,84	0,24
400	0,84	0,66	0,83	0,21
500	0,80	0,63	0,79	0,20
600	0,76	0,61	0,72	0,19
700	0,71	0,51	0,55	0,24
800	0,63	0,40	0,34	0,35
900	0,45	0,19	0,18	0,38
1000	0,20	0,10	0,09	0,40
1100	0,10	0,05	0,04	0,40
1200	0,00	0,00	0,00	0,40
Grade 1.4571				
20	1,00	1,00	1,00	0,25
100	0,96	0,89	0,88	0,25
200	0,92	0,83	0,81	0,25
300	0,88	0,77	0,80	0,24
400	0,84	0,72	0,80	0,22
500	0,80	0,69	0,77	0,21
600	0,76	0,66	0,71	0,21
700	0,71	0,59	0,57	0,25
800	0,63	0,50	0,38	0,35
900	0,45	0,28	0,22	0,38
1000	0,20	0,15	0,11	0,40
1100	0,10	0,075	0,055	0,40
1200	0,00	0,00	0,00	0,40

Continued

Table C.1 continued

Steel Temperature θ_a	Reduction factor (relative to E_a) for the slope of the linear elastic range $k_{E,\theta} = E_{a,\theta}/E_a$	Reduction factor (relative to f_y) for proof strength $k_{0.2p,\theta} = f_{0.2p,\theta}/f_y$	Reduction factor (relative to f_u) for tensile strength $k_{u,\theta} = f_{u,\theta}/f_u$	Factor for determination of the yield strength $f_{y,\theta}$ $k_{2\%,\theta}$
Grade 1.4003				
20	1,00	1,00	1,00	0,37
100	0,96	1,00	0,94	0,37
200	0,92	1,00	0,88	0,37
300	0,88	0,98	0,86	0,37
400	0,84	0,91	0,83	0,42
500	0,80	0,80	0,81	0,40
600	0,76	0,45	0,42	0,45
700	0,71	0,19	0,21	0,46
800	0,63	0,13	0,12	0,47
900	0,45	0,10	0,11	0,47
1000	0,20	0,07	0,09	0,47
1100	0,10	0,035	0,045	0,47
1200	0,00	0,00	0,00	0,47
Grade 1.4462				
20	1,00	1,00	1,00	0,35
100	0,96	0,91	0,93	0,35
200	0,92	0,80	0,85	0,32
300	0,88	0,75	0,83	0,30
400	0,84	0,72	0,82	0,28
500	0,80	0,65	0,71	0,30
600	0,76	0,56	0,57	0,33
700	0,71	0,37	0,38	0,40
800	0,63	0,26	0,29	0,41
900	0,45	0,10	0,12	0,45
1000	0,20	0,03	0,04	0,47
1100	0,10	0,015	0,02	0,47
1200	0,00	0,00	0,00	0,47

Table C.1: Reduction factor and ultimate strain for the use of advanced calculation methods

Steel Temperature θ_a	Reduction factor (relative to E_a) for the slope of the linear elastic range $k_{Ect,\theta} = E_{ct,\theta}/E_a$	Ultimate strain $\varepsilon_{u,\theta}$ [-]
Grade 1.4301		
20	0,11	0,40
100	0,05	0,40
200	0,02	0,40
300	0,02	0,40
400	0,02	0,40
500	0,02	0,40
600	0,02	0,35
700	0,02	0,30
800	0,02	0,20
900	0,02	0,20
1000	0,02	0,20
1100	0,02	0,20
1200	0,02	0,20
Grade 1.4401 / 1.4404		
20	0,050	0,40
100	0,049	0,40
200	0,047	0,40
300	0,045	0,40
400	0,030	0,40
500	0,025	0,40
600	0,020	0,40
700	0,020	0,30
800	0,020	0,20
900	0,020	0,20
1000	0,020	0,20
1100	0,020	0,20
1200	0,020	0,20
Grade 1.4571		
20	0,060	0,40
100	0,060	0,40
200	0,050	0,40
300	0,040	0,40
400	0,030	0,40
500	0,025	0,40
600	0,020	0,35
700	0,020	0,30
800	0,020	0,20
900	0,020	0,20
1000	0,020	0,20
1100	0,020	0,20
1200	0,020	0,20

Continued

Table C.1 continued

Steel Temperature θ_a	Reduction factor (relative to E_a) for the slope of the linear elastic range $k_{Ect,\theta} = E_{ct,\theta}/E_a$	Ultimate strain $\varepsilon_{u,\theta}$ [-]
Grade 1.4003		
20	0,055	0,20
100	0,030	0,20
200	0,030	0,20
300	0,030	0,20
400	0,030	0,15
500	0,030	0,15
600	0,030	0,15
700	0,030	0,15
800	0,030	0,15
900	0,030	0,15
1000	0,030	0,15
1100	0,030	0,15
1200	0,030	0,15
Grade 1.4462		
20	0,100	0,20
100	0,070	0,20
200	0,037	0,20
300	0,035	0,20
400	0,033	0,20
500	0,030	0,20
600	0,030	0,20
700	0,025	0,15
800	0,025	0,15
900	0,025	0,15
1000	0,025	0,15
1100	0,025	0,15
1200	0,025	0,15

C.3 Thermal properties

C.3.1 Thermal elongation

PE

(1) The thermal elongation of austenitic stainless steel $\Delta l/l$ may be determined from the following:

$$\Delta l/l = (16 + 4{,}79 \times 10^{-3}\,\theta_a - 1{,}243 \times 10^{-6}\,\theta_a^2) \times (\theta_a - 20)\,10^{-6} \quad \text{(C.1)}$$

where:

- l is the length at 20°C;
- Δl is the temperature induced expansion;
- θ_a is the steel temperature [°C].

NOTE: The variation of the thermal elongation with temperature is illustrated in figure C.2.

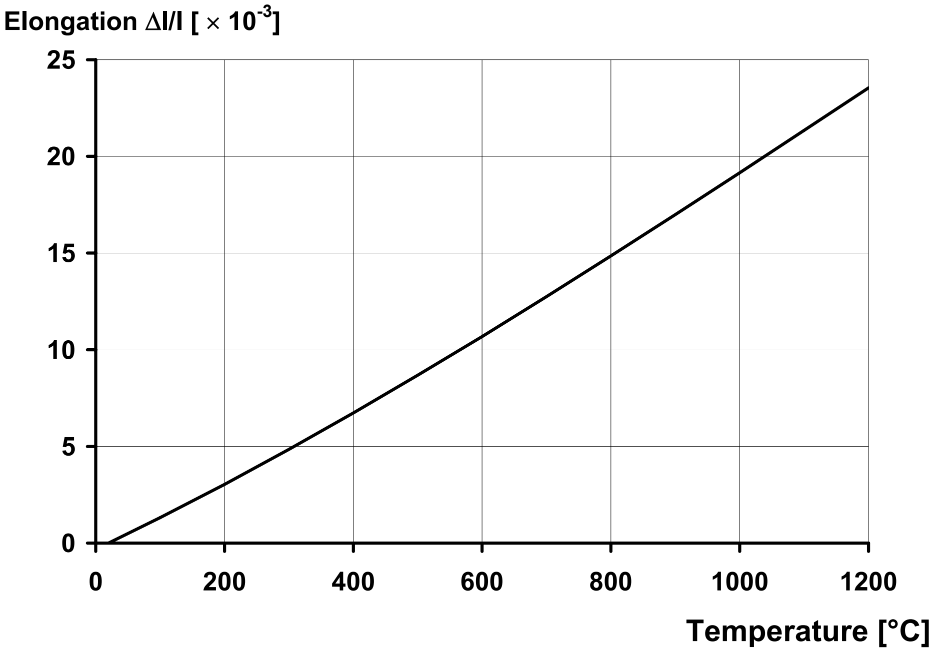


Figure C,2: Thermal elongation of stainless steel as a function of the temperature

C.3.2 Specific heat

PE

(1) The specific heat of stainless steel c_a may be determined from the following:

$$c_a = 450 + 0{,}280 \times \theta_a - 2{,}91 \times 10^{-4}\,\theta_a^2 + 1{,}34 \times 10^{-7}\,\theta_a^3 \text{ J/kgK} \quad \text{(C.2)}$$

where:

- θ_a is the steel temperature [°C].

NOTE: The variation of the specific heat with temperature is illustrated in figure C.3.

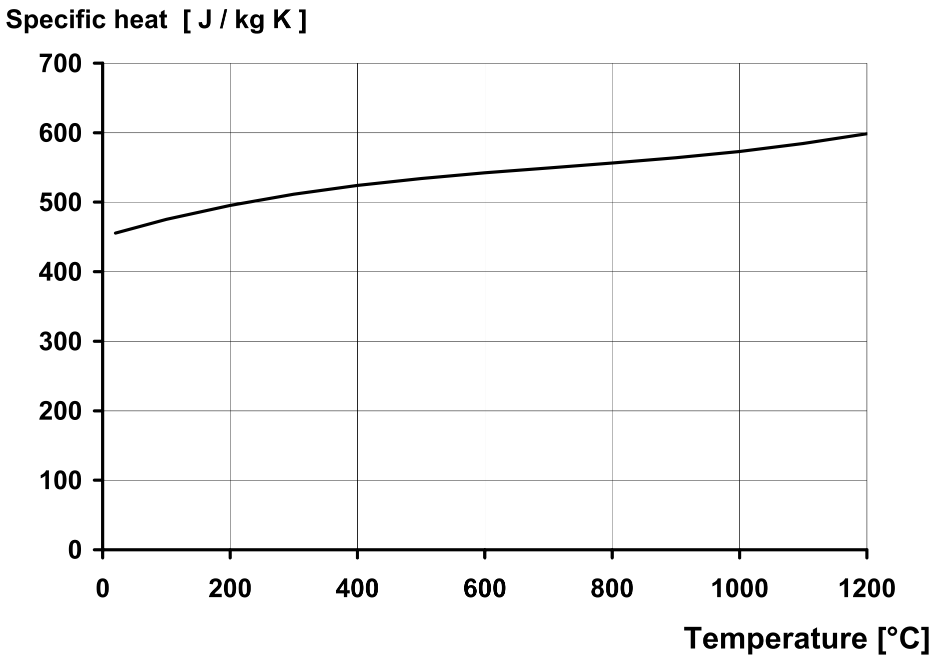


Figure C,3: Specific heat of stainless steel as a function of the temperature

C.3.3 Thermal conductivity

PE (1) The thermal conductivity of stainless steel λ_a may be determined from the following:

$$\lambda_a = 14,6 + 1,27 \times 10^{-2}\,\theta_a \text{ W/mK} \tag{C.3}$$

where:

θ_a is the steel temperature [°C].

NOTE: The variation of the thermal conductivity with temperature is illustrated in figure C.4.

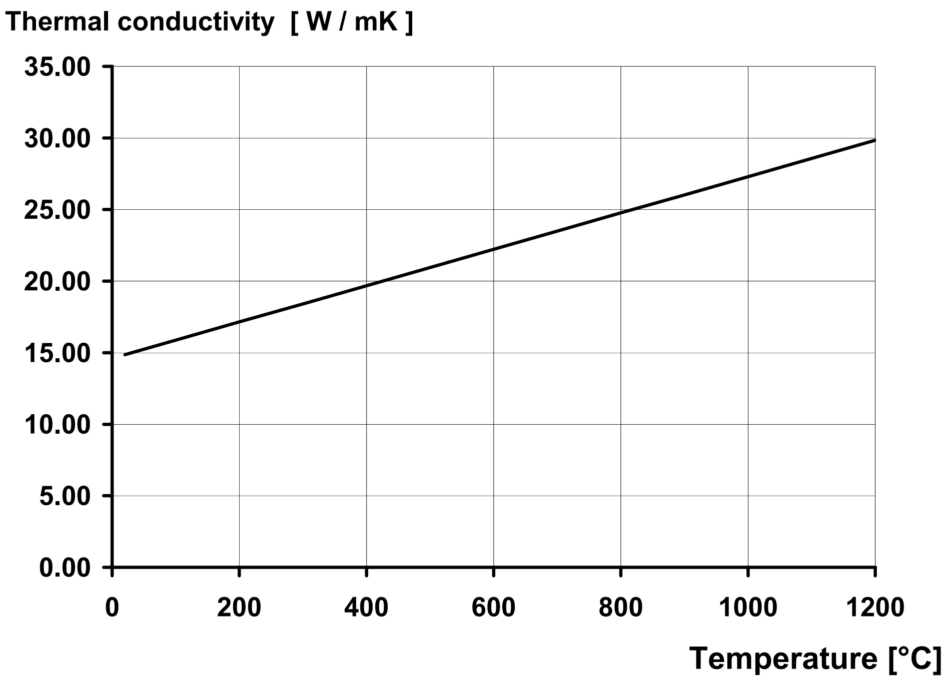


Figure C.4: Thermal conductivity of stainless steel as a function of the temperature

Annex D [informative] Connections

D.1 Bolted connections

PE (1) Net-section failure at fastener holes need not be considered, provided that there is a fastener in each hole, because the steel temperature is lower at connections due to the presence of additional material.

D1.1 Design Resistance of Bolts in Shear

D1.1.1Category A: Bearing Type

RC (1) The fire design resistance of bolts loaded in shear should be determined from:

$$F_{v,t,Rd} = F_{v,Rd}\, k_{b,\theta} \frac{\gamma_{M2}}{\gamma_{M,fi}} \qquad \text{(D.1)}$$

where

$k_{b,2}$ is the reduction factor determined for the appropriate bolt temperature from Table D.1;
$F_{v,Rd}$ is the design shear resistance of the bolt per shear plane calculated assuming that the shear plane passes through the threads of the bolt (clause 6.5.5 of EN 1993-1-8);
γ_{M2} is the partial safety factor at normal temperature;
$\gamma_{M,fi}$ is the partial safety factor for fire conditions.

RC (2) The design bearing resistance of bolts in fire should be determined from:

$$F_{b,t,Rd} = F_{b,Rd}\, k_{b,\theta} \frac{\gamma_{M2}}{\gamma_{M,fi}} \qquad \text{(D.2)}$$

where

$F_{b,Rd}$ is determined from clause 6.5.5 EN1993-1.8,
$k_{b,2}$ is the reduction factor determined for the appropriate bolt temperature from Table D.1

D1.1.2 Category B: Slip resistance at serviceability and category C Slip resistance at ultimate state

RC (1) Slip restraint connections should be considered as having slipped in fire and the resistance of a single bolt should be determined as for bearing type bolts, see D1.1.1.

D1.2 Design Resistance of Bolts in Tension

D1.2.1 Category D and E: Non-preloaded and preloaded bolts

RC (1) The design tension resistance of a single bolt in fire should be determined from:

$$F_{ten,t,Rd} = F_{t,Rd}\, k_{b,\theta} \frac{\gamma_{M2}}{\gamma_{M,fi}} \qquad \text{(D.3)}$$

where

$F_{t,Rd}$ is determined from clause 6.5.5 of EN 1993-1-8,
$k_{b,2}$ is the reduction factor determined for the appropriate bolt temperature from Table D.1

Table D.1: Strength Reduction Factors for Bolts and Welds

Temperature θ_a	Reduction factor for bolts, $k_{b,2}$ (Tension and shear)	Reduction factor for welds, $k_{w\,2}$
20	1,000	1,000
100	0,968	1,000
150	0,952	1,000
200	0,935	1,000
300	0,903	1,000
400	0,775	0,876
500	0,550	0,627
600	0,220	0,378
700	0,100	0,130
800	0,067	0,074
900	0,033	0,018
1000	0,000	0,000

D.2 Design Resistance of Welds

D2.1 Butt Welds

RC

(1) The design strength of a full penetration butt weld, for temperatures up to 700 °C, should be taken as equal to the strength of the weaker part joined using the appropriate reduction factors for structural steel. For temperatures >700 °C the reduction factors given for fillet welds can also be applied to butt welds.

D2.2 Fillet Welds

RC

(1) The design resistance per unit length of a fillet weld in fire should be determined from :

$$F_{w,t,Rd} = F_{w,Rd}\, k_{w,\theta} \frac{\gamma_{M2}}{\gamma_{M,fi}} \tag{D.4}$$

where

$k_{w,2}$ is obtained form Table D.1 for the appropriate weld temperature;
$F_{w,Rd}$is determined from clause 6.6.5. EN1 993-1-8.

D.3 Temperature of connections in fire

D3.1 General

PE (1) The temperature of a connection may be assessed using the local A/V value of the components comprising the connection.

PE (2) As a simplification an uniform distributed temperature may be assessed within the connection; this temperature may be calculated using the maximum value of the ratios A/V of the connected steel members in the vicinity of the connection.

PE (3) For beam to column and beam to beam connection, where the beams are supporting any type of concrete floor, the temperature for the connection may be obtained from the temperature of the bottom flange of the mid span.

PE (4) In applying the method in 4.2.5 the temperature of the connection components may be determined as follows:

a) If the depth of the beam is less than 400mm

$$\theta_h = 0{,}88\theta_o\,[1 - 0{,}3(h/D)] \tag{D.5}$$

where

θ_a is the temperature at height h (mm) of the steel beam (Figure D.1);
θ_o is the bottom flange temperature of the steel beam remote from the connection;
h is the height of the component being considered above the bottom of the beam in (mm);
D is the depth of the beam in (mm).

b) If the depth of the beam is greater than 400mm

i) When h is less than D/2

$$\theta_h = 0{,}88\theta_o \tag{D.6}$$

ii) When h is greater than D/2

$$\theta_h = 0{,}88\theta_o\,[1 + 0{,}2\,(1 - 2h/D)] \tag{D.7}$$

where

θ_o is the bottom flange temperature of the steel beam remote from the connection;
h is the height of the component being considered above the bottom of the beam in (mm);
D is the depth of the beam in (mm).

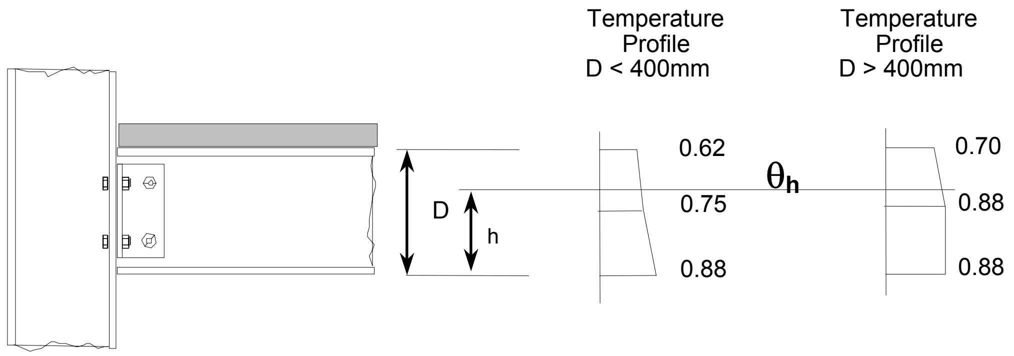


Figure D.1 Thermal gradient within the depth of a composite connection

Annex E [informative] Class 4 Cross-Sections

E.1 Advanced calculation models

PE (1) Advanced calculation models may be used for the design of class 4 sections when all stability effects are taken into account.

E.2 Simple calculation models

RC (1) The resistance of members with a class 4 cross section should be verified with the equations given in 4.2.3.2 for compression members, in 4.2.3.4 for beams, and in 4.2.3.5 for members subject to bending and axial compression, in which the area is replaced by the effective area and the section modulus is replaced by the effective section modulus.

RC (2) The effective cross section area and the effective section modulus should be determined in accordance with EN 1993-1-3 and EN 1993-1-5, i.e. based on the material properties at 20°C.

RC (3) For the design under fire conditions the design strength of steel should be taken as the 0,2 percent proof strength. This design strength may be used to determine the resistance to tension, compression, moment or shear.

PE (4) Reduction factors for the design strength of carbon steels relative to the yield strength at 20°C may be taken from table E.1:

- design strength , relative to yield strength at 20°C: $k_{p0,2,\theta} = f_{p0,2,\theta}/f_y$
- slope of linear elastic range, relative to slope at 20°C: $k_{E,\theta} = E_{a,\theta}/E_a$

NOTE: These reductions factors are illustrated in figure E.1.

PE (5) Reduction factors for the design strength of stainless steels relative to the yield strength at 20°C may be taken from annex C.

Table E.1: Reduction factors for carbon steel for the design of class 4 sections at elevated temperatures

Steel Temperature θ_a	Reduction factor (relative to f_y) for the design strength of hot rolled and welded thin walled sections $k_{p0,2,,\theta} = f_{p0,2,\theta}/f_y$	Reduction factor (relative to f_{yb}) for the design strength of cold formed thin walled sections $k_{p0,2,\theta} = f_{p0,2,\theta}/f_{yb}$
20°C	1,00	
100°C	1,00	
200°C	0,89	
300°C	0,78	
400°C	0,65	
500°C	0,53	
600°C	0,30	
700°C	0,13	
800°C	0,07	
900°C	0,05	
1000°C	0,03	
1100°C	0,02	
1200°C	0,00	

NOTE 1: For intermediate values of the steel temperature, linear interpolation may be used.

NOTE 2: The definition for f_{yb} should be taken from EN1993-1-3

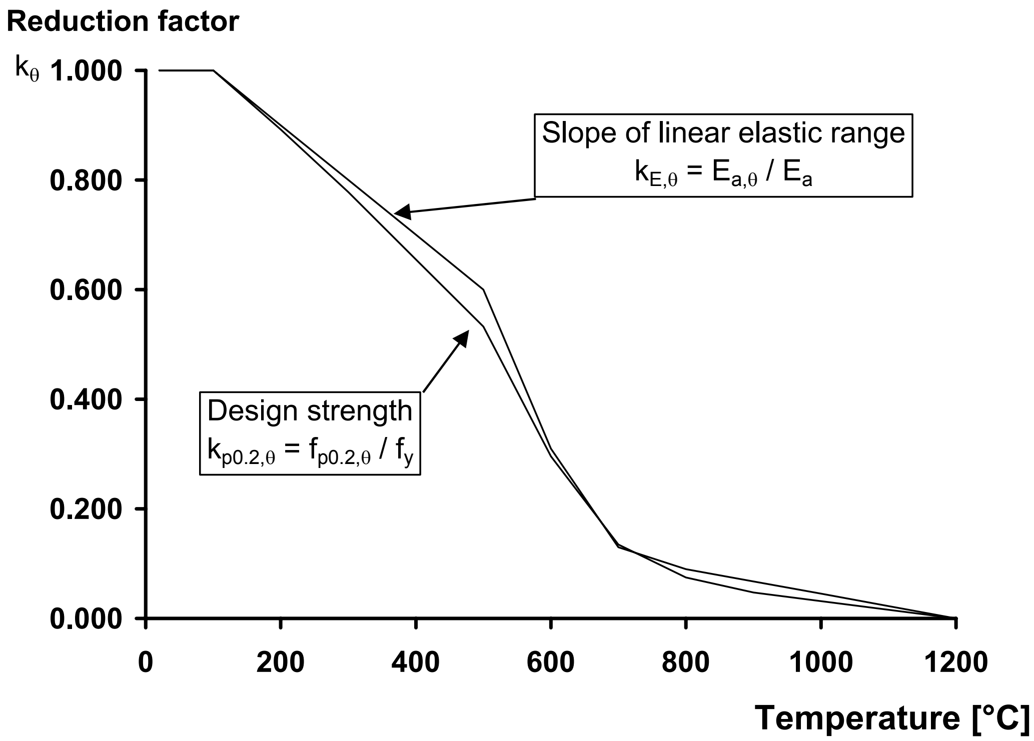


Figure E.2: Reduction factors for the stress-strain relationship of cold formed and hot rolled thin walled steel at elevated temperatures

EUROPEAN STANDARD **EN 1993-1-3 : 2004**

NORME EUROPÉENNE

EUROPÄISCHE NORM 04 July 2004

UDC

Descriptors:

English version

Eurocode 3 : Design of steel structures

Part 1-3: General rules
Supplementary rules for cold-formed members and sheeting

Eurocode 3: Calcul des structures en acier - Partie 1-3: Règles générales - Règles supplémentaires pour les profilés et plaques à parois minces formés à froid

Eurocode 3: Bemessung und Konstruktion von Stahlbauten - Teil 1-3: Allgemeine Regels - Ergänzende Regeln fur kaltgeformte dunnwandige Bauteile und Bleche

The improvements made on the version dated 1 March 2004 are marked

(improvements made on the original Project Team draft until 29 February 2004 are marked in the "dirty" version dated 1 March 2004)

Stage 34

CEN

European Committee for Standardisation

Comité Européen de Normalisation

Europäisches Komitee für Normung

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

 Ref. No. EN 1993-1-3 : 2004 E

CONTENT

Foreword

This document (prEN 1993-1-3: 2004) has been prepared by Technical Committee CEN/TC 250 "Structural Eurocodes", the secretariat of which is held be BSI.

This document is currently submitted to the Formal Vote.

This document will supersede ENV 1993-1-4.

National annex for EN 1993-1-3

This standard gives alternative procedures, values and recommendations for classes with notes indicating where national choices may have to be made. Therefore the National Standard implementing EN 1993-1-3 should have a National Annex containing all Nationally Determined Parameters to be used for the design of steel structures to be constructed in the relevant country.

National choice is allowed in EN 1993-1-3 through clauses:

- 2(3)
- 2(5)
- 3.1(4)
- 3.2.4(1)
- 5.3(4)
- 8.3(5)
- 8.3(13), Table 8.1 ~~(4 times)~~
- 8.3(13), Table 8.2
- 8.3(13), Table 8.3
- 8.3(13), Table 8.4
- 8.4(5)
- 8.5.1(4)
- 9(2)
- 10.1.1(1)
- 10.1.4.2(1)
- A.1(1), NOTE 2 ~~(2 times)~~
- A.1(1), NOTE 3
- A.6.4(4)
- E(1)

1 Introduction

1.1 Scope

(1) ~~Part 1-3 of~~ EN 1993-1-3 gives design requirements for cold-formed thin gauge members and sheeting. It applies to cold-formed steel products made from coated or uncoated thin gauge hot or cold rolled sheet or strip, that have been cold-formed by such processes as cold-rolled forming or press-braking. It may also be used for the design of profiled steel sheeting for composite steel and concrete slabs at the construction stage, see EN 1994. The execution of steel structures made of cold-formed thin gauge members and sheeting is covered in EN 1090.

NOTE The rules in this part complement the rules in other parts of EN 1993-1.

(2) Methods are also given for stressed-skin design using steel sheeting as a structural diaphragm.

(3) This part does not apply to cold-formed circular and rectangular structural hollow sections supplied to EN 10219, for which reference should be made to EN 1993-1-1 and EN 1993-1-8.

(4) ~~This Part 1-3 of~~ EN 1993-1-3 gives methods for design by calculation and for design assisted by testing. The methods for design by calculation apply only within stated ranges of material properties and geometrical proportions for which sufficient experience and test evidence is available. These limitations do not apply to design assisted by testing.

(5) EN 1993-1-3 does not cover load arrangement for testing for loads during execution and maintenance.

1.2 Normative references

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

EN 1993 *Eurocode 3 – Design of steel structures*

Part 1: *General rules and rules for buildings*

EN 10002 *Metallic materials - Tensile testing:*

Part 1: *Method of test (at ambient temperature)*;

EN 10025-1:2002 *Hot-rolled products of structural steels - Part 1: General delivery conditions;*

EN 10025-2:2002 *Hot-rolled products of structural steels - Part 2: Technical delivery conditions for non-alloy structural steels;*

EN 10025-3:2002 *Hot-rolled products of structural steels - Part 3: Technical delivery conditions for normalized / normalized rolled weldable fine grain structural steels;*

EN 10025-4:2002 *Hot-rolled products of structural steels - Part 4: Technical delivery conditions for thermomechanical rolled weldable fine grain structural steels;*

EN 10025-5:2002 *Hot-rolled products of structural steels - Part 5: Technical delivery conditions for structural steels with improved atmospheric corrosion resistance;*

EN 10142 *Continuously hot-dip zinc coated mild steel strip and sheet for cold-forming - Technical delivery conditions;*

EN 10143 *Continuously hot-dip metal coated steel sheet and strip - Tolerances on dimensions and shape*;

EN 10147 *Specification for continuously hot-dip zinc coated structural steel sheet - Technical delivery conditions*;

EN 10149 *Hot rolled flat products made of high yield strength steels for cold-forming:*

Part 2: *Delivery conditions for normalized/normalized rolled steels*;

Part 3: *Delivery conditions for thermomechanical rolled steels*;

EN 10154 *Continuously hot-dip aluminium-silicon (AS) coated steel strip and sheet - Technical delivery* *conditions;*

~~pr~~EN 10162 *Cold rolled steel sections - Technical delivery conditions - Dimensional and cross-sectional tolerance;*

EN 10204 *Metallic products. Types of inspection documents (includes amendment A 1:1995);*

EN 10214 *Continuously hot-dip zinc-aluminium (ZA) coated steel strip and sheet - Technical delivery conditions;*

EN 10215 *Continuously hot-dip aluminium-zinc (AZ) coated steel strip and sheet - Technical delivery conditions;*

~~EN 10219-1 *Cold formed welded structural hollow sections of non-alloy and fine grain steels - Technical delivery requirements;*~~

~~EN 10219-2 *Cold formed welded structural hollow sections of non-alloy and fine grain steels - Tolerances, dimensions and sectional properties;*~~

EN 10268 *Cold-rolled flat products made of high yield strength micro-alloyed steels for cold forming - General delivery conditions;*

EN 10292 *Continuously hot-dip coated strip and sheet of steels with higher yield strength for cold forming - Technical delivery conditions;*

EN-ISO 12944-2 *Paints and vanishes. Corrosion protection of steel structures by protective paint systems. Part 2: Classification of environments (ISO 12944-2:1998);*

EN 1090, Part 2 *Requirements for the execution of steel structures:*

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

EN ISO 1478 (ISO 1478:1983) *Tapping screws thread;*

EN ISO 1479 (ISO 1479:1983) *Hexagon head tapping screws;*

EN ISO 2702 (ISO 2702:1992) *Heat-treated steel tapping screws - Mechanical properties;*

EN ISO 7049 (ISO 7049:1983) *Cross recessed pan head tapping screws;*

ISO 1000

ISO 4997 *Cold reduced steel sheet of structural quality*;

EN 508-1 *Roofing products from metal sheet - Specification for self-supporting products of steel, aluminium or stainless steel sheet - Part 1: Steel;*

FEM 10.2.02 *Federation Europeenne de la manutention, Secion X, Equipment et proceedes de stockage, FEM 10.2.02, The design of static steel pallet racking, Racking design code, April 2001 Version 1.02.*

1.3 Termns and definitions

Supplementary to EN 1993-1-1, for the purposes of this Part 1-3 of EN 1993, the following terms and definitions apply:

1.3.1
basic material
The flat sheet steel material out of which cold-formed sections and profiled sheets are made by cold-forming.

1.3.2
basic yield strength
The tensile yield strength of the basic material.

1.3.3
diaphragm action
Structural behaviour involving in-plane shear in the sheeting.

1.3.4
liner tray
Profiled sheet with large lipped edge stiffeners, suitable for interlocking with adjacent liner trays to form a plane of ribbed sheeting that is capable of supporting a parallel plane of profiled sheeting spanning perpendicular to the span of the liner trays.

1.3.5
partial restraint
Restriction of the lateral or rotational movement, or the torsional or warping deformation, of a member or element, that increases its buckling resistance in a similar way to a spring support, but to a lesser extent than a rigid support.

1.3.6
relative slenderness
A normalized non-dimensional slenderness ratio.

1.3.7
restraint
Restriction of the lateral or rotational movement, or the torsional or warping deformation, of a member or element, that increases its buckling resistance to the same extent as a rigid support.

1.3.8
stressed-skin design
A design method that allows for the contribution made by diaphragm action in the sheeting to the stiffness and strength of a structure.

1.3.9
support
A location at which a member is able to transfer forces or moments to a foundation, or to another member or other structural component.

1.3.10
nominal thickness
A target average thickness inclusive zinc and other metallic coating layers when present rolled and defined by the steel supplier (t_{nom} not including organic coatings).

1.3.11
steel core thickness
A nominal thickness minus zinc and other metallic coating layers (t_{cor}).

1.3.12
design thickness
the steel core thickness used in design by calculation according to 3.2.4.

1.4 Symbols

(1) In addition to those given in EN 1993-1-1, the following main symbols are used:

Draft note:	Will be added later

(2) Additional symbols are defined where they occur.

1.5 Terminology and conventions for dimensions

1.5.1 Form of sections

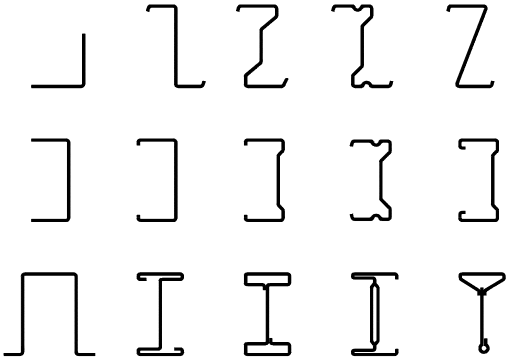
(1) Cold-formed members and profiled sheets have within the permitted tolerances a constant nominal

thickness over their entire length and may have either a ~~constant or a variable cross-section~~ uniform cross section or a tapering cross section along their length.

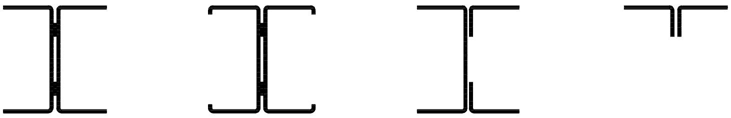
(2) The cross-sections of cold-formed members and profiled sheets essentially comprise a number of plane elements joined by curved elements.

(3) Typical forms of sections for cold-formed members are shown in figure 1.1.

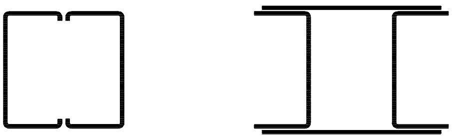
NOTE: The calculation methods of this Part 1-3 of EN 1993 does not cover all the cases shown in figures 1.1-1.2.



a) Single open sections



b) Open built-up sections

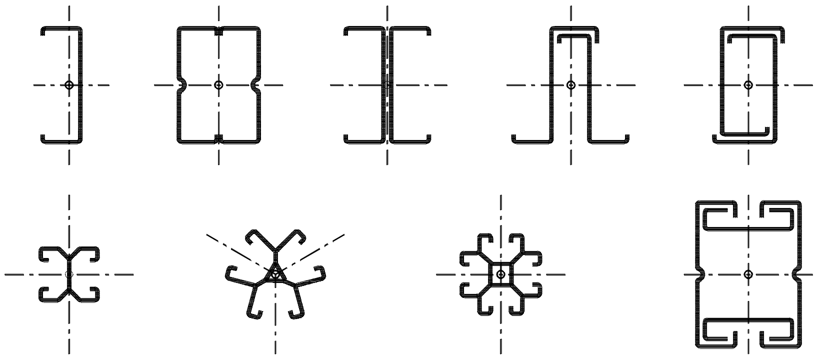


c) Closed built-up sections

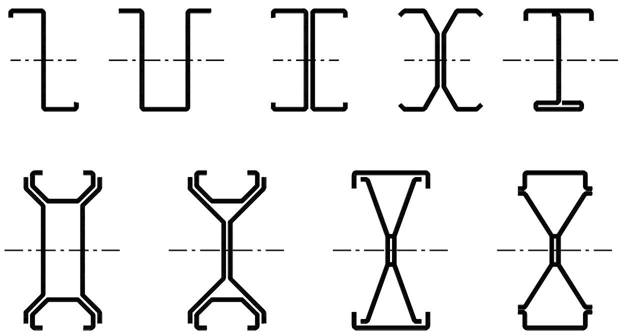
Figure 1.1: Typical forms of sections for cold-formed members

(4) Examples of cross-sections for cold-formed members and sheets are illustrated in figure 1.2.

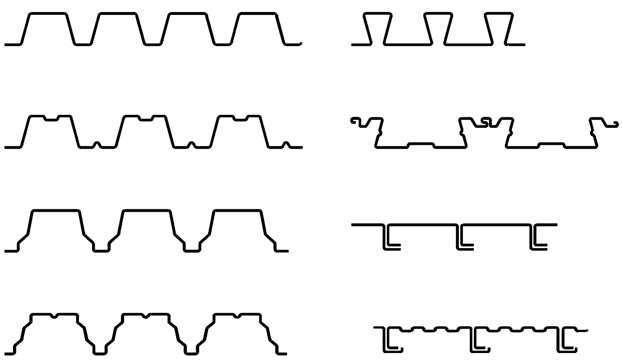
NOTE: All rules in this Part 1-3 of EN 1993 relate to the main axis properties, which are defined by the main axes y - y and z - z for symmetrical sections and u - u and v - v for unsymmetrical sections as e.g. angles and Zed-sections. In some cases the bending axis is imposed by connected structural elements whether the cross-section is symmetric or not.



a) Compression members and tension members



b) Beams and other members subject to bending



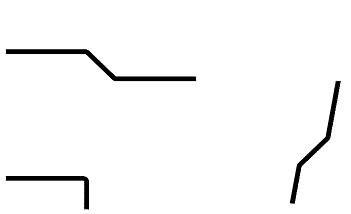
c) Profiled sheets and liner trays

Figure 1.2: Examples of cold-formed members and profiled sheets

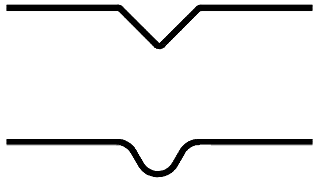
(5) Cross-sections of cold-formed members and sheets may either be unstiffened or incorporate longitudinal stiffeners in their webs or flanges, or in both.

1.5.2 Form of stiffeners

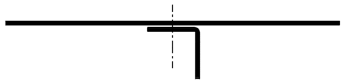
(1) Typical forms of stiffeners for cold-formed members and sheets are shown in figure 1.3.



a) Folds and bends



b) Folded groove and curved groove



c) Bolted angle stiffener

Figure 1.3: Typical forms of stiffeners for cold-formed members and sheeting

(2) Longitudinal flange stiffeners may be either edge stiffeners or intermediate stiffeners.

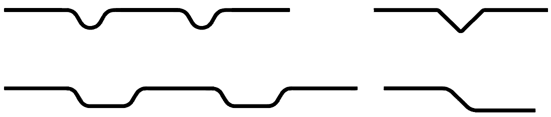
(3) Typical edge stiffeners are shown in figure 1.4.

a) Single edge fold stiffeners

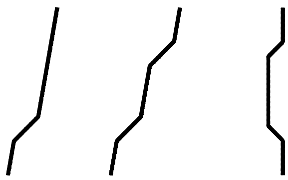
b) Double edge fold stiffeners

Figure 1.4: Typical edge stiffeners

(4) Typical intermediate longitudinal stiffeners are illustrated in figure 1.5.



a) Intermediate flange stiffeners



b) Intermediate web stiffeners

Figure 1.5: Typical intermediate longitudinal stiffeners

1.5.3 Cross-section dimensions

(1) Overall dimensions of cold-formed ~~thin gauge~~ members and sheeting, including overall width *b*, overall height *h*, internal bend radius *r* and other external dimensions denoted by symbols without subscripts, such as *a*, *c* or *d*, are measured to the face of the material, unless stated otherwise, as illustrated in figure 1.6.

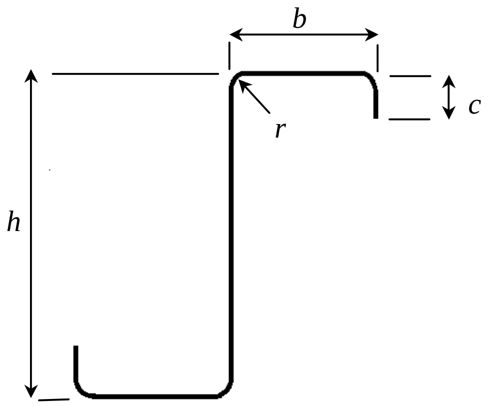


Figure 1.6: Dimensions of typical cross-section

(2) Unless stated otherwise, the other cross-sectional dimensions of cold-formed ~~thin gauge~~ members and sheeting, denoted by symbols with subscripts, such as b_d, h_w or s_w, are measured either to the midline of the material or the midpoint of the corner.

(3) In the case of sloping elements, such as webs of trapezoidal profiled sheets, the slant height s is measured parallel to the slope. The slope is straight line between intersection points of flanges and web.

(4) The developed height of a web is measured along its midline, including any web stiffeners.

(5) The developed width of a flange is measured along its midline, including any intermediate stiffeners.

(6) The thickness t is a steel design thickness (the steel core thickness extracted minus tolerance if needed as specified in clause 3.2.4), if not otherwise stated.

1.5.4 Convention for member axes

(1) In general the conventions for members is as used in Part 1-1 of EN 1993, see Figure 1.7.

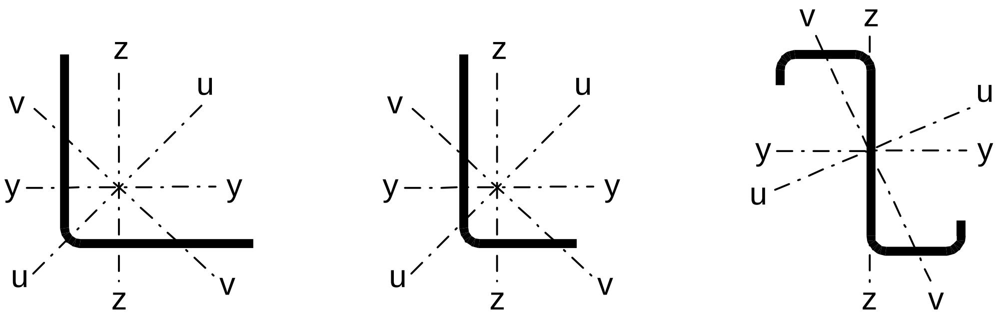


Figure 1.7: Axis convention

(2) For profiled sheets and liner trays the following axis convention is used:

- y - y axis parallel to the plane of sheeting;
- z - z axis perpendicular to the plane of sheeting.

2 Basis of design

(1) The design of cold formed ~~thin gauge~~ members and sheeting ~~shall~~ should be in accordance with the general rules given in EN 1990 and EN 1993-1-1. For a general approach with FE-methods (or others) see EN 1993-1-5, Annex C.

(2) Appropriate partial factors ~~shall~~ should be adopted for ultimate limit states and serviceability limit states.

(3) For verifications by calculation at ultimate limit states the partial factor γ_M ~~shall~~ should be taken as follows:

- resistance of cross-sections to excessive yielding including local and distortional buckling: γ_{M0}
- resistance of members and sheeting where failure is caused by global buckling: γ_{M1}
- resistance of net sections at bolt holes: γ_{M2}

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

$\gamma_{M0} = 1,00$;

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

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

(4) For values of γ_M for resistance of connections, see Section 8 of this Part 1-3.

(5) For verifications at serviceability limit states the partial factor $\gamma_{M,ser}$ ~~shall~~ should be used.

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

$\gamma_{M,ser} = 1,00$.

(6) For the design of structures made of cold formed ~~thin gauge~~ members and sheeting a distinction should be made between "structural classes" associated with failure consequences according to EN 1990 – Annex B defined as follows:

Structural Class I: Construction where cold-formed ~~thin gauge~~ members and sheeting are designed to contribute to the overall strength and stability of a structure;

Structural Class II: Construction where cold-formed ~~thin gauge~~ members and sheeting are designed to contribute to the strength and stability of individual structural elements;

Structural Class III: Construction where cold-formed sheeting is used as an element that only transfers loads to the structure.

NOTE 1: During different construction stages different ~~construction~~ structural classes may be considered.

NOTE 2: For requirements for execution of sheeting in structural classes I, II and III see EN 1090.

3 Materials

3.1 General

(1) All steels used for cold-formed members and profiled sheets ~~shall~~ should be suitable for cold-forming and welding, if needed. Steels used for members and sheets to be galvanized ~~shall~~ should also be suitable for galvanizing.

(2) The nominal values of material properties given in this Section should be adopted as characteristic values in design calculations.

(3) This part of EN 1993 covers the design of cold formed members and profiles sheets fabricated from steel material conforming to the steel grades listed in table 3.1a.

~~(4)Other materials and products not specified in European Product Standards may only be used if their use is evaluated in accordance with the relevant rules in this standard and the applicable National Annex.~~

Table 3.1a: Nominal values of basic yield strength f_{yb} and ultimate tensile strength f_u

Type of steel	Standard	Grade	f_{yb} N/mm²	f_u N/mm²
Hot rolled products of non-alloy structural steels. Part 2: Technical delivery conditions for non alloy structural steels	EN 10025: Part 2	S 235	235	360
		S 275	275	430
		S 355	355	510
Hot-rolled products of structural steels. Part 3: Technical delivery conditions for normalized/normalized rolled weldable fine grain structural steels	EN 10025: Part 3	S 275 N	275	370
		S 355 N	355	470
		S 420 N	420	520
		S 460 N	460	550
		S 275 NL	275	370
		S 355 NL	355	470
		S 420 NL	420	520
		S 460 NL	460	550
Hot-rolled products of structural steels. Part 4: Technical delivery conditions for thermomechanical rolled weldable fine grain structural steels	EN 10025: Part 4~~5~~	S 275 M	275	360
		S 355 M	355	450
		S 420 M	420	500
		S 460 M	460	530
		S 275 ML	275	360
		S 355 ML	355	450
		S 420 ML	420	500
		S 460 ML	460	530

~~1) Minimum values of the yield strength and ultimate tensile strength are not given in the standard. For all steel grades a minimum value of 140 N/mm² for yield strength and 270 N/mm² for ultimate tensile strength may be assumed.~~

~~2) The yield strength values given in the names of the materials correspond to transversal tension. The values for longitudinal tension are given in the table.~~

NOTE: For other steel materials and products see National Annex. Examples for steel grades that may conform to the requirements of this standard are given in Table 3.1b. ~~It is assued that for materials for which the nominal ultimate tensile strength is higher than 550 N/mm² the resistance and ductility is verified by testing.~~

Table 3.1b: Nominal values of basic yield strength f_{yb} and ultimate tensile strength f_u

Cold reduced steel sheet of structural quality	ISO 4997	CR 220	220	300
		CR 250	250	330
		CR 320	320	400
Continuous hot dip zinc coated carbon steel sheet of structural quality	EN 10147	S220GD+Z	220	300
		S250GD+Z	250	330
		S280GD+Z	280	360
		S320GD+Z	320	390
		S350GD+Z	350	420
Hot-rolled flat products made of high yield strength steels for cold forming. Part 2: Delivery conditions for thermomechanically rolled steels	EN 10149: Part 2	S 315 MC	315	390
		S 355 MC	355	430
		S 420 MC	420	480
		S 460 MC	460	520
		S 500 MC	500	550
		S 550 MC	550	600
		S 600 MC	600	650
		S 650 MC	650	700
		S 700 MC	700	750
	EN 10149: Part 3	S 260 NC	260	370
		S 315 NC	315	430
		S 355 NC	355	470
		S 420 NC	420	530
Cold-rolled flat products made of high yield strength micro-alloyed steels for cold forming	EN 10268	H240LA	240	340
		H280LA	280	370
		H320LA	320	400
		H360LA	360	430
		H400LA	400	460
Continuously hot-dip coated strip and sheet of steels with higher yield strength for cold forming	EN 10292	H260LAD	240 2)	340 2)
		H300LAD	280 2)	370 2)
		H340LAD	320 2)	400 2)
		H380LAD	360 2)	430 2)
		H420LAD	400 2)	460 2)
Continuously hot-dipped zinc-aluminium (ZA) coated steel strip and sheet	EN 10214	S220GD+ZA	220	300
		S250GD+ZA	250	330
		S280GD+ZA	280	360
		S320GD+ZA	320	390
		S350GD+ZA	350	420
Continuously hot-dipped aluminium-zinc (AZ) coated steel strip and sheet	EN 10215	S220GD+AZ~~A~~	220	300
		S250GD+AZ~~A~~	250	330
		S280GD+AZ~~A~~	280	360
		S320GD+AZ~~A~~	320	390
		S350GD+AZ~~A~~	350	420
Continuously hot-dipped zinc coated strip and sheet of mild steel for cold forming	EN 10142	DX51D+Z	140 1)	270 1)
		DX52D+Z	140 1)	270 1)
		DX53D+Z	140 1)	270 1)

1) Minimum values of the yield strength and ultimate tensile strength are not given in the standard. For all steel grades a minimum value of 140 N/mm^2 for yield strength and 270 N/mm^2 for ultimate tensile strength may be assumed.

2) The yield strength values given in the names of the materials correspond to transversal tension. The values for longitudinal tension are given in the table.

~~**Drafting note:** References to other technical specifications not included to Table 3.1 to be given.~~

3.2 Structural steel

3.2.1 Material properties of base material

(1) The nominal values of yield strength f_{yb} or tensile strength f_u ~~shall~~ should be obtained

a) either by adopting the values $f_y = R_{eH}$ or $R_{p0,2}$ and $f_u = R_m$ direct from product standards, or

b) by using the values given in Table 3.1

c) by appropriate tests.

(2) Where the characteristic values are determined from tests, such tests ~~shall~~ should be carried out in accordance with EN 10002-1. The number of test coupons should be at least 5 and should be taken from a lot in following way:

1. Coils: a. For a lot from one production (one pot of melted steel) at least one coupon per coil of 30% of the number of coils;

   b. For a lot from different productions at least one coupon per coil;

2. Strips: At least one coupon per 2000 kg from one production.

The coupons should be taken at random from the concerned lot of steel and the orientation should be in the length of the structural element. The characteristic values ~~shall~~ should be determined on basis of a statistical evaluation in accordance with EN 1990 Annex D.

(3) It may be assumed that the properties of steel in compression are the same as those in tension.

(4) The ductility requirements should comply with 3.2.2 of EN 1993-1-1.

(5) The design values for material coefficients ~~shall~~ should be taken as given in 3.2.6 of EN 1993-1-1

(6) The material properties for elevated temperatures are given in EN 1993-1-2.

3.2.2 Material properties of cold formed sections and sheeting

(1) Where the yield strength is specified using the symbol f_y the average yield strength f_{ya} may be used if (4) to (8) apply~~may be used, unless in (4) to (8) apply~~. ~~In that case~~ In other cases the basic yield strength f_{yb} ~~shall~~ should be used. Where the yield strength is specified using the symbol f_{yb} the basic yield strength f_{yb} ~~shall~~ should be used.

(2) The average yield strength f_{ya} of a cross-section due to cold working may be determined from the results of full size tests.

(3) Alternatively the increased average yield strength f_{ya} may be determined by calculation using:

$$f_{ya} = f_{yb} + (f_u - f_{yb})\frac{knt^2}{A_g} \quad \text{but} \quad f_{ya} \leq \frac{(f_u + f_{yb})}{2} \qquad \ldots (3.1)$$

where:

A_g is the gross cross-sectional area;

k is a numerical coefficient that depends on the type of forming as follows:

- $k = 7$ for roll forming;
- $k = 5$ for other methods of forming;

n is the number of $90°$ bends in the cross-section with an internal radius $r \leq 5t$ (fractions of $90°$ bends should be counted as fractions of n);

t is the design core thickness of the steel material before cold-forming, exclusive of metal and organic coatings, see 3.2.4.

(4) The increased yield strength due to cold forming may be taken into account as follows:

- in axially loaded members in which the effective cross-sectional area A_{eff} equals the gross area A_g;
- in determining A_{eff} the yield strength f_y should be taken as f_{yb}.

(5) The average yield strength f_{ya} may be utilised in determining:

- the cross-section resistance of an axially loaded tension member;
- the cross-section resistance and the buckling resistance of an axially loaded compression member with a fully effective cross-section;
- the moment resistance of a cross-section with fully effective flanges.

(6) To determine the moment resistance of a cross-section with fully effective flanges, the cross-section may be subdivided into m nominal plane elements, such as flanges. Expression (3.1) may then be used to obtain values of increased yield strength $f_{y,i}$ separately for each nominal plane element i, provided that:

$$\frac{\sum_{i=1}^{m} A_{g,i}\, f_{y,i}}{\sum_{i=1}^{m} A_{g,i}} \leq f_{ya} \qquad \text{… (3.2)}$$

where:

$A_{g,i}$ is the gross cross-sectional area of nominal plane element i,

and when calculating the increased yield strength $f_{y,i}$ using the expression (3.1) the bends on the edge of the nominal plane elements should be counted with the half their angle for each area $A_{g,i}$.

(7) The increase in yield strength due to cold forming ~~shall~~ should not be utilised for members that are subjected to heat treatment after forming at more than 580°C for more than one hour.

NOTE: For further information see EN 1090, Part 2.

(8) Special attention should be paid to the fact that some heat treatments (especially annealing) might induce a reduced yield strength lower than the basic yield strength f_{yb}.

NOTE: For welding in cold formed areas see also EN 1993-1-8.

3.2.3 Fracture toughness

(1) See EN 1993-1-1 and EN 1993-1-10.

3.2.4 Thickness and thickness tolerances

(1) The provisions for design by calculation given in this Part 1-3 of EN 1993 may be used for steel within given ranges of core thickness t_{cor}:

NOTE: The ranges of core thickness t_{cor} for sheeting and members may be given in the National Annex. The following values are recommended:

- for sheeting and members: 0,45 mm $\leq t_{cor} \leq$ 15 mm ~~except where otherwise specified, e.g. for joints in section 8 , where $t_{cor} \leq$ 4 mm.~~
- for connections: 0,45 mm $\leq t_{cor} \leq$ 4 mm , see 8.1(2)

(2) Thicker or thinner material may also be used, provided that the load bearing resistance is determined by design assisted by testing.

(3) The steel core thickness t_{cor} should be used as design thickness, where

$$t_{cor} = (t_{nom} - t_{metallic\ coatings}) \qquad \text{if } tol \leq 5\% \qquad \text{… (3.3a)}$$

$$t_{cor} = (t_{nom} - t_{metallic\ coatings})\frac{100 - tol}{95} \qquad \text{if } tol > 5\% \qquad \text{… (3.3b)}$$

where *tol* is the minus tolerance in %.

NOTE: For the usual Z 275 zinc coating, t_{zinc} = 0,04 mm.

(4) For continuously hot-dip metal coated members and sheeting supplied with negative tolerances less or equal to the “special tolerances (S)” given in EN 10143, the design thickness according to (3.3a) may be used. If the negative tolerance is beyond "special tolerance (S)" given in EN 10143 then the design thickness according to (3.3b) may be used.

(5) t_{nom} is the nominal sheet thickness after cold forming. It may be taken as the value to t_{nom} of the original sheet, if the calculative cross-sectional areas before and after cold forming do not differ more than 2%; otherwise the notional dimensions should be changed.

3.3 Connecting devices

3.3.1 Bolt assemblies

(1) Bolts, nuts and washers ~~shall~~ should conform to the requirements given in EN 1993-1-8.

3.3.2 Other types of mechanical fastener

(1) Other types of mechanical fasteners as:

- self-tapping screws as thread forming self-tapping screws, thread cutting self-tapping screws or self-drilling self-tapping screws,
- cartridge-fired pins,
- blind rivets

may be used where they comply with the relevant ~~EN Product Standards or ETAG or ETA~~ European Product Specification.

(2) The characteristic shear resistance $F_{v,Rk}$ and the characteristic minimum tension resistance $F_{t,Rk}$ of the mechanical fasteners may be taken from the EN Product Standard or ETAG ~~i~~or ETA.

3.3.3 Welding consumables

(1) Welding consumables ~~shall~~ should conform to the requirements given in EN 1993-1-8.

4 Durability

(1) For basic requirements see section 4 of EN 1993-1-1.

NOTE: EN 1090 lists the factors affecting execution that need to be specified during design.

(2) Special attention should be given to cases in which different materials are intended to act compositely, if these materials are such that electrochemical phenomena might produce conditions leading to corrosion.

NOTE 1 For corrosion resistance of fasteners for the environmental class following EN-ISO 12944-2 see Annex B

NOTE 2: For roofing products see EN 508-1.

NOTE 3: For other products see Part 1-1 of EN 1993.

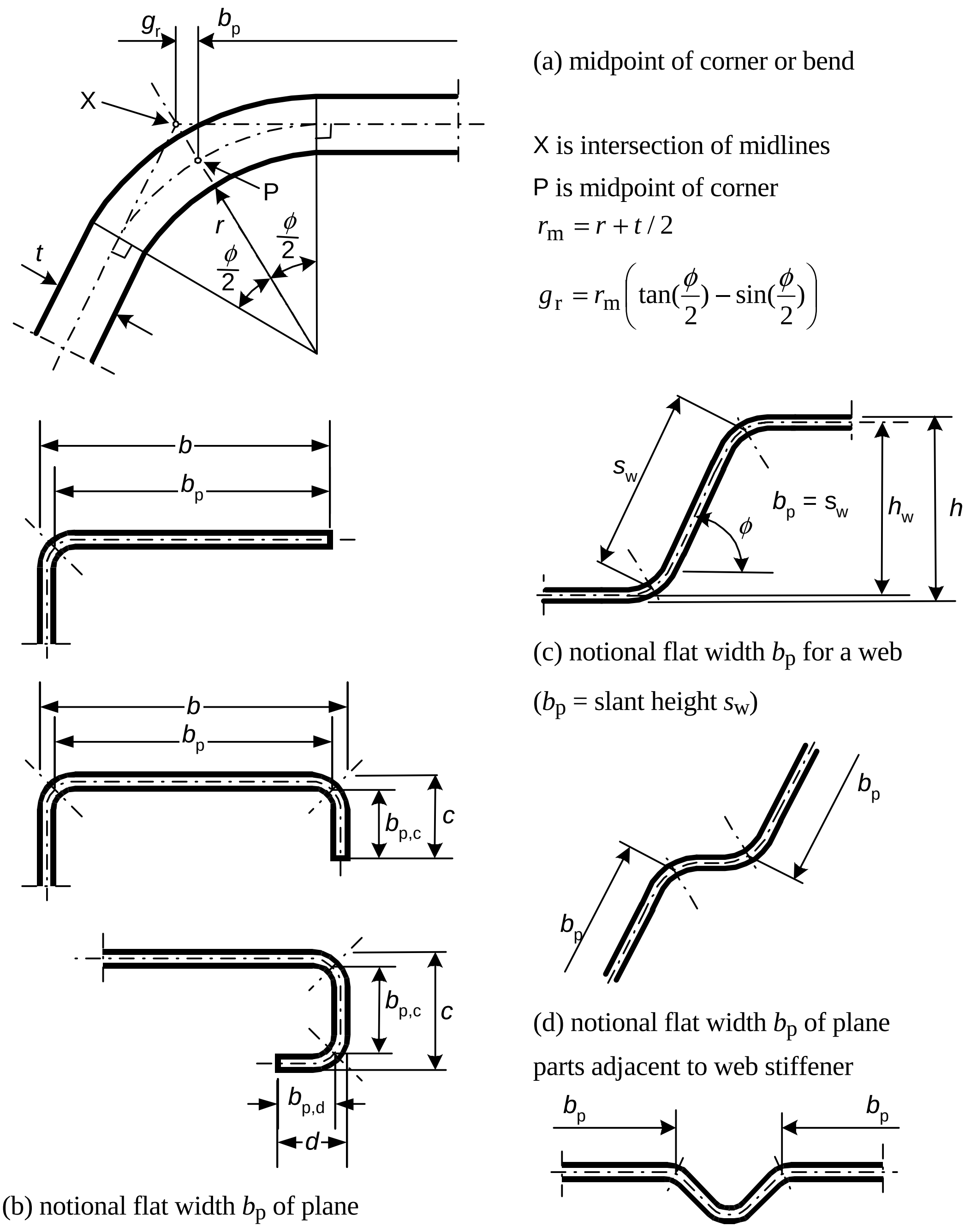
5 Structural analysis

5.1 Influence of rounded corners

(1) In cross-sections with rounded corners, the notional flat widths b_p of the plane elements ~~shall~~ should be measured from the midpoints of the adjacent corner elements as indicated in figure 5.3.

(2) In cross-sections with rounded corners, the calculation of section properties should be based upon the actual geometry of the cross-section.

(3) Unless more appropriate methods are used to determine the section properties the following approximate procedure may be used. The influence of rounded corners on cross-section resistance may be neglected if the internal radius $r \leq 5\,t$ and $r \leq 0{,}10\,b_p$ and the cross-section may be assumed to consist of plane elements with sharp corners (according to figure 5.4, note b_p for all flat plane elements, inclusive plane elements in tension). For cross-section stiffness properties the influence of rounded corners should always be taken into account.



(a) midpoint of corner or bend

(b) notional flat width b_p of plane parts of flanges

(c) notional flat width b_p for a web (b_p = slant height s_w)

(d) notional flat width b_p of plane parts adjacent to web stiffener

(e) notional flat width b_p of flat parts adjacent to flange stiffener

Figure 5.3: Notional widths of plane cross section parts b_p allowing for corner radii

(4) The influence of rounded corners on section properties may be taken into account by reducing the properties calculated for an otherwise similar cross-section with sharp corners, see figure 5.4, using the following approximations:

$$A_g \approx A_{g,sh}(1 - \delta) \qquad \text{… (5.1a)}$$

$$I_g \approx I_{g,sh}(1 - 2\delta) \qquad \text{… (5.1b)}$$

$$I_w \approx I_{w,sh}(1 - 4\delta) \qquad \text{… (5.1c)}$$

with:

$$\delta = 0{,}43 \frac{\sum_{j=1}^{n} r_j \frac{\phi_j}{90°}}{\sum_{i=1}^{m} b_{p,i}} \qquad \text{… (5.1d)}$$

where:

A_g	is	the area of the gross cross-section;
$A_{g,sh}$	is	the value of A_g for a cross-section with sharp corners;
$b_{p,i}$	is	the notional flat width of plane element i for a cross-section with sharp corners;
I_g	is	the second moment of area of the gross cross-section;
$I_{g,sh}$	is	the value of I_g for a cross-section with sharp corners;
I_w	is	the warping constant of the gross cross-section;
$I_{w,sh}$	is	the value of I_w for a cross-section with sharp corners;
ϕ	*is*	*the angle between two plane elements;*
m	is	the number of plane elements;
n	is	the number of curved elements;
r_j	is	the internal radius of curved element j.

(5) The reductions given by expression (5.1) may also be applied in calculating the effective section properties A_{eff}, $I_{y,eff}$, $I_{z,eff}$ and $I_{w,eff}$, provided that the notional flat widths of the plane elements are measured to the points of intersection of their midlines.

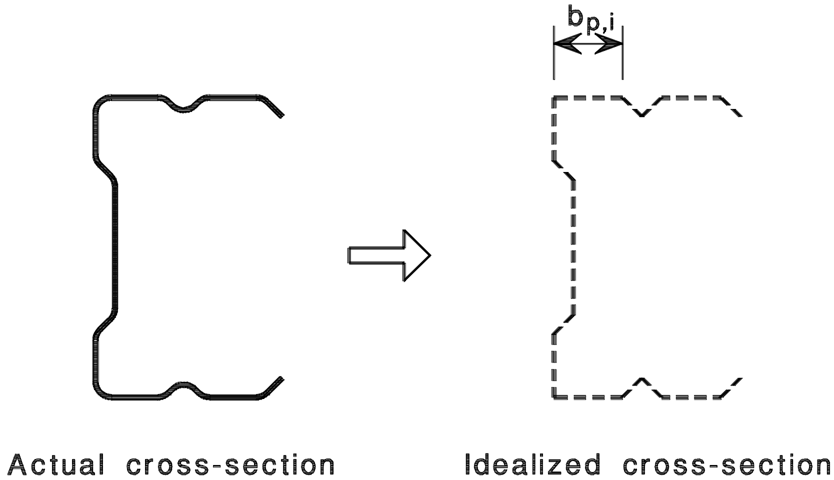


Figure 5.4: Approximate allowance for rounded corners

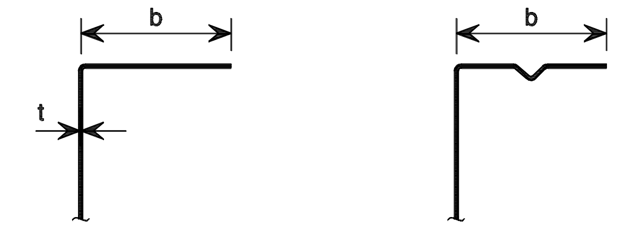
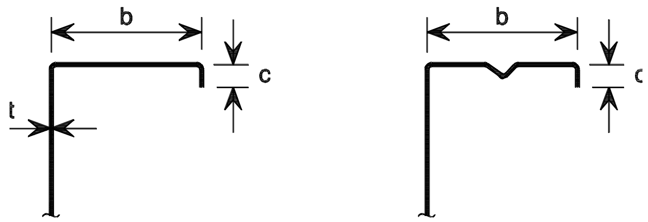
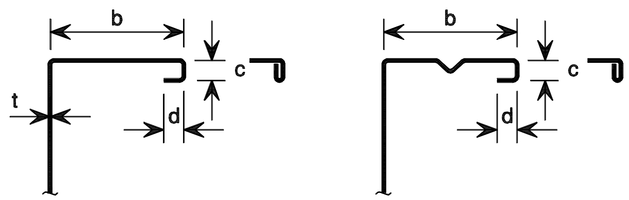
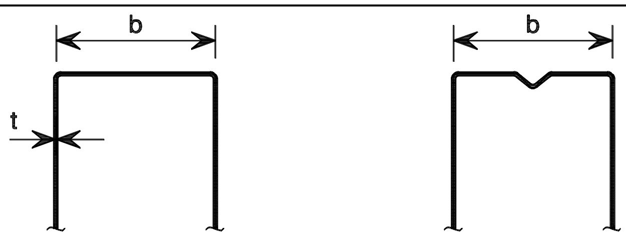
(6) Where the internal radius $r > 0{,}04\, t\, E / f_y$ then the resistance of the cross-section should be determined by tests.

5.2 Geometrical proportions

(1) The provisions for design by calculation given in this Part 1-3 of EN 1993 ~~shall~~ should not be applied to cross-sections outside the range of width-to-thickness ratios b/t, ~~and~~ h/t, c/t and d/t given in Table 5.1.

NOTE These limits b/t, ~~and~~ h/t, c/t and d/t given in table 5.1 may be assumed to represent the field for which sufficient experience and verification by testing is already available. Cross-sections with larger width-to-thickness ratios may also be used, provided that their resistance at ultimate limit states and their behaviour at serviceability limit states are verified by testing and/or by calculations, where the results are confirmed by an appropriate number of tests.

Table 5.1: Maximum width-to-thickness ratios

Element of cross-section	Maximum value
	$b/t \le 50$
	$b/t \le 60$ $c/t \le 50$
	$b/t \le 90$ $c/t \le 60$ $d/t \le 50$
	$b/t \le 500$
	$45° \le \phi \le 90°$ $h/t \le 500 \sin\phi$

(2) In order to provide sufficient stiffness and to avoid primary buckling of the stiffener itself, the sizes of stiffeners should be within the following ranges:

$$0{,}2 \le c/b \le 0{,}6 \qquad \text{… (5.2a)}$$

$$0{,}1 \le d/b \le 0{,}3 \qquad \text{… (5.2b)}$$

in which the dimensions b, c and d are as indicated in table 5.1. If $c/b \le$ 0,2 or $d/b \le$ 0,1 the lip should be ignored (c = 0 or d = 0).

NOTE 1 Where effective cross-section properties are determined by testing and by calculations, these limits do not apply.

NOTE 2: The lip measure c is perpendicular to the flange if the lip is not perpendicular to the flange.

NOTE 3 For FE-methods see Annex C of EN 1993-1-5.

5.3 Structural modelling for analysis

(1) Unless more appropriate models are used according to EN 1993-1-5 the elements of a cross-section may be modelled for analysis as indicated in table 5.2.

(2) The mutual influence of multiple stiffeners should be taken into account.

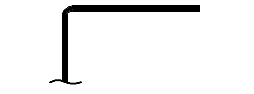
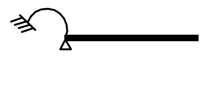
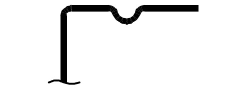
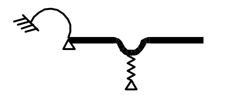
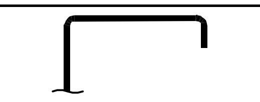
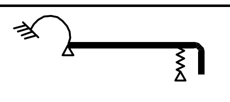
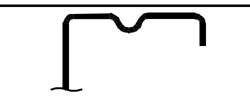
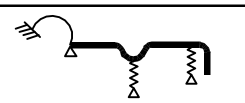
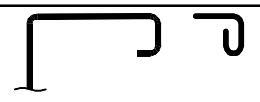
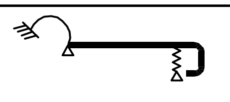
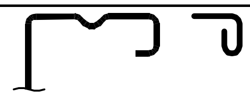
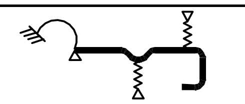
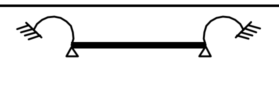
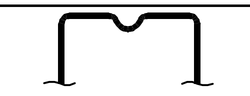
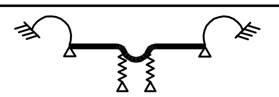
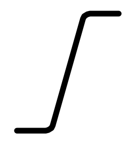
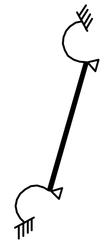
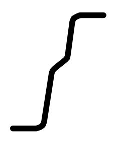
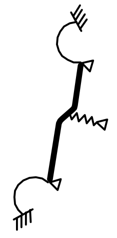
(3) Imperfections related to flexural buckling and torsional flexural buckling should be taken from table 5.1 of EN 1993-1-1

NOTE See also clause 5.3.4 of EN 1993-1-1.

(4) For imperfections related to lateral torsional buckling an initial bow imperfections e_0 of the weak axis of the profile may be ~~without~~ assumed without taking account at the same time an initial twist

NOTE The magnitude of the imperfection may be taken from the National Annex. The values $e_0/L = 1/600$ for elastic analysis and $e_0/L = 1/500$ for plastic analysis ~~$L/e_0 = 600$ is~~ are recommended for sections assigned to LTB buckling curve a~~b~~ taken from EN 1993-1-1, section 6.3.2.2.~~;~~

Table 5.2: Modelling of elements of a cross-section

Type of element	Model	Type of element	Model
			
			
			
			
			

5.4 Flange curling

(1) The effect on the loadbearing resistance of curling (i.e. inward curvature towards the neutral plane) of a very wide flange in a profile subjected to flexure, or of a flange in an arched profile subjected to flexure in which the concave side is in compression, should be taken into account unless such curling is less than 5% of the depth of the profile cross-section. If curling is larger, then the reduction in loadbearing resistance, for instance due to a decrease in the length of the lever arm for parts of the wide flanges, and to the possible effect of the bending of the webs should be taken into account.

NOTE: For liner trays this effect has been taken into account in 10.2.2.2.

(2) Calculation of the curling may be carried out as follows. The formulae apply to both compression and tensile flanges, both with and without stiffeners, but without closely spaced transversal stiffeners at flanges. For a profile which is straight prior to application of loading (see figure 5.5),

$$u = 2\frac{\sigma_a^{\ 2}}{E^2}\frac{b_s^{\ 4}}{t^2 z} \qquad \text{... (5.3a)}$$

For an arched beam:

$$u = 2\frac{\sigma_a}{E}\frac{b_s^{\ 4}}{t^2 r} \qquad \text{... (5.3b)}$$

where:

u is bending of the flange towards the neutral axis (curling), see figure 5.5;

b_s is one half the distance between webs in box and hat sections, or the width of the portion of flange projecting from the web, see figure 5.5;

t is flange thickness;

z is distance of flange under consideration from neutral axis;

r is radius of curvature of arched beam;

σ_a is mean stress in the flanges calculated with gross area. If the stress has been calculated over the effective cross-section, the mean stress is obtained by multiplying the stress for the effective cross-section by the ratio of the effective flange area to the gross flange area.

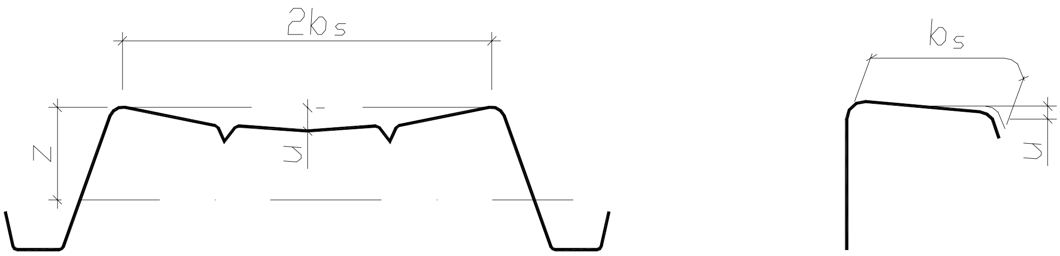


Figure 5.5: Flange curling

5.5 Local and distortional buckling

5.5.1 General

(1) The effects of local and distortional buckling ~~shall~~ should be taken into account in determining the resistance and stiffness of cold-formed members and sheeting.

(2) Local buckling effects may be accounted for by using effective cross-sectional properties, calculated on the basis of the effective widths, see EN 1993-1-5.

(3) In determining resistance to local buckling, the yield strength f_y should be taken as f_{yb} when calculating effective widths of compressed elements in EN 1993-1-5.

NOTE: For resistance see 6.1.3(1).

(4) For serviceability verifications, the effective width of a compression element should be based on the compressive stress $\sigma_{com,Ed,ser}$ in the element under the serviceability limit state loading.

(5) The distortional buckling for elements with edge or intermediate stiffeners as indicated in figure 5.6(d) are considered in Section 5.5.3.

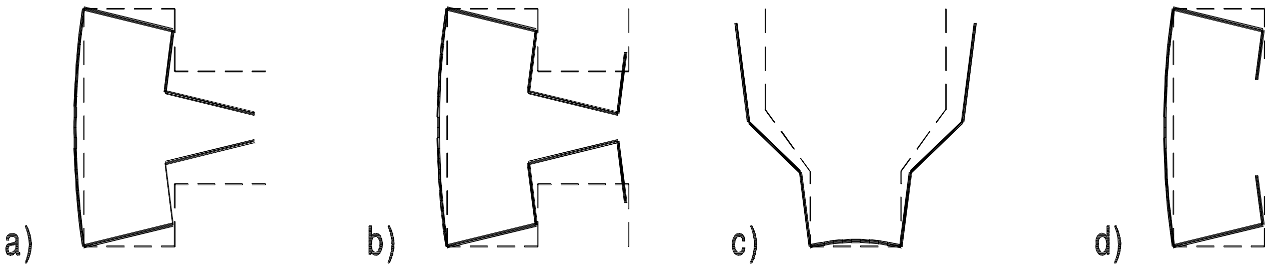


Figure 5.6: Examples of distortional buckling modes

(6) The effects of distortional buckling should be allowed for in cases such as those indicated in figures 5.6(a), (b) and (c). In these cases the effects of distortional buckling should be determined performing linear (see 5.5.1(8)) or non-linear buckling analysis (see EN 1993-1-5) using numerical methods or column stub tests.

(7) Unless the simplified procedure in 5.5.3 is used and where the elastic buckling stress is obtained from linear buckling analysis the following procedure may be applied:

1) For the wavelength up to the actual member length, calculate the elastic buckling stress and identify the corresponding buckling modes, see figure 5.7a.
2) Calculate the effective width(s) according to 5.5.2 for locally buckled cross-section parts based on the minimum local buckling stress, see figure 5.7b.
3) Calculate the reduced thickness (see 5.5.3.1(7)) of edge and intermediate stiffeners or other cross-section parts undergoing distortional buckling based on the minimum distortional buckling stress, see figure 5.7b.
4) Calculate overall buckling resistance according to 6.2 (flexural, torsional or lateral-torsional buckling depending on buckling mode) for actual member length and based on the effective cross-section from 2) and 3).

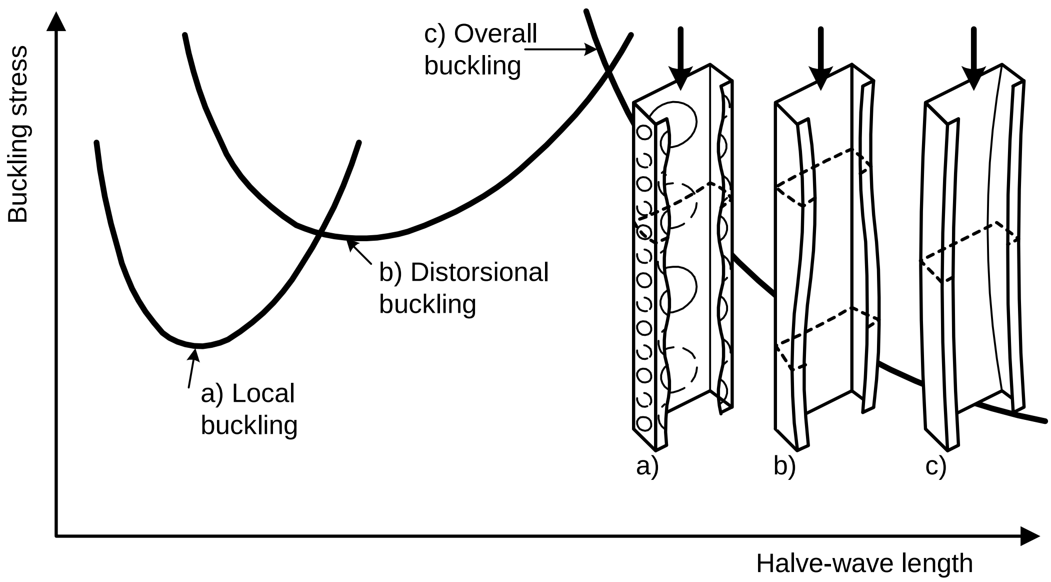


Figure 5.7a: Examples of elastic critical stress for various buckling modes as function of *halve-wave length* and examples of buckling modes.

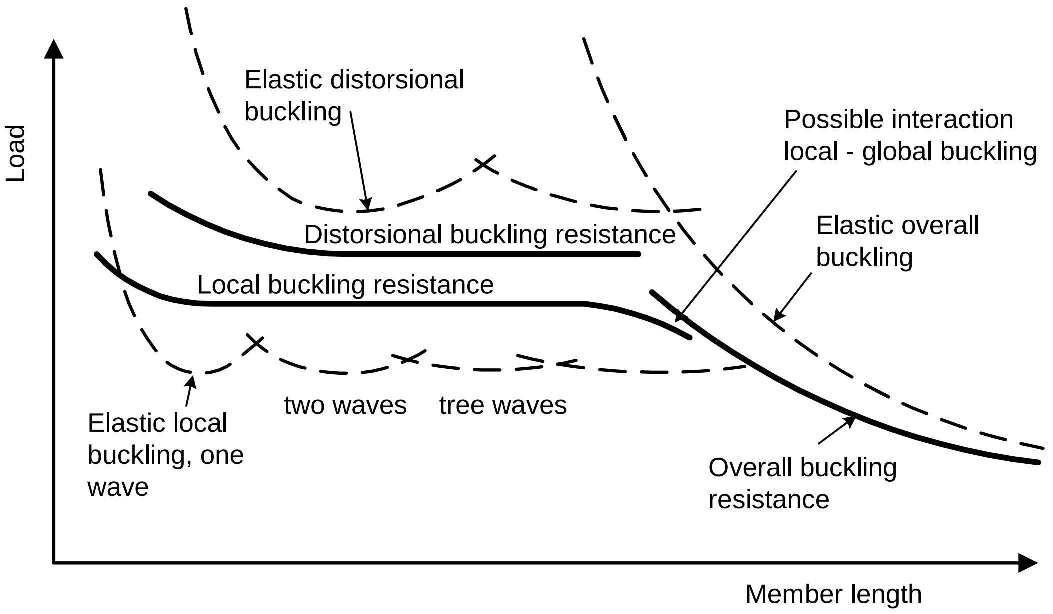


Figure 5.7b: Examples of elastic buckling load and buckling resistance as a function of *member length*

5.5.2 Plane elements without stiffeners

(1) The effective widths of unstiffened elements should be obtained from EN 1993-1-5 using the notional flat width b_p for $\overline{b}$.

(2) The notional flat width b_p of a plane element should be determined as specified in figure 5.3 of section 5.1.4. In the case of plane elements in a sloping webs, the appropriate slant height should be used.

NOTE For outstands a more refined method for calculating effective widths is given in Annex D.

(3) In applying the method in EN 1993-1-5 the following procedure may be used:

- The stress ratio ψ, from tables 5.3 and 5.4 used to determine the effective width of flanges of a section subject to stress gradient, may be based on gross section properties.
- The stress ratio ψ, from table 5.3 and 5.4 used to determine the effective width of web, may be obtained using the effective area of compression flange and the gross area of the web.
- The effective section properties may be refined by ~~repeating (7) an (8) iteratively, but~~ using ~~in (7)~~ the stress ratio ψ based on the effective cross-section already found in place of the gross cross-section. The minimum steps in the iteration dealing with the stress gradient are two.
- The simplified method given in 5.5.3.4 may be used in the case of webs of trapezoidal sheeting under stress gradient.

5.5.3 Plane elements with edge or intermediate stiffeners

5.5.3.1 General

(1) The design of compression elements with edge or intermediate stiffeners should be based on the assumption that the stiffener behaves as a compression member with continuous partial restraint, with a spring stiffness that depends on the boundary conditions and the flexural stiffness of the adjacent plane elements.

(2) The spring stiffness of a stiffener should be determined by applying an unit load per unit length u as illustrated in figure 5.8. The spring stiffness K per unit length may be determined from:

$$K = u/\delta \qquad \text{... (5.9)}$$

where:

δ is the deflection of the stiffener due to the unit load u acting in the centroid (b_1) of the effective part of the cross-section.

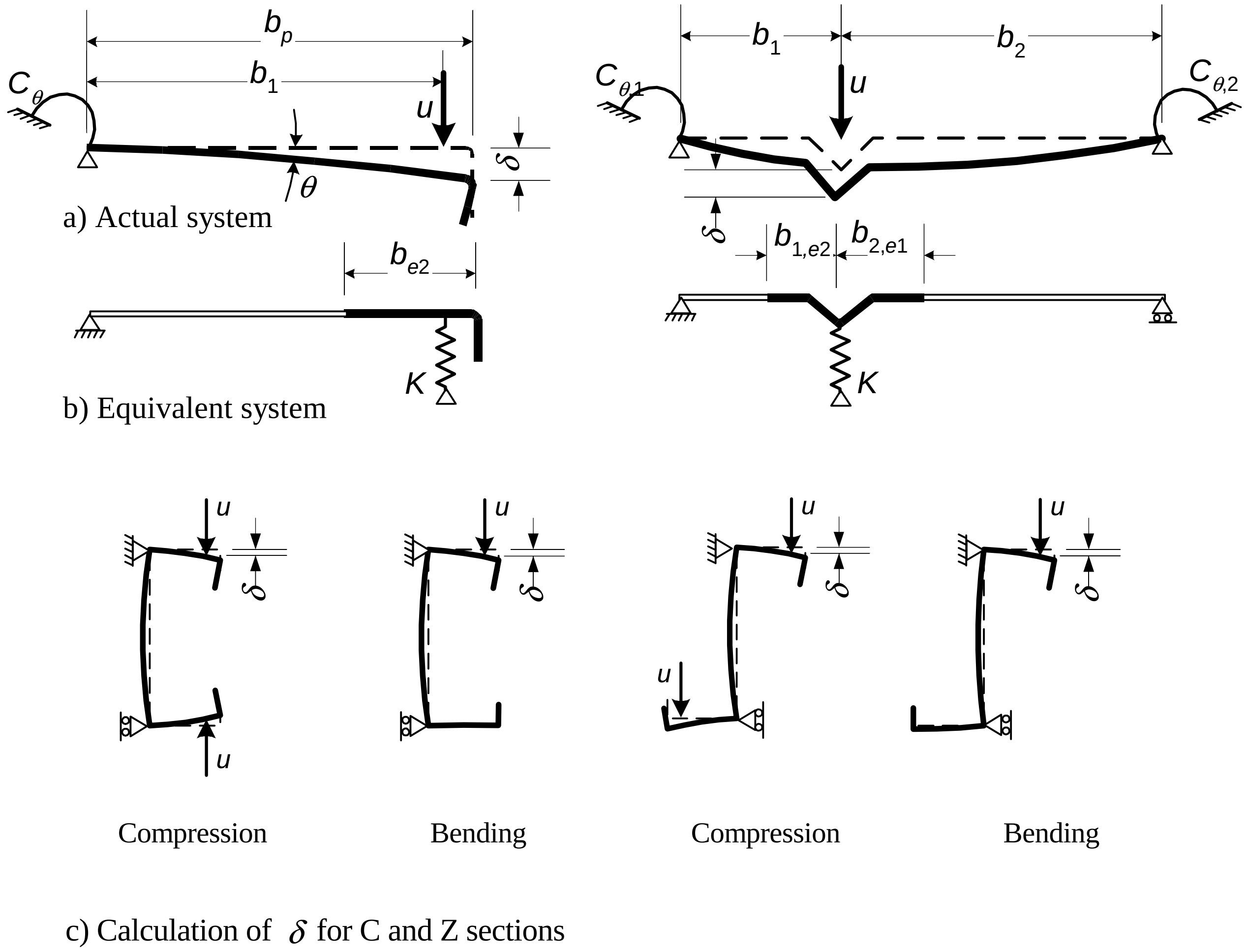


Figure 5.8: Determination of spring stiffness

(3) In determining the values of the rotational spring stiffnesses C_θ, $C_{\theta,1}$ and $C_{\theta,2}$ from the geometry of the cross-section, account should be taken of the possible effects of other stiffeners that exist on the same element, or on any other element of the cross-section that is subject to compression.

(4) For an edge stiffener, the deflection δ should be obtained from:

$$\delta = \theta\, b_\mathrm{p} + \frac{u b_\mathrm{p}^{\,3}}{3} \cdot \frac{12(1-\nu^2)}{E t^3} \qquad \text{... (5.10)}$$

with:

$$\theta = u\, b_\mathrm{p} / C_\theta$$

(5) In the case of the edge stiffeners of lipped C-sections and lipped Z-sections, C_θ should be determined with the unit loads u applied as shown in figure 5.8(c). This results in the following expression for the spring stiffness K_1 for the flange 1:

$$K_1 = \frac{E t^3}{4(1-\nu^2)} \cdot \frac{1}{b_1^2 h_\mathrm{w} + b_1^3 + 0{,}5\, b_1 b_2 h_\mathrm{w} k_\mathrm{f}} \qquad \text{… (5.10b)}$$

where:

b_1 is the distance from the web-to-flange junction to the gravity center of the effective area of the edge stiffener (including effective part b_{e2} of the flange) of flange 1, see figure 5.8(a);

b_2 is the distance from the web-to-flange junction to the gravity center of the effective area of the edge stiffener (including effective part of the flange) of flange 2;

h_w is the web depth;

$k_f = 0$ if flange 2 is in tension (e.g. for beam in bending about the y-y axis);

$k_f = \dfrac{A_{eff\,2}}{A_{eff\,1}}$ if flange 2 is also in compression (e.g. for a beam in axial compression);

$k_f = 1$ for a symmetric section in compression.

A_{eff1} and A_{eff2} is the effective area of the edge stiffener (including effective part b_{e2} of the flange, see figure 5.8(b)) of flange 1 and flange 2 respectively.

(6) For an intermediate stiffener, as a conservative alternative the values of the rotational spring stiffnesses $C_{\theta,1}$ and $C_{\theta,2}$ may be taken as equal to zero, and the deflection δ may be obtained from:

$$\delta = \frac{u b_1^{\,2} b_2^{\,2}}{3(b_1 + b_2)} \cdot \frac{12(1-\nu^2)}{Et^3} \qquad \text{... (5.11)}$$

(7) The reduction factor χ_d for the distortional buckling resistance (flexural buckling of a stiffener) should be obtained from the relative slenderness $\bar{\lambda}_d$ from:

$$\chi_d = 1{,}0 \qquad \text{if } \bar{\lambda}_d \le 0{,}65 \qquad \text{... (5.12a)}$$

$$\chi_d = 1{,}47 - 0{,}723\bar{\lambda}_d \qquad \text{if } 0{,}65 < \bar{\lambda}_d < 1{,}38 \qquad \text{... (5.12b)}$$

$$\chi_d = \frac{0{,}66}{\bar{\lambda}_d} \qquad \text{if } \bar{\lambda}_d \ge 1{,}38 \qquad \text{... (5.12c)}$$

where:

$$\bar{\lambda}_d = \sqrt{f_{yb}/\sigma_{cr,s}} \qquad \text{… (5.12d)}$$

where:

$\sigma_{cr,s}$ is the elastic critical stress for the stiffener(s) from 5.5.3.2, 5.5.3.3 or 5.5.3.4.

(8) Alternatively, the elastic critical buckling stress $\sigma_{cr,s}$ may be obtained from elastic first order buckling analysis using numerical methods (see 5.5.1(11)).

(9) In the case of a plane element with an edge and intermediate stiffener(s) in the absence of a more accurate method the effect of the intermediate stiffener(s) may be neglected.

5.5.3.2 Plane elements with edge stiffeners

(1) The following ~~op~~procedure is applicable to an edge stiffener if the requirements in 5.2 are met and the angle between the stiffener and the plane element is between 45° and 135°.

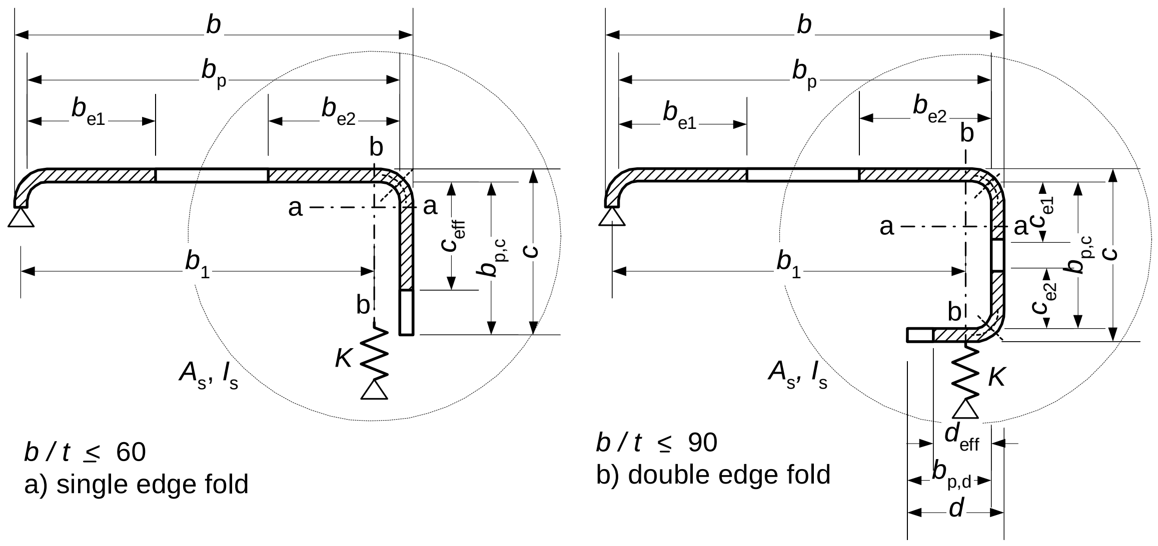


Figure 5.9: Edge stiffeners

(2) The cross-section of an edge stiffener should be taken as comprising the effective portions of the stiffener, element c or elements c and d as shown in figure 5.9, plus the adjacent effective portion of the plane element b_p.

(3) The procedure, which is illustrated in figure 5.10, should be carried out in steps as follows:

- **Step 1**: Obtain an initial effective cross-section for the stiffener using effective widths determined by assuming that the stiffener gives full restraint and that $\sigma_{com,Ed} = f_{yb}/\gamma_{M0}$, see (3) to (5);

- **Step 2**: Use the initial effective cross-section of the stiffener to determine the reduction factor for distortional buckling (flexural buckling of a stiffener), allowing for the effects of the continuous spring restraint, see (6), ~~and~~ (7) and (8);

- **Step 3**: Optionally iterate to refine the value of the reduction factor for buckling of the stiffener, see (9~~8~~) and (10~~9~~).

(4) Initial values of the effective widths b_{e1} and b_{e2} shown in figure 5.9 should be determined from clause 5.5.2 by assuming that the plane element b_p is doubly supported, see table 5.3.

(5) Initial values of the effective widths c_{eff} and d_{eff} shown in figure 5.9 should be obtained as follows:

a) for a single edge fold stiffener:

$$c_{\mathrm{eff}} = \rho\, b_{\mathrm{p,c}} \quad \text{... (5.13a)}$$

with ρ obtained from 5.5.2, except using a value of the buckling factor k_σ given by the following:

- if $b_{\mathrm{p,c}}/b_{\mathrm{p}} \leq 0{,}35$:

$$k_\sigma = 0{,}5 \quad \text{... (5.13b)}$$

- if $0{,}35 < b_{\mathrm{p,c}}/b_{\mathrm{p}} \leq 0{,}6$:

$$k_\sigma = 0{,}5 + 0{,}83\sqrt[3]{\left(b_{\mathrm{p,c}} / \mathrm{b_p} - 0{,}35\right)^2} \quad \text{...(5.13c)}$$

b) for a double edge fold stiffener:

$$c_{\mathrm{eff}} = \rho\, b_{\mathrm{p,c}} \quad \text{... (5.13d)}$$

with ρ obtained from 5.5.2 with a buckling factor k_σ for a doubly supported element from table 5.3;

$$d_{\mathrm{eff}} = \rho\, b_{\mathrm{p,d}} \quad \text{... (5.13e)}$$

with ρ obtained from 5.5.2 with a buckling factor k_σ for an outstand element from table 5.4.

(6) The effective cross-sectional area of the edge stiffener A_{s} should be obtained from:

$$A_{\mathrm{s}} = t\,(b_{\mathrm{e2}} + c_{\mathrm{eff}}) \qquad \text{or} \qquad \text{... (5.14a)}$$

$$A_{\mathrm{s}} = t\,(b_{\mathrm{e2}} + c_{\mathrm{e1}} + c_{\mathrm{e2}} + d_{\mathrm{eff}}) \quad \text{...(5.14b)}$$

respectively.

NOTE: The rounded corners should be taken into account if needed, see 5.1.

(7) The elastic critical buckling stress $\sigma_{\mathrm{cr,s}}$ for an edge stiffener should be obtained from:

$$\sigma_{\mathrm{cr,s}} = \frac{2\sqrt{K\,E\,I_{\mathrm{s}}}}{\mathrm{A_s}} \quad \text{... (5.15)}$$

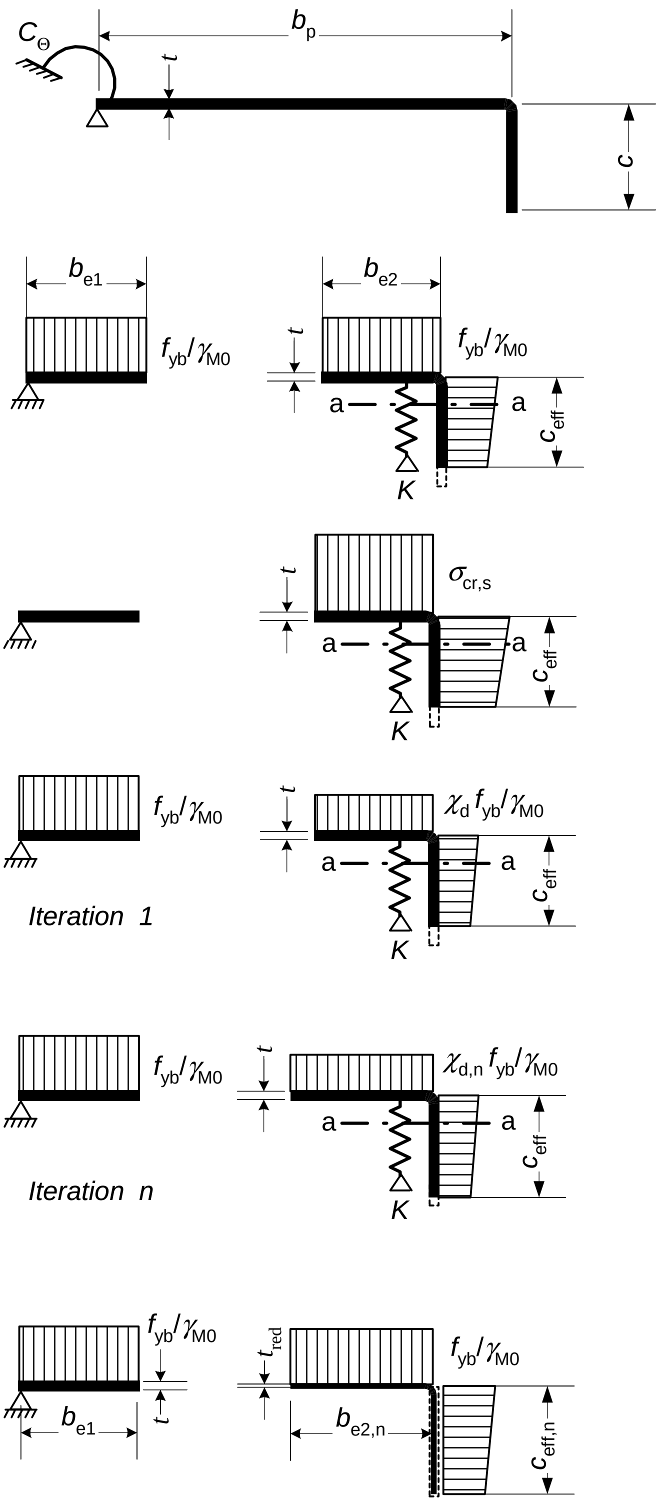
where:

K is the spring stiffness per unit length, see 5.5.3.1(2).

I_{s} is the effective second moment of area of the stiffener, taken as that of its effective area A_{s} about the centroidal axis a - a of its effective cross-section, see figure 5.9.

(8) Alternatively, the elastic critical buckling stress $\sigma_{\mathrm{cr,s}}$ may be obtained from elastic first order buckling analyses using numerical methods, see 5.5.1(8).

(9) The reduction factor χ_{d} for the distortional buckling (flexural buckling of a stiffener) resistance of an edge stiffener should be obtained from the value of $\sigma_{\mathrm{cr,s}}$ using the method given in 5.5.3.1(7).



a) Gross cross-section and boundary conditions

b) **Step 1**: Effective cross-section for $K = \infty$ based on $\sigma_{com,Ed} = f_{yb}/\gamma_{M0}$

c) **Step 2**: Elastic critical stress $\sigma_{cr,s}$ for effective area of stiffener A_s from step 1

d) Reduced strength $\chi_d f_{yb}/\gamma_{M0}$ for effective area of stiffener A_s, with reduction factor χ_d based on $\sigma_{cr,s}$

e) **Step 3**: Optionally repeat step 1 by calculating the effective width with a reduced compressive stress $\sigma_{com,Ed,i} = \chi_d f_{yb} / \gamma_{M0}$ with χ_d from previous iteration, continuing until $\chi_{d,n} \approx \chi_{d,(n-1)}$ but $\chi_{d,n} \leq \chi_{d,(n-1)}$.

f) Adopt an effective cross-section with b_{e2}, c_{eff} and reduced thickness t_{red} corresponding to $\chi_{d,n}$

Figure 5.10: Compression resistance of a flange with an edge stiffener

(10)If $\chi_d < 1$ it may be refined iteratively, starting the iteration with modified values of ρ obtained using 5.5.2(5) with $\sigma_{com,Ed,i}$ equal to $\chi_d f_{yb}/\gamma_{M0}$, so that:

$$\overline{\lambda}_{p,red} = \overline{\lambda}_p \sqrt{\chi_d} \qquad \text{... (5.16)}$$

(11)The reduced effective area of the stiffener $A_{s,red}$ allowing for flexural buckling should be taken as:

$$A_{s,red} = \chi_d A_s \frac{f_{yb}/\gamma_{M0}}{\sigma_{com,Ed}} \quad \text{but } A_{s,red} \leq A_s \qquad \text{... (5.17)}$$

where

$\sigma_{com,Ed}$ is compressive stress at the centreline of the stiffener calculated on the basis of the effective cross-section.

(12)In determining effective section properties, the reduced effective area $A_{s,red}$ should be represented by using a reduced thickness $t_{red} = t\, A_{s,red} / A_s$ for all the elements included in A_s.

5.5.3.3 Plane elements with intermediate stiffeners

(1) The following procedure is applicable to one or two equal in~~m~~termediate stiffeners formed by grooves or bends provided that all plane elements are calculated accor~~r~~ding to 5.5.2.

(2) The cross-section of an intermediate stiffener should be taken as comprising the stiffener itself plus the adjacent effective portions of the adjacent plane elements $b_{p,1}$ and $b_{p,2}$ shown in figure 5.11.

(3) The procedure, which is illustrated in figure 5.12, should be carried out in steps as follows:

- **Step 1**: Obtain an initial effective cross-section for the stiffener using effective widths determined by assuming that the stiffener gives full restraint and that $\sigma_{com,Ed} = f_{yb}/\gamma_{M0}$, see (4~~3~~) and (5~~4~~);

- **Step 2**: Use the initial effective cross-section of the stiffener to determine the reduction factor for distortional buckling (flexural buckling of an intermediate stiffener), allowing for the effects of the continuous spring restraint, see (6~~5~~), (7) and (8~~6~~);

- **Step 3**: Optionally iterate to refine the value of the reduction factor for buckling of the stiffener, see (9~~7~~) and (10~~8~~).

(4) Initial values of the effective widths $b_{1,e2}$ and $b_{2,e1}$ shown in figure 5.11 should be determined from 5.5.2 by assuming that the plane elements $b_{p,1}$ and $b_{p,2}$ are doubly supported, see table 5.3.

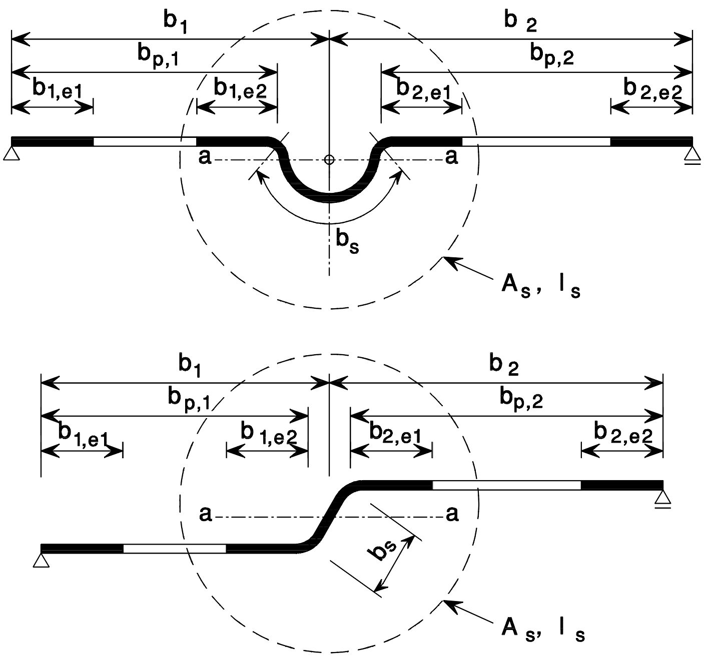


Figure 5.11: Intermediate stiffeners

(5) The effective cross-sectional area of an intermediate stiffener A_s should be obtained from:

$$A_s = t\,(b_{1,e2} + b_{2,e1} + b_s) \qquad \text{... (5.18)}$$

in which the stiffener width b_s is as shown in figure 5.11.

NOTE: The rounded corners should be taken into account if needed, see 5.1.

(6) The critical buckling stress $\sigma_{cr,s}$ for an intermediate stiffener should be obtained from:

$$\sigma_{cr,s} = \frac{2\sqrt{KEI_s}}{A_s} \qquad \text{... (5.19)}$$

where:

K is the spring stiffness per unit length, see 5.5.3.1(2).

I_s is the effective second moment of area of the stiffener, taken as that of its effective area A_s about the centroidal axis a - a of its effective cross-section, see figure 5.11.

(7) Alternatively, the elastic critical buckling stress $\sigma_{cr,s}$ may be obtained from elastic first order buckling analyses using numerical methods, see 5.5.1(11).

(8) The reduction factor χ_d for the distortional buckling resistance (flexural buckling of an intermediate stiffener) should be obtained from the value of $\sigma_{cr,s}$ using the method given in 5.5.3.1(7).

(9) If $\chi_d < 1$ it may optionally be refined iteratively, starting the iteration with modified values of ρ obtained using 5.5.2(5) with $\sigma_{com,Ed,i}$ equal to $\chi_d f_{yb} / \gamma_{M0}$, so that:

$$\bar{\lambda}_{p,red} = \bar{\lambda}_p \sqrt{\chi_d} \qquad \text{... (5.20)}$$

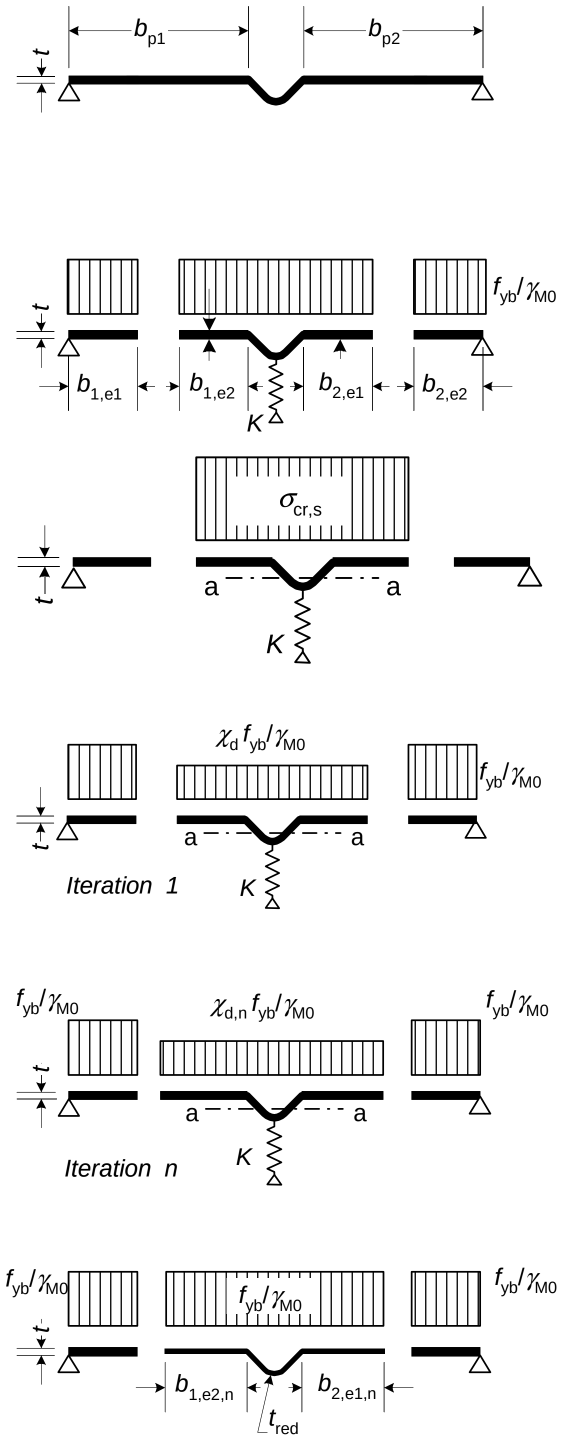
(10)The reduced effective area of the stiffener $A_{s,red}$ allowing for distortional buckling (flexural buckling of a stiffener) should be taken as:

$$A_{s,red} = \chi_d A_s \frac{f_{yb}/\gamma_{M0}}{\sigma_{com,Ed}} \quad \text{but } A_{s,red} \leq A_s \qquad \text{... (5.21)}$$

where

$\sigma_{com,Ed}$ is compressive stress at the centreline of the stiffener calculated on the basis of the effective cross-section.

(11)In determining effective section properties, the reduced effective area $A_{s,red}$ should be represented by using a reduced thickness $t_{red} = t\, A_{s,red} / A_s$ for all the elements included in A_s .



a) Gross cross-section and boundary conditions

b) **Step 1**: Effective cross-section for $K = \infty$ based on $\sigma_{com,Ed} = f_{yb} / \gamma_{M0}$

c) **Step 2**: Elastic critical stress $\sigma_{cr,s}$ for effective area of stiffener A_s from step 1

d) Reduced strength $\chi_d f_{yb} / \gamma_{M0}$ for effective area of stiffener A_s, with reduction factor χ_d based on $\sigma_{cr,s}$

e) **Step 3**: Optionally repeat step 1 by calculating the effective width with a reduced compressive stress $\sigma_{com,Ed,i} = \chi_d f_{yb} / \gamma_{M0}$ with χ_d from previous iteration, continuing until $\chi_{d,n} \approx \chi_{d,(n-1)}$ but $\chi_{d,n} \leq \chi_{d,(n-1)}$.

f) Adopt an effective cross-section with $b_{1,e2}$, $b_{2,e1}$ and reduced thickness t_{red} corresponding to $\chi_{d,n}$

Figure 5.12: Compression resistance of a flange with an intermediate stiffener

5.5.3.4 Trapezoidal sheeting profiles with intermediate stiffeners

5.5.3.4.1 General

(1) This sub-clause 5.5.3.4 should be used for trapezoidal profiled sheets, in association with 5.5.3.3 for flanges with intermediate flange stiffeners and 5.5.3.3 for webs with intermediate stiffeners.

(2) Interaction between the buckling of intermediate flange stiffeners and intermediate web stiffeners should also be taken into account using the method given in 5.5.3.4.4.

5.5.3.4.2 Flanges with intermediate stiffeners

(1) If it is subject to uniform compression, the effective cross-section of a flange with intermediate stiffeners should be assumed to consist of the reduced effective areas $A_{s,red}$ ~~and~~ including two strips of width $0{,}5b_{eff}$ (or $15\,t$, see figure 5.13) adjacent to the stiffener.

(2) For one central flange stiffener, the elastic critical buckling stress $\sigma_{cr,s}$ should be obtained from:

$$\sigma_{cr,s} = \frac{4{,}2\,k_w E}{A_s}\sqrt{\frac{I_s\,t^3}{4\,b_p^2\,(2\,b_p + 3\,b_s)}} \qquad \text{... (5.22)}$$

where:

b_p is the notional flat width of plane element shown in figure 5.13;

b_s is the stiffener width, measured around the perimeter of the stiffener, see figure 5.13;

A_s, I_s are the cross-section area and the second moment of area of the stiffener cross-section according to figure 5.13;

k_w is a coefficient that allows for partial rotational restraint of the stiffened flange by the webs or other adjacent elements, see (5) and (6). For the calculation of the effective cross-section in axial compression the value $k_w = 1{,}0$.

The equation 5.22 may be used for wide grooves provided that flat part of the stiffener is reduced due to local buckling and b_p in the equation 5.22 is replaced by the larger of b_p and $0{,}25(3b_p+b_r)$, see figure 5.13. Similar method is valid for flange with two or more wide grooves.

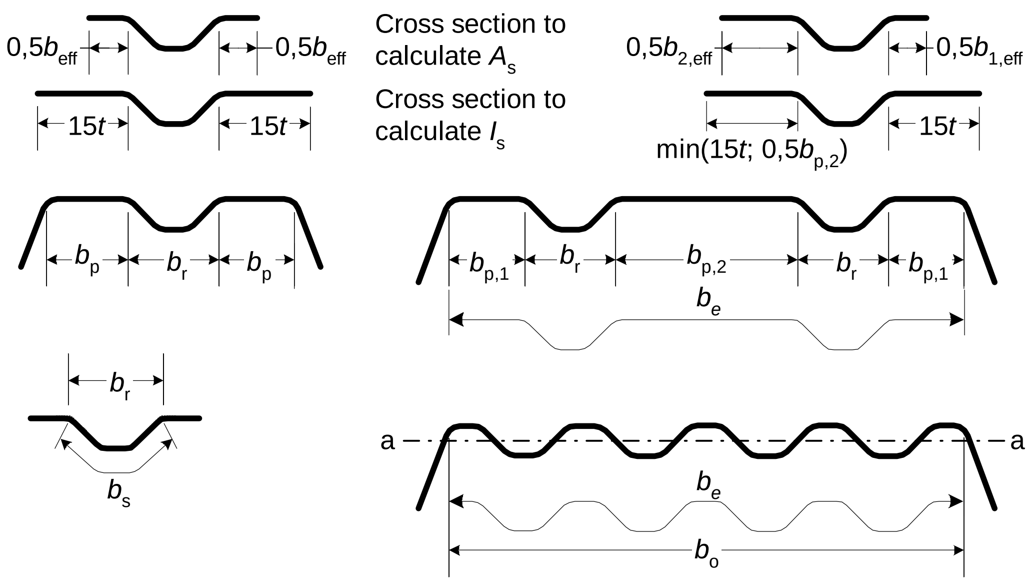


Figure 5.13: Compression flange with one, two or multiple stiffeners

(3) For two symmetrically placed flange stiffeners, the elastic critical buckling stress $\sigma_{cr,s}$ should be obtained from:

$$\sigma_{cr,s} = \frac{4{,}2\ k_w\ E}{A_s}\sqrt{\frac{I_s\ t^3}{8\ b_1^2\ (3\ b_e - 4\ b_1)}} \qquad \text{... (5.23a)}$$

with:

$b_e = 2b_{p,1} + b_{p,2} + 2b_s$

$b_1 = b_{p,1} + 0{,}5\, b_r$

where:

$b_{p,1}$	is the notional flat width of an outer plane element, as shown in figure 5.13;
$b_{p,2}$	is the notional flat width of the central plane element, as shown in figure 5.13;
b_r	is the overall width of a stiffener, see figure 5.13;
A_s, I_s	are the cross-section area and the second moment of area of the stiffener cross-section according to figure 5.13.

(4) For a multiple stiffened flange (three or more equal stiffeners) the effective area of the *entire flange* is

$$A_{eff} = \rho b_e t \qquad \text{…(5.23b)}$$

where ρ is the reduction factor according to ~~expression (5.4b)~~ EN 1993-1-5, Annex E for the slenderness $\bar{\lambda}_p$ based on the elastic buckling stress

$$\sigma_{cr,s} = 1{,}8E\sqrt{\frac{I_s\, t}{b_o^2 b_e^3}} + 3{,}6\frac{Et^2}{b_o^2} \qquad \text{…(5.23c)}$$

$$\cancel{\sigma_{cr,s} = 1{,}8E\sqrt{\frac{I_s\, t}{b_o^3 b_e^2}} + 3{,}6\frac{Et^2}{b_o^2}}$$

where:

I_s	is the sum of the second moment of area of the stiffeners about the centroidal axis a-a, neglecting the thickness terms $bt^3/12$;
b_o	is the width of the flange as shown in 5.13;
b_e	is the developed width of the flange as shown in figure 5.13.

(5) The value of k_w may be calculated from the compression flange buckling wavelength l_b as follows:

- if $l_b / s_w \geq 2$:

$$k_w = k_{wo} \qquad \text{...(5.24a)}$$

- if $l_b / s_w < 2$:

$$k_w = k_{wo} - (k_{wo} - 1)\left[\frac{2l_b}{s_w} - \left(\frac{l_b}{s_w}\right)^2\right] \qquad \text{...(5.24b)}$$

where:

s_w	is the slant height of the web, see figure 5.3(c).

(6) Alternatively, the rotational restraint coefficient k_w may conservatively be taken as equal to 1,0 corresponding to a pin-jointed condition.

(7) The values of l_b and k_{wo} may be determined from the following:

- for a compression flange with one intermediate stiffener:

$$l_b = 3{,}07 \sqrt[4]{\frac{I_s \, b_p^2 \left(2\, b_p + 3\, b_s \right)}{t^3}} \qquad \text{... (5.25)}$$

$$k_{wo} = \sqrt{\frac{s_w + 2\, b_d}{s_w + 0{,}5\, b_d}} \qquad \text{... (5.26)}$$

with:

$b_d = 2b_p + b_s$

- for a compression flange with two ~~or three~~ intermediate stiffeners:

$$l_b = 3{,}65 \sqrt[4]{I_s \, b_1^2 \left(3\, b_e - 4\, b_1 \right) / t^3} \qquad \text{... (5.27)}$$

$$k_{wo} = \sqrt{\frac{\left(2\, b_e + s_w \right)\left(3\, b_e - 4\, b_1 \right)}{b_1 \left(4\, b_e - 6\, b_1 \right) + s_w \left(3\, b_e - 4\, b_1 \right)}} \qquad \text{... (5.28)}$$

(8) The reduced effective area of the stiffener $A_{s,red}$ allowing for distortional buckling (flexural buckling of an intermediate stiffener) should be taken as:

$$A_{s.red} = \chi_d \, A_s \frac{f_{yb} / \gamma_{M0}}{\sigma_{com,ser}} \qquad \text{but } A_{s,red} \leq A_s \qquad \text{... (5.29)}$$

(9) If the webs are unstiffened, the reduction factor χ_d should be obtained directly from $\sigma_{cr,s}$ using the method given in 5.5.3.1(7).

(10)If the webs are also stiffened, the reduction factor χ_d should be obtained using the method given in 5.5.3.1(7), but with the modified elastic critical stress $\sigma_{cr,mod}$ given in 5.5.3.4.4.

(11)In determining effective section properties, the reduced effective area $A_{s,red}$ should be represented by using a reduced thickness $t_{red} = t\, A_{s,red} / A_s$ for all the elements included in A_s .

5.5.3.4.3 Webs with up to two intermediate stiffeners

(1) The effective cross-section of the compression zone of a web (or other element of a cross-section that is subject to stress gradient) should be assumed to consist of the reduced effective areas $A_{s,red}$ of up to two intermediate stiffeners, a strip adjacent to the compression flange and a strip adjacent to the centroidal axis of the effective cross-section, see figure 5.14.

(2) The effective cross-section of a web as shown in figure 5.14 should be taken to include:

a) a strip of width $s_{eff,1}$ adjacent to the compression flange;

b) the reduced effective area $A_{s,red}$ of each web stiffener, up to a maximum of two;

c) a strip of width $s_{eff,n}$ adjacent to the effective centroidal axis;

d) the part of the web in tension.

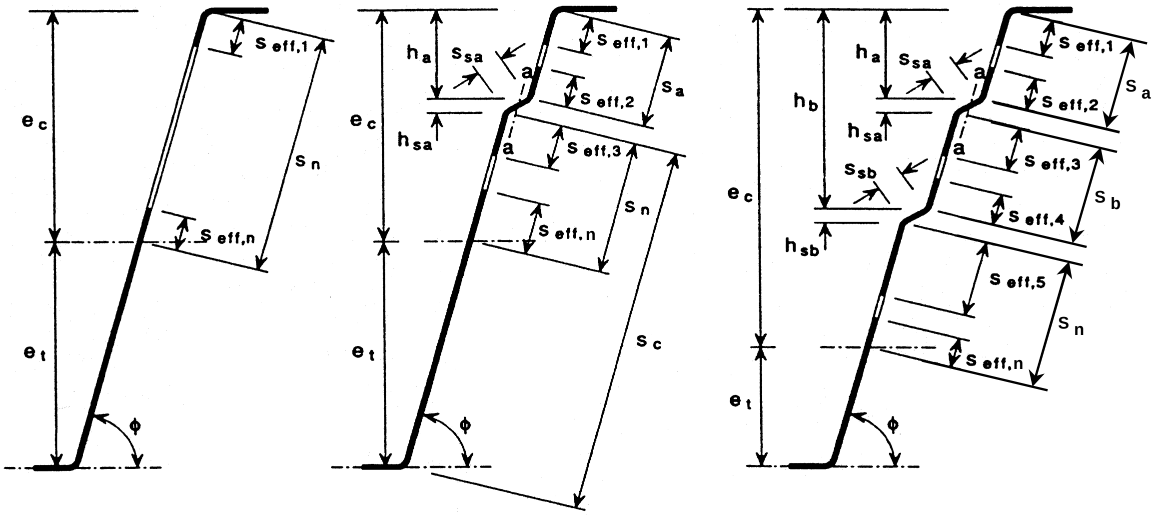


Figure 5.14: Effective cross-sections of webs of trapezoidal profiled sheets

(3) The effective areas of the stiffeners should be obtained from the following:

- for a single stiffener, or for the stiffener closer to the compression flange:

$$A_{sa} = t\,(s_{eff,2} + s_{eff,3} + s_{sa}) \qquad \text{... (5.30)}$$

- for a second stiffener:

$$A_{sb} = t\,(s_{eff,4} + s_{eff,5} + s_{sb}) \qquad \text{... (5.31)}$$

in which the dimensions $s_{eff,1}$ to $s_{eff,n}$ and s_{sa} and s_{sb} are as shown in figure 5.14.

(4) Initially the location of the effective centroidal axis should be based on the effective cross-sections of the flanges but the gross cross-sections of the webs. In this case the basic effective width $s_{eff,0}$ should be obtained from:

$$s_{eff,0} = 0{,}76\,t\,\sqrt{E\,/\left(\gamma_{M0}\,\sigma_{com,Ed}\right)} \qquad \text{... (5.32)}$$

where:

$\sigma_{com,Ed}$ is the stress in the compression flange when the cross-section resistance is reached.

(5) If the web is not fully effective, the dimensions $s_{eff,1}$ to $s_{eff,n}$ should be determined as follows:

$$s_{eff,1} = s_{eff,0} \qquad \text{... (5.33a)}$$

$$s_{eff,2} = (1 + 0{,}5h_a / e_c)\,s_{eff,0} \qquad \text{... (5.33b)}$$

$$s_{eff,3} = [1 + 0{,}5(h_a + h_{sa})/ e_c]\,s_{eff,0} \qquad \text{... (5.33c)}$$

$$s_{eff,4} = (1 + 0{,}5h_b / e_c)\,s_{eff,0} \qquad \text{... (5.33d)}$$

$$s_{eff,5} = [1 + 0{,}5(h_b + h_{sb})/ e_c]\,s_{eff,0} \qquad \text{... (5.33e)}$$

$$s_{eff,n} = 1{,}5 s_{eff,0} \qquad \text{... (5.33f)}$$

where:

e_c is the distance from the effective centroidal axis to the system line of the compression flange, see figure 5.14;

and the dimensions h_a, h_b, h_{sa} and h_{sb} are as shown in figure 5.14.

(6) The dimensions $s_{\mathrm{eff},1}$ to $s_{\mathrm{eff,n}}$ should initially be determined from (5) and then revised if the relevant plane element is fully effective, using the following:

- in an unstiffened web, if $s_{\mathrm{eff},1} + s_{\mathrm{eff,n}} \geq s_{\mathrm{n}}$ the entire web is effective, so revise as follows:

$$s_{\mathrm{eff},1} = 0{,}4 s_{\mathrm{n}} \qquad \text{... (5.34a)}$$

$$s_{\mathrm{eff,n}} = 0{,}6 s_{\mathrm{n}} \qquad \text{... (5.34b)}$$

- in stiffened web, if $s_{\mathrm{eff},1} + s_{\mathrm{eff},2} \geq s_{\mathrm{a}}$ the whole of s_{a} is effective, so revise as follows:

$$s_{\mathrm{eff},1} = \frac{s_{\mathrm{a}}}{2 + 0{,}5 h_{\mathrm{a}} / e_{\mathrm{c}}} \qquad \text{... (5.35a)}$$

$$s_{\mathrm{eff},2} = s_{\mathrm{a}} \frac{(1 + 0{,}5 h_{\mathrm{a}} / e_{\mathrm{c}})}{2 + 0{,}5 h_{\mathrm{a}} / e_{\mathrm{c}}} \qquad \text{... (5.35b)}$$

- in a web with one stiffener, if $s_{\mathrm{eff},3} + s_{\mathrm{eff,n}} \geq s_{\mathrm{n}}$ the whole of s_{n} is effective, so revise as follows:

$$s_{\mathrm{eff},3} = s_{\mathrm{n}} \frac{[1 + 0{,}5(h_{\mathrm{a}} + h_{\mathrm{sa}})/e_{\mathrm{c}}]}{2{,}5 + 0{,}5(h_{\mathrm{a}} + h_{\mathrm{sa}})/e_{\mathrm{c}}} \qquad \text{... (5.36a)}$$

$$s_{\mathrm{eff,n}} = \frac{1{,}5 s_{\mathrm{n}}}{2{,}5 + 0{,}5(h_{\mathrm{a}} + h_{\mathrm{sa}})/e_{\mathrm{c}}} \qquad \text{... (5.36b)}$$

- in a web with two stiffeners:
  - if $s_{\mathrm{eff},3} + s_{\mathrm{eff},4} \geq s_{\mathrm{b}}$ the whole of s_{b} is effective, so revise as follows:

$$s_{\mathrm{eff},3} = s_{\mathrm{b}} \frac{1 + 0{,}5(h_{\mathrm{a}} + h_{\mathrm{sa}})/e_{\mathrm{c}}}{2 + 0{,}5(h_{\mathrm{a}} + h_{\mathrm{sa}} + h_{\mathrm{b}})/e_{\mathrm{c}}} \qquad \text{... (5.37a)}$$

$$s_{\mathrm{eff},4} = s_{\mathrm{b}} \frac{1 + 0{,}5 h_{\mathrm{b}} / e_{\mathrm{c}}}{2 + 0{,}5(h_{\mathrm{a}} + h_{\mathrm{sa}} + h_{\mathrm{b}})/e_{\mathrm{c}}} \qquad \text{... (5.37b)}$$

  - if $s_{\mathrm{eff},5} + s_{\mathrm{eff,n}} \geq s_{\mathrm{n}}$ the whole of s_{n} is effective, so revise as follows:

$$s_{\mathrm{eff},5} = s_{\mathrm{n}} \frac{1 + 0{,}5(h_{\mathrm{b}} + h_{\mathrm{sb}})/e_{\mathrm{c}}}{2{,}5 + 0{,}5(h_{\mathrm{b}} + h_{\mathrm{sb}})/e_{\mathrm{c}}} \qquad \text{... (5.38a)}$$

$$s_{\mathrm{eff,n}} = \frac{1{,}5 s_{\mathrm{n}}}{2{,}5 + 0{,}5(h_{\mathrm{b}} + h_{\mathrm{sb}})/e_{\mathrm{c}}} \qquad \text{... (5.38b)}$$

(7) For a single stiffener, or for the stiffener closer to the compression flange in webs with two stiffeners, the elastic critical buckling stress $\sigma_{cr,sa}$ should be determined using:

$$\sigma_{cr,sa} = \frac{1{,}05\ k_f\ E\sqrt{I_s\ t^3\ s_1}}{A_{sa}\ s_2\ (s_1 - s_2)} \qquad \text{... (5.39a)}$$

in which s_1 is given by the following:

- for a single stiffener:

$$s_1 = 0{,}9\,(s_a + s_{sa} + s_c) \qquad \text{... (5.39b)}$$

- for the stiffener closer to the compression flange, in webs with two stiffeners:

$$s_1 = s_a + s_{sa} + s_b + 0{,}5(s_{sb} + s_c) \qquad \text{... (5.39c)}$$

with:

$$s_2 = s_1 - s_a - 0{,}5 s_{sa} \qquad \text{... (5.39d)}$$

where:

k_f is a coefficient that allows for partial rotational restraint of the stiffened web by the flanges;

I_s is the second moment of area of a stiffener cross-section comprising the fold width s_{sa} and two adjacent strips, each of width $s_{eff,1}$, about its own centroidal axis parallel to the plane web elements, see figure 5.15. In calculating I_s the possible difference in slope between the plane web elements on either side of the stiffener may be neglected;.

s_c as defined in Figure 5.14.

(8) In the absence of a more detailed investigation, the rotational restraint coefficient k_f may conservatively be taken as equal to 1,0 corresponding to a pin-jointed condition.

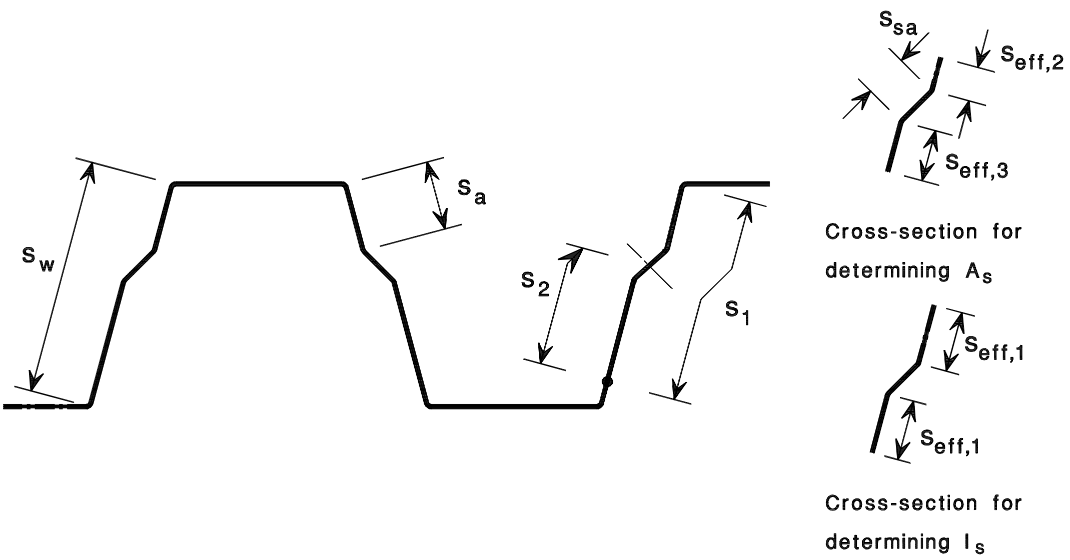


Figure 5.15: Web stiffeners for trapezoidal profiled sheeting

(9) For a single stiffener in compression, or for the stiffener closer to the compression flange in webs with two stiffeners, the reduced effective area $A_{sa,red}$ should be determined from:

$$A_{sa,red} = \frac{\chi_d\ A_{sa}}{1-(h_a + 0{,}5 h_{sa})/e_c} \quad \text{but } A_{sa,red} \le A_{sa} \qquad \text{... (5.40)}$$

(10)If the flanges are unstiffened, the reduction factor χ_d should be obtained directly from $\sigma_{cr,sa}$ using the method given in 5.5.3.1(7).

(11)If the flanges are also stiffened, the reduction factor χ_d should be obtained using the method given in 5.5.3.1(7), but with the modified elastic critical stress $\sigma_{cr,mod}$ given in 5.5.3.4.4.

(12)For a single stiffener in tension, the reduced effective area $A_{sa,red}$ should be taken as equal to A_{sa}.

(13)For webs with two stiffeners, the reduced effective area $A_{sb,red}$ for the second stiffener, should be taken as equal to A_{sb}.

(14)In determining effective section properties, the reduced effective area $A_{sa,red}$ should be represented by using a reduced thickness $t_{red} = \chi_d t$ for all the elements included in A_{sa}.

(15)The effective section properties *of the stiffeners* at serviceability limit states should be based on the design thickness t.

(16)Optionally, the effective section properties may be refined iteratively by basing the location of the effective centroidal axis on the effective cross-sections of the webs determined by the previous iteration and the effective cross-sections of the flanges determined using the reduced thickness t_{red} for all the elements included in the flange stiffener areas A_s. This iteration should be based on an increased basic effective width $s_{eff,0}$ obtained from:

$$s_{eff,0} = 0{,}95\, t \sqrt{\frac{E}{\gamma_{M0}\, \sigma_{com,Ed}}} \qquad \text{... (5.41)}$$

5.5.3.4.4 Sheeting with flange stiffeners and web stiffeners

(1) In the case of sheeting with intermediate stiffeners in the flanges and in the webs, see figure 5.16, interaction between the distorsional buckling (flexural buckling of the flange stiffeners and the web stiffeners) should be allowed for by using a modified elastic critical stress $\sigma_{cr,mod}$ for both types of stiffeners, obtained from:

$$\sigma_{cr,mod} = \frac{\sigma_{cr,s}}{\sqrt[4]{1 + \left[\beta_s \dfrac{\sigma_{cr,s}}{\sigma_{cr,sa}}\right]^4}} \qquad \text{... (5.42)}$$

where:

$\sigma_{cr,s}$ is the elastic critical stress for an intermediate flange stiffener, see 5.5.3.4.2(2) for a flange with a single stiffener or 5.5.3.4.2(3) for a flange with two stiffeners;

$\sigma_{cr,sa}$ is the elastic critical stress for a single web stiffener, or the stiffener closer to the compression flange in webs with two stiffeners, see 5.5.3.4.3(7);

A_s is the effective cross-section area of an intermediate flange stiffener;

A_{sa} is the effective cross-section area of an intermediate web stiffener;

β_s = $1 - (h_a + 0{,}5\, h_{ha}) / e_c$ for a profile in bending;

β_s = 1 for a profile in axial compression.

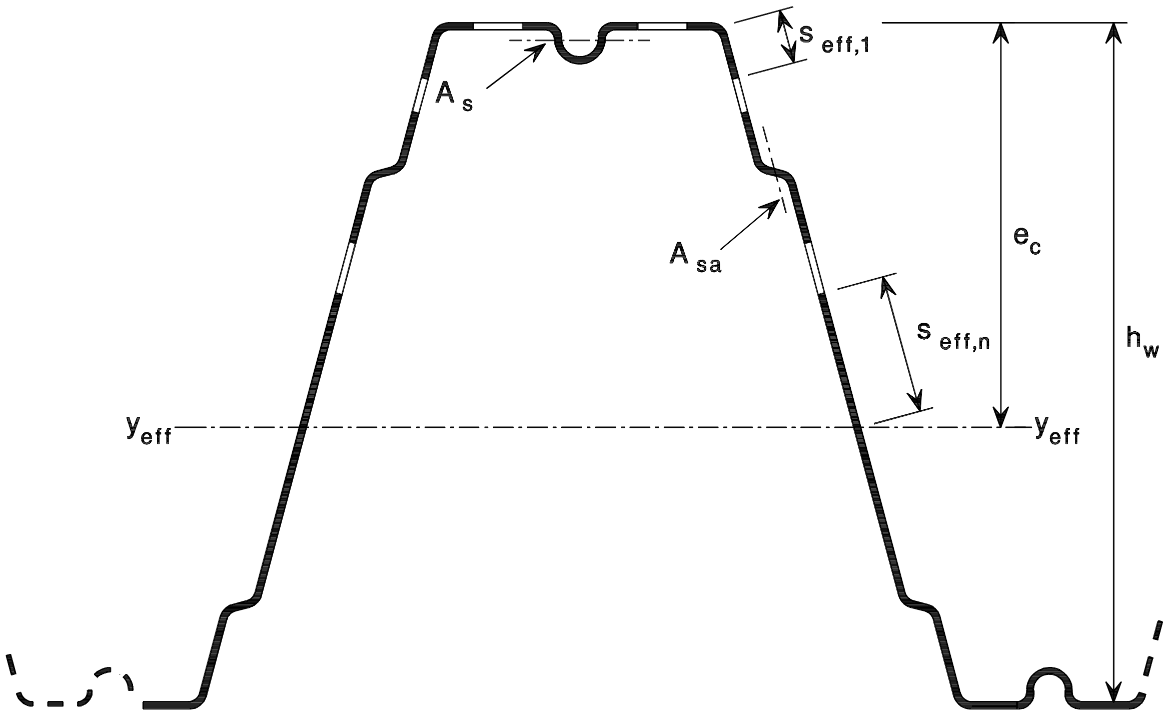


Figure 5.16: Trapezoidal profiled sheeting with flange stiffeners and web stiffeners

5.6 Buckling between fasteners

(1) Buckling between fasteners should be checked for composed elements of plates and mechanical fasteners, see Table 3.3 of EN 1993-1-8.

6 Ultimate limit states

6.1 Resistance of cross-sections

6.1.1 General

(1) Design assisted by testing may be used instead of design by calculation for any of these resistances.

NOTE: Design assisted by testing is particularly likely to be beneficial for cross-sections with relatively high b_p / t ratios, e.g. in relation to inelastic behaviour, web crippling or shear lag.

(2) For design by calculation, the effects of local buckling ~~shall~~ should be taken into account by using effective section properties determined as specified in Section 5.5.

(3) The buckling resistance of members ~~shall~~ should be verified as specified in Section 6.2.

(4) In members with cross-sections that are susceptible to cross-sectional distortion, account ~~shall~~ should be taken of possible lateral buckling of compression flanges and lateral bending of flanges generally, see 5.5, and 10.1.

6.1.2 Axial tension

(1) The design resistance of a cross-section for uniform tension $N_{t,Rd}$ should be determined from:

$$N_{t,Rd} = \frac{f_{ya} A_g}{\gamma_{M0}} \quad \text{but} \quad N_{t,Rd} \leq F_{n,Rd} \qquad \text{... (6.1)}$$

where:

A_g is the gross area of the cross-section;

$F_{n,Rd}$ is the net-section resistance from 8.4 for the appropriate type of mechanical fastener;

f_{ya} is the average yield strength, see 3.2.3.

(2) The design resistance of an angle for uniform tension connected through one leg, or other types of section connected through outstands, should be determined as specified in EN 1993-1-1.

6.1.3 Axial compression

(1) The design resistance of a cross-section for compression $N_{c,Rd}$ should be determined from:

- if the effective area A_{eff} is less than the gross area A_g (section with reduction due to local and/or distortional buckling)

$$N_{c,Rd} = A_{eff} f_{yb} / \gamma_{M0} \quad \ldots(6.2)$$

- if the effective area A_{eff} is equal to the gross area A_g (section with no reduction due to local or distortional buckling)

$$N_{c,Rd} = A_g \left(f_{yb} + (f_{ya} - f_{yb}) 4(1 - \lambda / \lambda_{el}) \right) / \gamma_{M0} \text{ but not more than } A_g f_{ya} / \gamma_{M0} \quad \ldots(6.3)$$

where

A_{eff} is the effective area of the cross-section, obtained from Section 5.5 by assuming a uniform compressive stress equal to f_{yb} / γ_{M0};

f_{ya} is the average yield strength, see 3.2.2;

f_{yb} is the basic yield strength.;

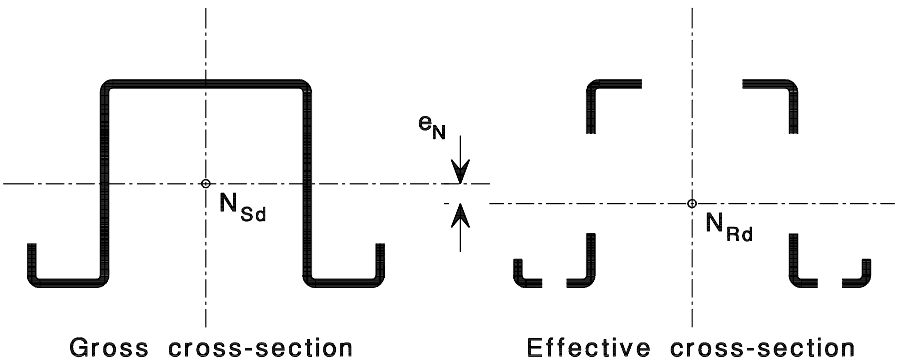
λ is the slenderness of the element which correspond to the largest value of λ / λ_{el};

For plane elements $\lambda = \bar{\lambda}_p$ and $\lambda_{el} = 0{,}673$, see 5.5.2;

For stiffened elements $\lambda = \bar{\lambda}_d$ and $\lambda_{el} = 0{,}65$, see 5.5.3.

(2) The internal normal force in a member should be taken as acting at the centroid of its gross cross-section. This is a conservative assumption, but may be used without further analysis. Further analysis may give a more realistic situation of the internal forces for instance in case of uniformly building-up of normal force in the compression element.

(3) The design compression resistance of a cross-section refers to the axial load acting ~~in~~ at the centroid of its effective cross-section. If this does not coincide with the centroid of its gross cross-section, the shift e_N of the centroidal axes (see figure 6.1) should be taken into account, using the method given in 6.1.9. When the shift of the neutral axis gives a favourable result in the stress/unity check, then that shift should be neglected only if the shift has been calculated at yield strength and not with the actual compressive stresses.



6.1.4 Bending moment

6.1.4.1 Elastic and elastic-plastic resistance with yielding at the compressed flange

(1) The design moment resistance of a cross-section for bending about one principal axis $M_{c,Rd}$ is determined as follows (see figure 6.2):

- if the effective section modulus W_{eff} is less than the gross elastic section modulus W_{el}

$$M_{c,Rd} = W_{eff}\, f_{yb} / \gamma_{M0} \qquad ...(6.4)$$

- if the effective section modulus W_{eff} is equal to the gross elastic section modulus W_{el}

$$M_{c,Rd} = f_{yb}\left(W_{el} + (W_{pl} - W_{el})4(1 - \lambda / \lambda_{el})\right) / \gamma_{M0} \text{ but not more than } W_{pl}\, f_{yb} / \gamma_{M0} \qquad ...(6.5)$$

where

λ is the slenderness of the element which correspond to the largest value of λ / λ_{el};

For double supported plane elements $\lambda = \overline{\lambda}_p$ and $\lambda_{el} = 0,5 + \sqrt{0,25 - 0,055(3 + \psi)}$ where ψ is the stress ratio, see 5.5.2;

For outstand elements $\lambda = \overline{\lambda}_p$ and $\lambda_{el} = 0,673$, see 5.5.2;

For stiffened elements $\lambda = \overline{\lambda}_d$ and $\lambda_{el} = 0,65$, see 5.5.3.

The resulting bending moment resistance as a function of a decisive element is illustrated in the figure 6.2.

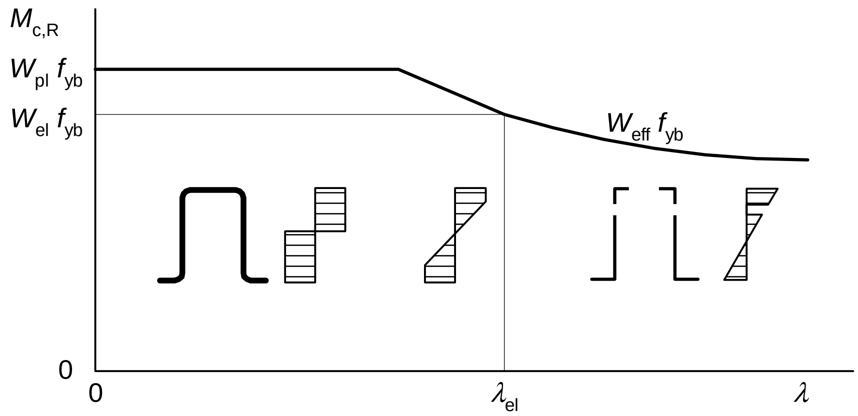


Figure 6.2: Bending moment resistance as a function of slenderness

(2) Expression (6.5) is applicable provided that the following conditions are satisfied:

a) Bending moment is applied only about one principal axes of the cross-section;

b) The member is not subject to torsion or to torsional, torsional flexural or lateral-torsional or distortional buckling;

c) The angle ϕ between the web (see figure 6.5) and the flange is larger than 60°.

(3) If (2) is not fulfilled the following expression may be used:

$$M_{c,Rd} = W_{el}\, f_{ya} / \gamma_{M0} \qquad …(6.6)$$

(4) The effective section modulus W_{eff} should be based on an effective cross-section that is subject only to bending moment about the relevant principal axis, with a maximum stress $\sigma_{max,Ed}$ equal to f_{yb} / γ_{M0}, allowing for the effects of local and distortional buckling as specified in Section 5.5. Where shear lag is relevant, allowance should also be made for its effects.

(5) The stress ratio $\psi = \sigma_2 / \sigma_1$ used to determine the effective portions of the web may be obtained by using the effective area of the compression flange but the gross area of the web, see figure 6.3.

(6) If yielding occurs first at the compression edge of the cross-section, unless the conditions given in 6.1.4.2 are met the value of W_{eff} should be based on a linear distribution of stress across the cross-section.

(7) For biaxial bending the following criterion may be used:

$$\frac{M_{y,Ed}}{M_{cy,Rd}} + \frac{M_{z,Ed}}{M_{cz,Rd}} \leq 1 \qquad \text{... (6.7)}$$

where:

$M_{y,Ed}$ is the applied bending moment about the major main axis;

$M_{z,Ed}$ is the applied bending moment about the minor main axis;

$M_{cy,Rd}$ is the resistance of the cross-section if subject only to moment about the main y – y axis;

$M_{cz,Rd}$ is the resistance of the cross-section if subject only to moment about the main z – z axis.

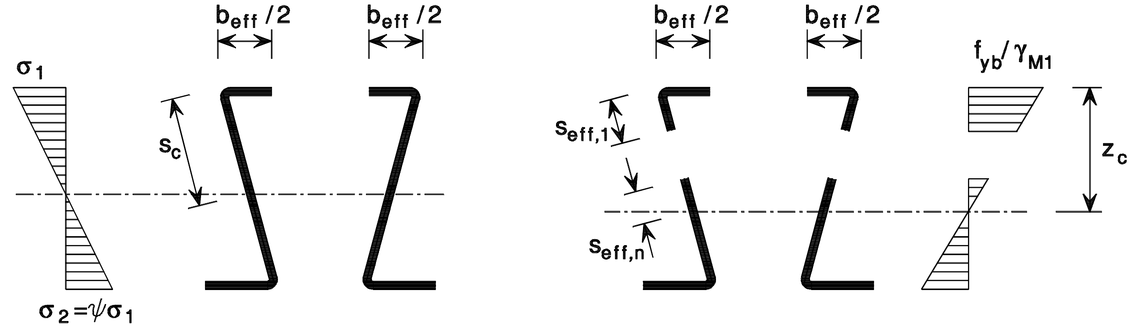


Figure 6.3: Effective cross-section for resistance to bending moments

(8) If redistribution of bending moments is assumed in the global analysis, it should be demonstrated from the results of tests in accordance with Section 9 that the provisions given in 7.2 are satisfied.

6.1.4.2 Elastic and elastic-plastic resistance with yielding at the tension flange only

(1) Provided that bending moment is applied only about one principal axis of the cross-section, and provided that yielding occurs first at the tension edge, plastic reserves in the tension zone may be utilised without any strain limit until the maximum compressive stress $\sigma_{com,Ed}$ reaches f_{yb} / γ_{M0}. In this clause only the bending case is considered. For axial load and bending the clause 6.1.8 or 6.1.9 should be applied.

(2) In this case, the effective partially plastic section modulus $W_{pp,eff}$ should be based on a stress distribution that is bilinear in the tension zone but linear in the compression zone.

(3) In the absence of a more detailed analysis, the effective width b_{eff} of an element subject to stress gradient may be obtained using 5.5.2 by basing b_c on the bilinear stress distribution (see figure 6.4), by assuming $\psi = -1$.

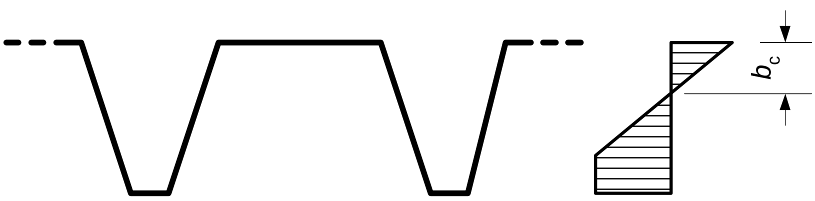


Figure 6.4: Measure b_c for determination of effective width

(4) If redistribution of bending moments is assumed in the global analysis, it should be demonstrated from the results of tests in accordance with Section 9 that the provisions given in 7.2 are satisfied.

6.1.4.3 Effects of shear lag

(1) The effects of shear lag ~~shall~~ should be taken into account according to EN 1993-1-5.

6.1.5 Shear force

(1) The shear resistance $V_{\mathrm{b,Rd}}$ should be determined from:

$$V_{\mathrm{b,Rd}} = \frac{\dfrac{h_{\mathrm{w}}}{\sin\phi} t f_{\mathrm{bv}}}{\gamma_{\mathrm{M0}}} \qquad \text{... (6.8)}$$

where:

f_{bv} is the shear strength considering buckli~~o~~ng according to Table 6.1;

h_{w} is the web height between the midlines of the flanges, see figure 5.3(c);

ϕ is the slope of the web relative to the flanges, see figure 6.5.

Table 6.1: Shear buckling strength f_{bv}

Relative web slenderness	Web without stiffening at the support	Web with stiffening at the support [1)]
$\bar{\lambda}_{\mathrm{w}} \leq 0{,}83$ ~~$\bar{\lambda}_{\mathrm{w}} \geq 0{,}83$~~	$0{,}58\ f_{\mathrm{yb}}$	$0{,}58\ f_{\mathrm{yb}}$
$0{,}83 < \bar{\lambda}_{\mathrm{w}} < 1{,}40$	$0{,}48 f_{\mathrm{yb}} / \bar{\lambda}_{\mathrm{w}}$	$0{,}48 f_{\mathrm{yb}} / \bar{\lambda}_{\mathrm{w}}$
$\bar{\lambda}_{\mathrm{w}} \geq 1{,}40$	$0{,}67 f_{\mathrm{yb}} / \bar{\lambda}_{\mathrm{w}}^2$	$0{,}48 f_{\mathrm{yb}} / \bar{\lambda}_{\mathrm{w}}$

[1)] Stiffening at the support, such as cleats, arranged to prevent distortion of the web and designed to resist the support reaction.

(2) The relative web slenderness $\bar{\lambda}_{\mathrm{w}}$ should be obtained from the following:

- for webs without longitudinal stiffeners:

$$\bar{\lambda}_{\mathrm{w}} = 0{,}346 \frac{s_{\mathrm{w}}}{t} \sqrt{\frac{f_{\mathrm{yb}}}{E}} \qquad \text{... (6.10a)}$$

- for webs with longitudinal stiffeners, see figure 6.5:

$$\bar{\lambda}_{\mathrm{w}} = 0{,}346 \frac{s_{\mathrm{d}}}{t} \sqrt{\frac{5{,}34}{k_\tau} \frac{f_{\mathrm{yb}}}{E}} \quad \text{but} \quad \bar{\lambda}_{w} \geq 0{,}346 \frac{s_{\mathrm{p}}}{t} \sqrt{\frac{f_{\mathrm{yb}}}{E}} \qquad \text{... (6.10b)}$$

with:

$$k_\tau = 5{,}34 + \frac{2{,}10}{t} \left(\frac{\Sigma I_{\mathrm{s}}}{s_{\mathrm{d}}} \right)^{1/3}$$

where:

I_{s} is the second moment of area of the individual longitudinal stiffener as defined in 5.5.3.4.3(7), about the axis a – a as indicated in figure 6.5;

s_{d} is the total developed slant height of the web, as indicated in figure 6.5;

s_{p} is the slant height of the largest plane element in the web, see figure 6.5;

s_{w} is the slant height of the web, as shown in figure 6.5, between the midpoints of the corners, these points are the median points of the corners, see figure 5.3(c).

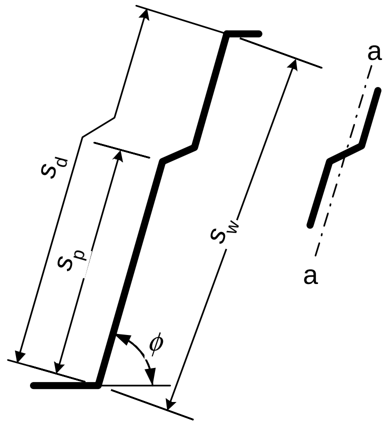


Figure 6.5: Longitudinally stiffened web

6.1.6 Torsional moment

(1) Where loads are applied eccentric to the shear centre of the cross-section, the effects of torsion ~~shall~~ should be taken into account.

(2) The centroidal axis and shear centre and imposed rotation centre to be used in determining the effects of the torsional moment, should be taken as those of the gross cross-section.

(3) The direct stresses due to the axial force N_{Ed} and the bending moments $M_{y,Ed}$ and $M_{z,Ed}$ should be based on the respective effective cross-sections used in 6.1.2 to 6.1.4. The shear stresses due to transverse shear forces, the shear stress due to uniform (St. Venant) torsion and the direct stresses and shear stresses due to warping, should all be based on the properties of the gross cross-section.

(4) In cross-sections subject to torsion, the following conditions should be satisfied (average yield strength is allowed here, see 3.2.1(5)):

$$\sigma_{tot,Ed} \leq f_{ya} / \gamma_{M0} \quad \text{... (6.11a)}$$

$$\tau_{tot,Ed} \leq \frac{f_{ya} / \sqrt{3}}{\gamma_{M0}} \quad \text{... (6.11b)}$$

$$\sqrt{\sigma_{tot,Ed}^{2} + 3\tau_{tot,Ed}^{2}} \leq 1{,}1 \frac{f_{ya}}{\gamma_{M0}} \quad \text{... (6.11c)}$$

where:

$\sigma_{tot,Ed}$ is the total direct stress, calculated on the relevant effective cross-section;

$\tau_{tot,Ed}$ is the total shear stress, calculated on the gross cross-section.

(5) The total direct stress $\sigma_{tot,Ed}$ and the total shear stress $\tau_{tot,Ed}$ should by obtained from:

$$\sigma_{tot,Ed} = \sigma_{N,Ed} + \sigma_{My,Ed} + \sigma_{Mz,Ed} + \sigma_{w,Ed} \quad \text{... (6.12a)}$$

$$\tau_{tot,Ed} = \tau_{Vy,Ed} + \tau_{Vz,Ed} + \tau_{t,Ed} + \tau_{w,Ed} \quad \text{... (6.12b)}$$

where:

$\sigma_{My,Ed}$ is the direct stress due to the bending moment $M_{y,Ed}$ (using effective cross-section);

$\sigma_{Mz,Ed}$ is the direct stress due to the bending moment $M_{z,Ed}$ (using effective cross-section);

$\sigma_{N,Ed}$ is the direct stress due to the axial force N_{Ed} (using effective cross-section);

$\sigma_{w,Ed}$ is the direct stress due to warping (using gross cross-section);

$\tau_{Vy,Ed}$ is the shear stress due to the transverse shear force $V_{y,Ed}$ (using gross cross-section);

$\tau_{Vz,Ed}$ is the shear stress due to the transverse shear force $V_{z,Ed}$ (using gross cross-section);

$\tau_{t,Ed}$ is the shear stress due to uniform (St. Venant) torsion (using gross cross-section);

$\tau_{w,Ed}$ is the shear stress due to warping (using gross cross-section).

6.1.7 Local transverse forces

6.1.7.1 General

(1) To avoid crushing, crippling or buckling in a web subject to a support reaction or other local transverse force applied through the flange, the transverse force F_{Ed} should satisfy:

$$F_{Ed} \leq R_{w,Rd} \qquad \text{... (6.13)}$$

where:

$R_{w,Rd}$ is the local transverse resistance of the web.

(2) The local transverse resistance of a web $R_{w,Rd}$ should be obtained as follows:

a) for an unstiffened web:

- for a cross-section with a single web: from 6.1.7.2;
- for any other case, including sheeting: from 6.1.7.3;

b) for a stiffened web: from 6.1.7.4.

(3) Where the local load or support reaction is applied through a cleat that is arranged to prevent distortion of the web and is designed to resist the local transverse force, the local resistance of the web to the transverse force need not be considered.

(4) In beams with I-shaped cross-sections built up from two channels, or with similar cross-sections in which two components are interconnected through their webs, the connections between the webs should be located as close as practicable to the flanges of the beam.

6.1.7.2 Cross-sections with a single unstiffened web

(1) For a cross-section with a single unstiffened web, see figure 6.6, the local transverse resistance of the web may be determined as specified in (2), provided that the cross-section satisfies the following criteria:

$$h_w / t \leq 200 \qquad \text{... (6.14a)}$$

$$r / t \leq 6 \qquad \text{... (6.14b)}$$

$$45° \leq \phi \leq 90° \qquad \text{... (6.14c)}$$

where:

h_w is the web height between the midlines of the flanges;

r is the internal radius of the corners;

ϕ is the slope of the web relative to the flanges [degrees].

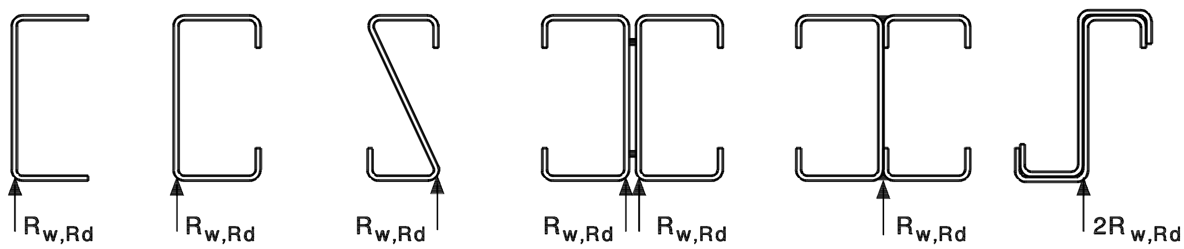


Figure 6.6: Examples of cross-sections with a single web

(2) For cross-sections that satisfy the criteria specified in (1), the local transverse resistance of a web $R_{w,Rd}$ may be determined as shown if figure 6.7.

(3) The values of the constants k_1 to k_5 should be determined as follows:

$k_1 = 1,33 - 0,33\,k$

$k_2 = 1,15 - 0,15\,r/t$ but $k_2 \geq 0,50$ and $k_2 \leq 1,0$

$k_3 = 0,7 + 0,3\,(\phi / 90)^2$

$k_4 = 1,22 - 0,22\,k$

$k_5 = 1,06 - 0,06\,r/t$ but $k_5 \leq 1,0$

where:

$k = f_{yb} / 228$ [with f_{yb} in N/mm^2];

s_s is the actual length of stiff bearing.

In the case of two equal and opposite local transverse forces distributed over unequal bearing lengths, the smaller value of s_s should be used.

(4) If the web rotation is prevented either by suitable restraint or because of the section geometry (e.g. I-beams, see fourth and fifth from the left in the figure 6.6) then the local transverse resistance of a web $R_{w,Rd}$ may be determined as follows:

a) for a single load or support reaction

i) $c < 1.5\,h_w$ (near or at free end)

for a cross-section of stiffened and unstiffened flanges

$$R_{w,Rd} = \frac{k_7 \left[8,8 + 1,1\sqrt{\frac{s_s}{t}} \right] t^2 f_{yb}}{\gamma_{M1}} \quad \ldots (6.16a)$$

ii) $c > 1.5\,h_w$ (far from free end)

for a cross-section of stiffened and unstiffened flanges

$$R_{w,Rd} = \frac{k_5^* k_6 \left[13,2 + 2,87\sqrt{\frac{s_s}{t}} \right] t^2 f_{yb}}{\gamma_{M1}} \quad \ldots (6.16b)$$

b) for opposite loads or reactions

i) $c < 1.5\,h_w$ (near or at free end)

for a cross-section of stiffened and unstiffened flanges

$$R_{w,Rd} = \frac{k_{10} k_{11} \left[8,8 + 1,1\sqrt{\frac{s_s}{t}} \right] t^2 f_{yb}}{\gamma_{M1}} \quad \ldots (6.16c)$$

ii) $c > 1.5\,h_w$ (loads or reactions far from free end)

for a cross-section of stiffened and unstiffened flanges

$$R_{w,Rd} = \frac{k_8 k_9 \left[13,2 + 2,87\sqrt{\frac{s_s}{t}} \right] t^2 f_{yb}}{\gamma_{M1}} \quad \ldots (6.16d)$$

Where the values of constants k_5^* to k_{11} should be determined as follows:

$k_5^* = 1,49 - 0,53\,k$ but $k_5^* \geq 0,6$

$k_6 = 0,88 - 0,12\,t / 1,9$

$k_7 = 1 + s_s / t / 750$ when $s_s / t < 150$; $k_7 = 1,20$ when $s_s / t > 150$

$k_8 = 1 / k$ when $s_s / t < 66,5$; $k_8 = (1,10 - s_s / t / 665) / k$ when $s_s / t > 66,5$

$k_9 = 0,82 + 0,15\, t / 1,9$

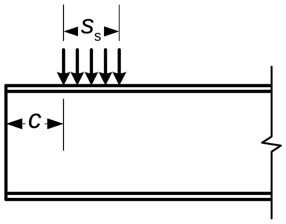
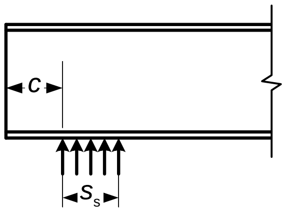
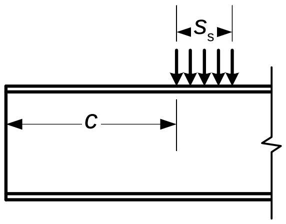
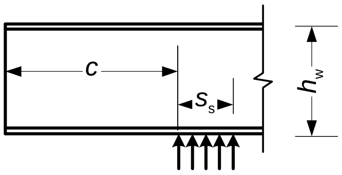
$k_{10} = (0,98 - s_s / t / 865) / k$

$k_{11} = 0,64 + 0,31\, t / 1,9$

where:

$k = f_{yb} / 228$ [with f_{yb} in N/mm^2];

s_s is the actual length of stiff bearing.

 	a) For a single local load or support reaction i) $c \leq 1,5\, h_w$ clear from a free end: - for a cross-section with stiffened flanges: $$R_{w,Rd} = \frac{k_1 k_2 k_3 \left[9,04 - \frac{h_w / t}{60}\right]\left[1 + 0,01 \frac{s_s}{t}\right] t^2 f_{yb}}{\gamma_{M1}} \quad (6.15a)$$ - for a cross-section with unstiffened flanges: - if $s_s/t \leq 60$: $$R_{w,Rd} = \frac{k_1 k_2 k_3 \left[5,92 - \frac{h_w / t}{132}\right]\left[1 + 0,01 \frac{s_s}{t}\right] t^2 f_{yb}}{\gamma_{M1}} \quad (6.15b)$$ - if $s_s/t > 60$: $$R_{w,Rd} = \frac{k_1 k_2 k_3 \left[5,92 - \frac{h_w / t}{132}\right]\left[0,71 + 0,015 \frac{s_s}{t}\right] t^2 f_{yb}}{\gamma_{M1}} \quad (6.15c)$$
 	ii) $c > 1,5\, h_w$ clear from a free end: - if $s_s/t \leq 60$: $$R_{w,Rd} = \frac{k_3 k_4 k_5 \left[14,7 - \frac{h_w / t}{49,5}\right]\left[1 + 0,007 \frac{s_s}{t}\right] t^2 f_{yb}}{\gamma_{M1}} \quad (6.15d)$$ - if $s_s/t > 60$: $$R_{w,Rd} = \frac{k_3 k_4 k_5 \left[14,7 - \frac{h_w / t}{49,5}\right]\left[0,75 + 0,011 \frac{s_s}{t}\right] t^2 f_{yb}}{\gamma_{M1}} \quad (6.15e)$$

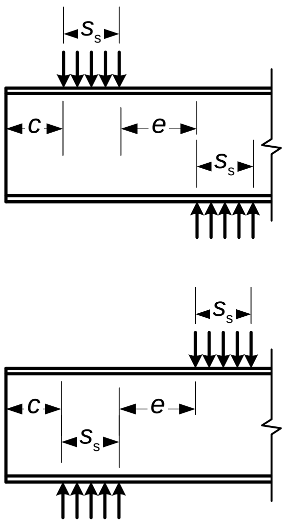
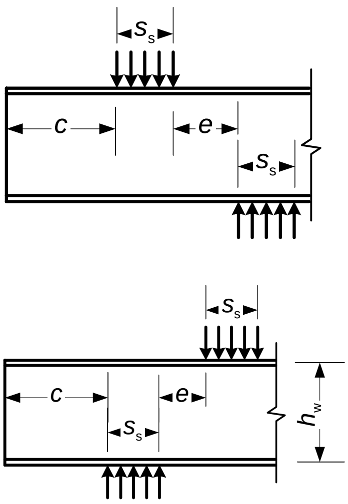
	b) For two opposing local transverse forces closer together than $1,5\ h_w$: i) $c \leq 1,5\ h_w$ clear from a free end: $$R_{w,Rd} = \frac{k_1 k_2 k_3 \left[6,66 - \frac{h_w/t}{64}\right]\left[1 + 0,01\frac{s_s}{t}\right] t^2 f_{yb}}{\gamma_{M1}} \qquad (6.15f)$$
	ii) $c > 1,5\ h_w$ clear from a free end: $$R_{w,Rd} = \frac{k_3 k_4 k_5 \left[21,0 - \frac{h_w/t}{16,3}\right]\left[1 + 0,0013\frac{s_s}{t}\right] t^2 f_{yb}}{\gamma_{M1}} \qquad (6.15g)$$

Figure 6.7: Local loads and supports — cross-sections with a single web

6.1.7.3 Cross-sections with two or more unstiffened webs

(1) In cross-sections with two or more webs, including sheeting, see figure 6.8, the local transverse resistance of an unstiffened web should be determined as specified in (2), provided that both of the following conditions are satisfied:

- the clear distance c from the actual bearing length for the support reaction or local load to a free end, see figure 6.9, is at least 40 mm;
- the cross-section satisfies the following criteria:

$$r/t \leq 10 \qquad \text{... (6.17a)}$$

$$h_w/t \leq 200 \sin \phi \qquad \text{... (6.17b)}$$

$$45° \leq \phi \leq 90° \qquad \text{... (6.17c)}$$

where:

h_w is the web height between the midlines of the flanges;

r is the internal radius of the corners;

ϕ is the slope of the web relative to the flanges [degrees].

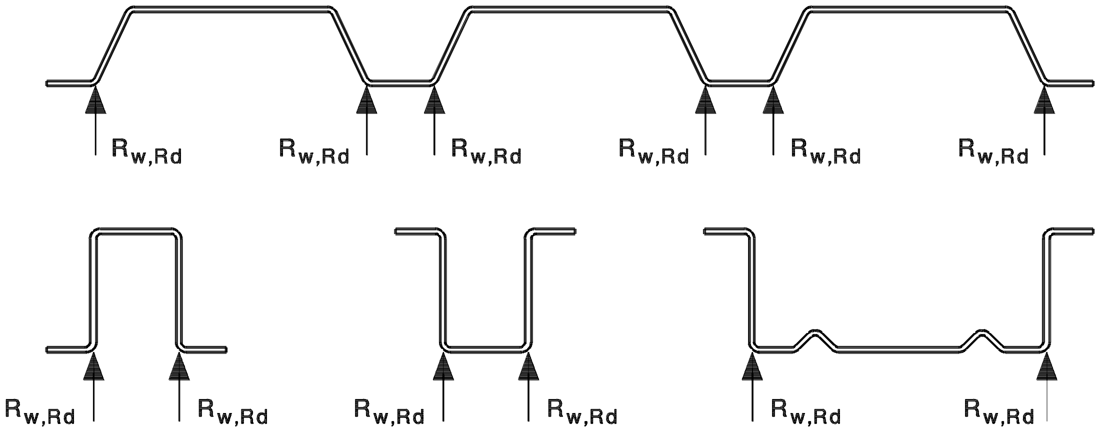


Figure 6.8: Examples of cross-sections with two or more webs

(2) Where both of the conditions specified in (1) are satisfied, the local transverse resistance $R_{w,Rd}$ per web of the cross-section should be determined from

$$R_{w,Rd} = \alpha t^2 \sqrt{f_{yb} E} \left(1 - 0{,}1\sqrt{r/t}\right)\left[0{,}5 + \sqrt{0{,}02\, l_a / t}\right]\left(2{,}4 + (\phi/90)^2\right)/\gamma_{M1} \qquad \text{... (6.18)}$$

where:

l_a is the effective bearing length for the relevant category, see (3);

α is the coefficient for the relevant category, see (3).

(3) The values of l_a and α should be obtained from (4) and (5) respectively. The maximum design value for l_a = 200 mm. When the support is a cold-formed section with one web or round tube, for s_s should be taken a value of 10 mm. The relevant category (1 or 2) should be based on the clear distance e between the local load and the nearest support, or the clear distance c from the support reaction or local load to a free end, see figure 6.9.

(4) The value of the effective bearing length l_a should be obtained from the following:

a) for Category 1: $l_a = 10$ mm ... (6.19a)

b) for Category 2:

- $\beta_V \leq 0{,}2$: $l_a = s_s$ ... (6.19b)
- $\beta_V \geq 0{,}3$: $l_a = 10$ mm ... (6.19c)
- $0{,}2 < \beta_V < 0{,}3$: Interpolate linearly between the values of l_a for 0,2 and 0,3

with:

$$\beta_v = \frac{|V_{Ed,1}| - |V_{Ed,2}|}{|V_{Ed,1}| + |V_{Ed,2}|}$$

in which $|V_{Ed,1}|$ and $|V_{Ed,2}|$ are the absolute values of the transverse shear forces on each side of the local load or support reaction, and $|V_{Ed,1}| \geq |V_{Ed,2}|$ and s_s is the actual length of stiff bearing.

(5) The value of the coefficent α should be obtained from the following:

a) for Category 1:

- for sheeting profiles: $\alpha = 0{,}075$ ... (6.20a)
- for liner trays and hat sections: $\alpha = 0{,}057$ ... (6.20b)

b) for Category 2:

- for sheeting profiles: $\alpha = 0{,}15$ ... (6.20c)
- for liner trays and hat sections: $\alpha = 0{,}115$ ... (6.20d)

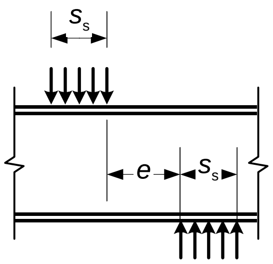
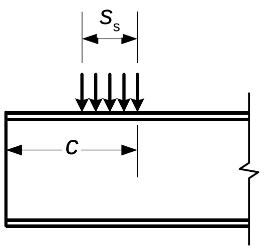
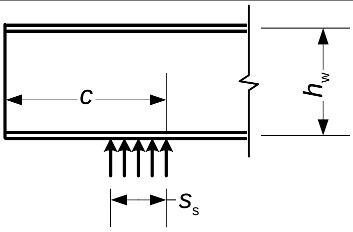
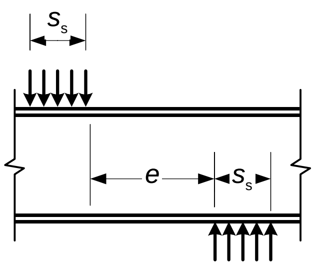
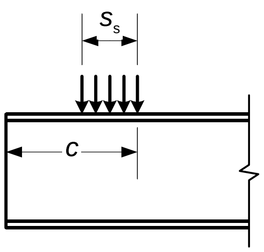
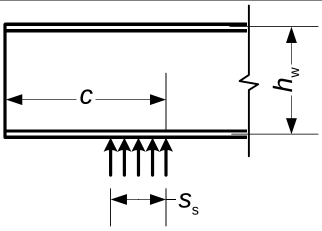
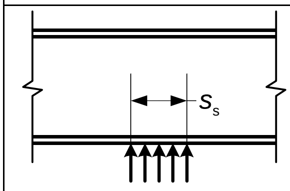
	Category 1 - local load applied with $e \leq 1{,}5\, h_w$ clear from the nearest support;
	Category 1 - local load applied with $c \leq 1{,}5\, h_w$ clear from a free end;
	Category 1 - reaction at end support with $c \leq 1{,}5\, h_w$ clear from a free end.
	Category 2 - local load applied with $e > 1{,}5\, h_w$ clear from the nearest support;
	Category 2 - local load applied with $c > 1{,}5\, h_w$ clear from a free end;
	Category 2 - reaction at end support with $c > 1{,}5\, h_w$ clear from a free end;
	Category 2 - reaction at internal support.

Figure 6.9: Local loads and supports —categories of cross-sections with two or more webs

6.1.7.4 Stiffened webs

(1) The local transverse resistance of a stiffened web may be determined as specified in (2) for cross-sections with longitudinal web stiffeners folded in such a way that the two folds in the web are on opposite sides of the system line of the web joining the points of intersection of the midline of the web with the midlines of the flanges, see figure 6.10, that satisfy the condition:

$$2 < \frac{e_{\max}}{t} < 12 \qquad \text{... (6.21)}$$

where:

$e_{\max}$ is the larger eccentricity of the folds relative to the system line of the web.

(2) For cross-sections with stiffened webs satisfying the conditions specified in (1), the local transverse resistance of a stiffened web may be determined by multiplying the corresponding value for a similar unstiffened web, obtained from 6.1.7.2 or 6.1.7.3 as appropriate, by the factor $\kappa_{a,s}$ given by:

$$\kappa_{a,s} = 1{,}45 - 0{,}05\, e_{\max}/t \quad \text{but} \quad \kappa_{a,s} \leq 0{,}95 + 35\,000\, t^2 e_{\min} / (b_d^{\,2} s_p) \qquad \text{... (6.22)}$$

where:

b_d is the developed width of the loaded flange, see figure 6.10;

$e_{\min}$ is the smaller eccentricity of the folds relative to the system line of the web;

s_p is the slant height of the plane web element nearest to the loaded flange, see figure 6.10.

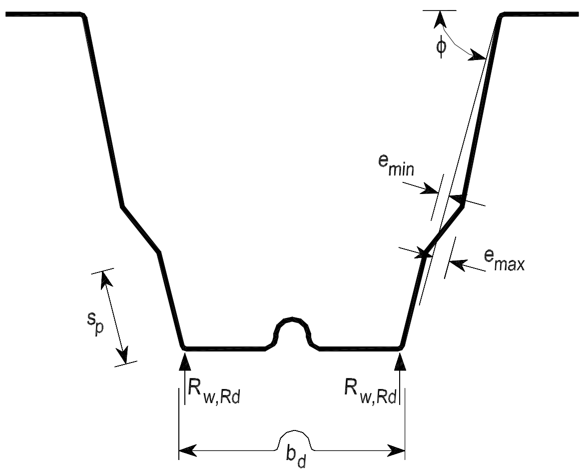


Figure 6.10: Stiffened webs

6.1.8 Combined tension and bending

(1) Cross-sections subject to combined axial tension N_{Ed} and bending moments $M_{y,Ed}$ and $M_{z,Ed}$ should satisfy the criterion:

$$\frac{N_{Ed}}{N_{t,Rd}} + \frac{M_{y,Ed}}{M_{cy,Rd,ten}} + \frac{M_{z,Ed}}{M_{cz,Rd,ten}} \leq 1 \qquad \text{... (6.23)}$$

where:

$N_{t,Rd}$ is the design resistance of a cross-section for uniform tension (6.1.2);

$M_{cy,Rd,ten}$ is the design moment resistance of a cross-section for maximum tensile stress if subject only to moment about the y - y axis (6.1.4);

$M_{cz,Rd,ten}$ is the design moment resistance of a cross-section for maximum tensile stress if subject only to moment about the z - z axis (6.1.4).

(2) If $M_{cy,Rd,com} \leq M_{cy,Rd,ten}$ or $M_{cz,Rd,com} \leq M_{cz,Rd,ten}$ (where $M_{cy,Rd,com}$ and $M_{cz,Rd,com}$ are the moment resistances for the maximum compressive stress in a cross-section that is subject only to moment about the relevant axis), the following criterion should also be satisfied:

$$\frac{M_{y,Ed}}{M_{cy,Rd.com}} + \frac{M_{z,Ed}}{M_{cz,Rd,com}} - \frac{N_{Ed}}{N_{t,Rd}} \leq 1 \qquad \text{... (6.24)}$$

6.1.9 Combined compression and bending

(1) Cross-sections subject to combined axial compression N_{Ed} and bending moments $M_{y,Ed}$ and $M_{z,Ed}$ should satisfy the criterion:

$$\frac{N_{Ed}}{N_{c,Rd}} + \frac{M_{y,Ed} + \Delta M_{y,Ed}}{M_{cy,Rd,com}} + \frac{M_{z,Ed} + \Delta M_{z,Ed}}{M_{cz,Rd,com}} \leq 1 \qquad \text{... (6.25)}$$

in which $N_{c,Rd}$ is as defined in 6.1.3, $M_{cy,Rd,com}$ and $M_{cz,Rd,com}$ are as defined in 6.1.8.

(2) The additional moments $\Delta M_{y,Ed}$ and $\Delta M_{z,Ed}$ due to shifts of the centroidal axes should be taken as:

$$\Delta M_{y,Ed} = N_{Ed}\, e_{Ny}$$

$$\Delta M_{z,Ed} = N_{Ed}\, e_{Nz}$$

in which e_{Ny} and e_{Nz} are the shifts of y-y and z-z centroidal axis due to axial forces, see 6.1.3(3).

(3) If $M_{cy,Rd,ten} \leq M_{cy,Rd,com}$ or $M_{cz,Rd,ten} \leq M_{cz,Rd,com}$ the following criterion should also be satisfied:

$$\frac{M_{y,Ed} + \Delta M_{y,Ed}}{M_{cy,Rd,ten}} + \frac{M_{z,Ed} + \Delta M_{z,Ed}}{M_{cz,Rd,ten}} - \frac{N_{Ed}}{N_{c,Rd}} \leq 1 \qquad \text{... (6.26)}$$

in which $M_{cy,Rd,ten}$, $M_{cz,Rd,ten}$ are as defined in 6.1.8.

6.1.10 Combined shear force, axial force and bending moment

(1) For ~~C~~cross-sections subject to the combined action of an axial force N_{Ed}, a bending moment M_{Ed} and a shear force V_{Ed} no reduction due to shear force need not be done provided that $V_{Ed} \leq 0{,}5\, V_{w,Rd}$. If the shear force is larger than half of the shear force resistance then following equations should be satisfied:

$$\frac{N_{Ed}}{N_{Rd}} + \frac{M_{y,Ed}}{M_{y,Rd}} + \left(1 - \frac{M_{f,Rd}}{M_{pl,Rd}}\right)\left(\frac{2V_{Ed}}{V_{w,Rd}} - 1\right)^2 \leq 1{,}0 \qquad \text{...(6.27)}$$

where:

- N_{Rd} is the design resistance of a cross-section for uniform tension or compression given in 6.1.2 or 6.1.3;
- $M_{y,Rd}$ is the design moment resistance of the cross-section given in 6.1.4;
- $V_{w,Rd}$ is the design shear resistance of the web given in 6.1.5(1);
- $M_{f,Rd}$ is the design plastic moment resistance of a cross-section consisting only of flanges, see EN 1993-1-5;
- $M_{pl,Rd}$ is the plastic moment resistance of the cross-section, see EN 1993-1-5.

For members and sheeting with more than one web $V_{w,Rd}$ is the sum of the resistances of the webs. See also EN 1993-1-5.

6.1.11 Combined bending moment and local load or support reaction

(1) Cross-sections subject to the combined action of a bending moment M_{Ed} and a transverse force due to a local load or support reaction F_{Ed} should satisfy the following:

$$M_{Ed} / M_{c,Rd} \leq 1 \quad \text{... (6.28a)}$$

$$F_{Ed} / R_{w,Rd} \leq 1 \quad \text{... (6.28b)}$$

$$\frac{M_{Ed}}{M_{c,Rd}} + \frac{F_{Ed}}{R_{w,Rd}} \leq 1{,}25 \quad \text{... (6.28c)}$$

where:

$M_{c,Rd}$ is the moment resistance of the cross-section given in 6.1.4.1(1);

$R_{w,Rd}$ is the appropriate value of the local transverse resistance of the web from 6.1.7.

In equation (6.2.8c) the bending moment M_{Ed} may be calculated at the edge of the support. For members and sheeting with more than one web, $R_{w,Rd}$ is the sum of the local transverse resistances of the individual webs.

6.2 Buckling resistance

6.2.1 General

(1) In members with cross-sections that are susceptible to cross-sectional distortion, account ~~shall~~ should be taken of possible lateral buckling of compression flanges and lateral bending of flanges generally, see 6.2.4.

(2) The effects of local and distortional buckling ~~shall~~ should be taken into account as specified in Section 5.5.

6.2.2 Flexural buckling

(1) The design buckling resistance $N_{b,Rd}$ for flexural buckling should be obtained from EN 1993-1-1 using the appropriate buckling curve from table 6.3 according to the type of cross-section~~, and~~ axis of buckling and yield strength used, see (3).

(2) The buckling curve for a cross-section not included in table 6.3 may be obtained by analogy.

(3) The buckling resistance of a closed built-up cross-section should be determined using either:

- buckling curve b in association with the basic yield strength f_{yb} of the flat sheet material out of which the member is made by cold forming;
- buckling curve c in association with the average yield strength f_{ya} of the member after cold forming, determined as specified in 3.2.3, provided that $A_{eff} = A_g$.

6.2.3 Torsional buckling and torsional-flexural buckling

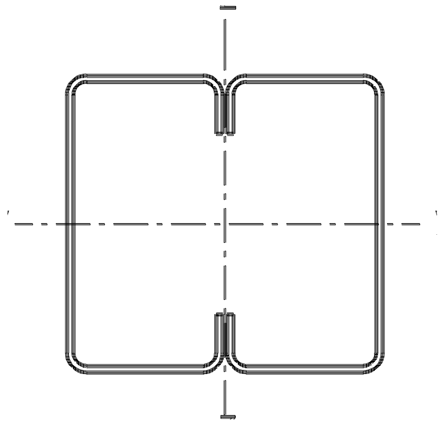
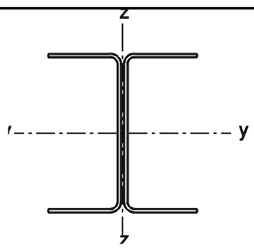
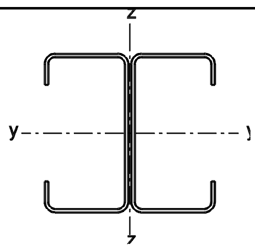
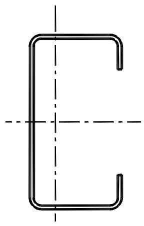
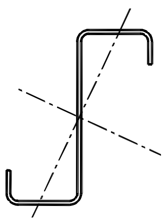
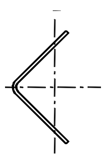
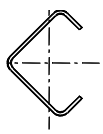
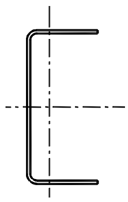
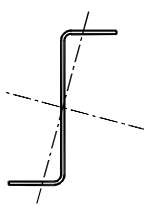
(1) For members with point-symmetric open cross-sections (e.g Z-purlin with equal flanges), account ~~shall~~ should be taken of the possibility that the resistance of the member to torsional buckling might be less than its resistance to flexural buckling.

(2) For members with mono-symmetric open cross-sections, see figure 6.12, account ~~shall~~ should be taken of the possibility that the resistance of the member to torsional-flexural buckling might be less than its resistance to flexural buckling.

(3) For members with non-symmetric open cross-sections, account ~~shall~~ should be taken of the possibility that the resistance of the member to either torsional or torsional-flexural buckling might be less than its resistance to flexural buckling.

(4) The design buckling resistance $N_{b,Rd}$ for torsional or torsional-flexural buckling should be obtained from ~~6.2.1~~ EN 1993-1-1, 6.3.1.1 using the relevant buckling curve for buckling about the z-z axis obtained from table 6.3.

Table 6.3: Appropriate buckling curve for various types of cross-section

Type of cross-section		Buckling about axis	Buckling curve
	if f_{yb} is used	Any	b
	if f_{ya} is used *)	Any	c
 		y - y z - z	a b
 		Any	b
    or other cross-section		Any	c

*) The average yield strength f_{ya} should not be used unless $A_{eff} = A_g$

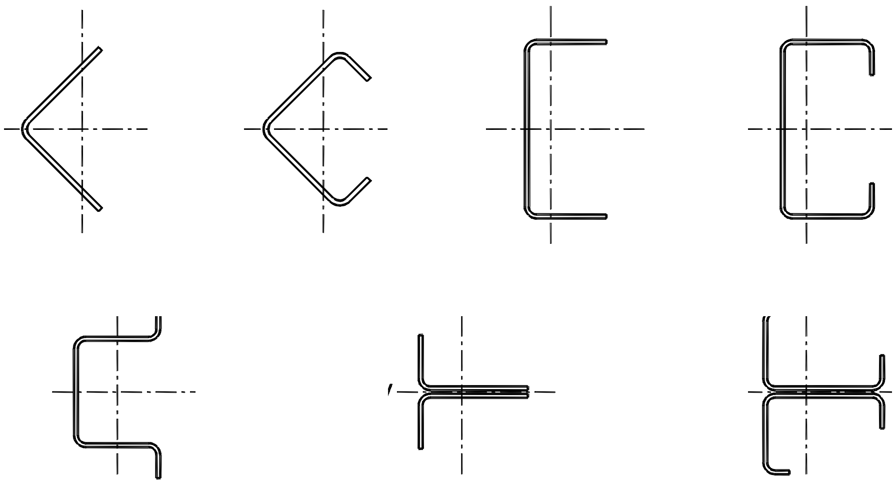


Figure 6.12: Cross-sections susceptible to torsional-flexural buckling

(5) The elastic critical force $N_{cr,T}$ for torsional buckling of simply supported beam should be determined from:

$$N_{cr,T} = \frac{1}{i_o^2}\left(G I_t + \frac{\pi^2 E I_w}{l_T^2}\right) \quad \text{... (6.33a)}$$

with:

$$i_o^2 = i_y^2 + i_z^2 + y_o^2 + z_o^2 \quad \text{... (6.33b)}$$

where:

- G is the shear modulus;
- I_t is the torsion constant of the gross cross-section;
- I_w is the warping constant of the gross cross-section;
- i_y is the radius of gyration of the gross cross-section about the y - y axis;
- i_z is the radius of gyration of the gross cross-section about the z - z axis;
- l_T is the buckling length of the member for torsional buckling;
- y_o, z_o are the shear centre co-ordinates with respect to the centroid of the gross cross-section.

(6) For doubly symmetric cross-sections (e.g. $y_o = z_o = 0$), the elastic critical force $N_{cr,TF}$ for torsional-flexural buckling should be determined from:

$$N_{cr,TF} = N_{cr,T} \quad \text{...(6.34)}$$

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

(7) For cross-sections that are symmetrical about the y - y axis (e.g. $z_o = 0$), the elastic critical force $N_{cr,TF}$ for torsional-flexural buckling should be determined from:

$$N_{cr,TF} = \frac{N_{cr,y}}{2\beta}\left[1 + \frac{N_{cr,T}}{N_{cr,y}} - \sqrt{\left(1 - \frac{N_{cr,T}}{N_{cr,y}}\right)^2 + 4\left(\frac{y_o}{i_o}\right)^2 \frac{N_{cr,T}}{N_{cr,y}}}\right] \quad \text{... (6.35)}$$

with:

$$\beta = 1 - \left(\frac{y_o}{i_o}\right)^2 .$$

(8) The buckling length l_T for torsional or torsional-flexural buckling should be determined taking into account the degree of torsional and warping restraint at each end of the system length L_T.

(9) For practical connections at each end, the value of l_T / L_T may be taken as follows:

- 1,0 for connections that provide partial restraint against torsion and warping, see figure 6.13(a);
- 0,7 for connections that provide significant restraint against torsion and warping, see figure 6.13(b).

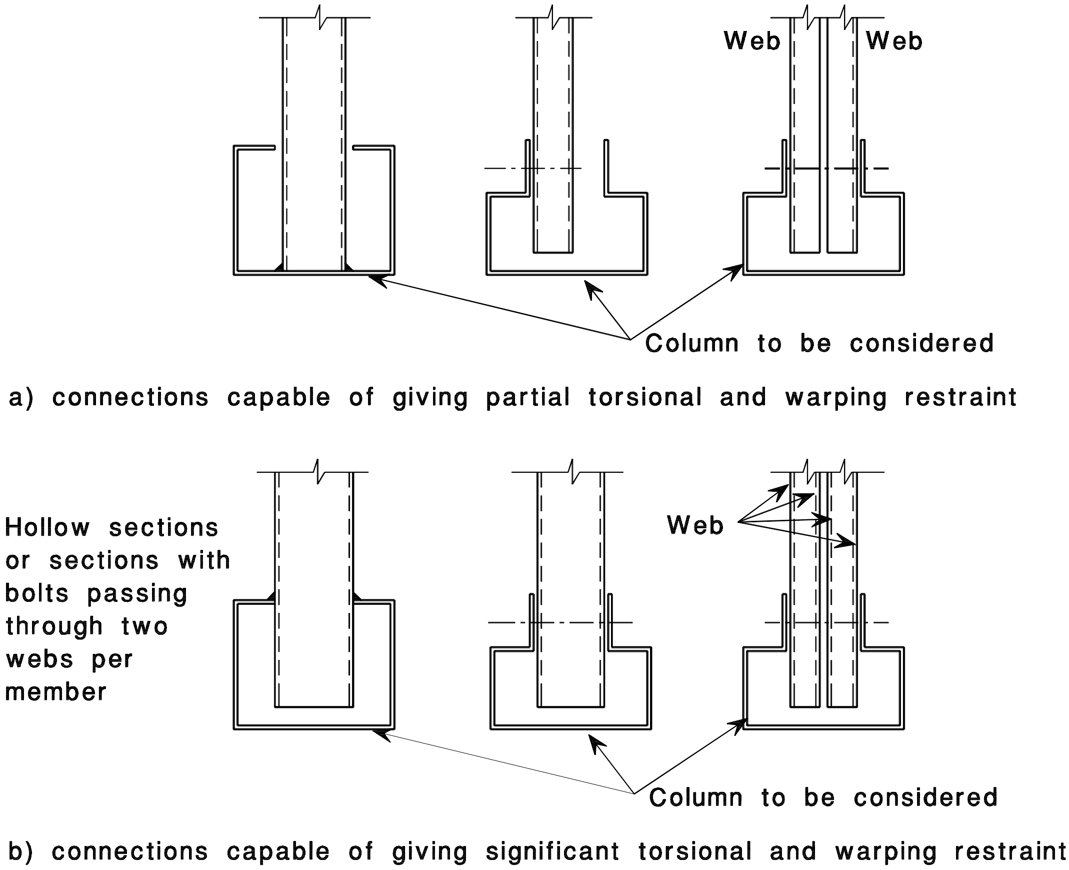


Figure 6.13: Torsional and warping restraint from practical connections

6.2.4 Lateral-torsional buckling of members subject to bending

(1) The design buckling resistance moment of a member that is susceptible to lateral-torsional buckling should be determined according to EN 1993-1-1, section ~~6.3.4~~ 6.3.2.2 using the lateral buckling curve a or b~~a with α_{LT} = 0,21~~.

(2) This method should not be used for the sections that have a significant angle between the principal axes of the effective cross-section, compared to those of the gross cross-section.

6.2.5 Bending and axial compression

(1) The interaction between axial force and bending moment may be obtained from a second-order analysis of the member as specified in EN 1993-1-1, based on the properties of the effective cross-section obtained from Section 5.5. See also 5.3.

(2) As an alternative the interaction formula (6.38) may be used

$$\left(\frac{N_{Ed}}{N_{b,Rd}}\right)^{0,8} + \left(\frac{M_{Ed}}{M_{b,Rd}}\right)^{0,8} \leq 1,0 \qquad \ldots(6.38)$$

where $N_{b,Rd}$ is the design buckling resistance of a compression member according to 6.2.2 (flexural, torsional or torsional-flexural buckling) and $M_{b,Rd}$ is the design bending moment resistance according to 6.2.~~4~~3.

6.3 Bending and axial tension

(1) The interaction equations for compressive force in ~~6.2~~ 6.2.5 are applicable.

7 Serviceability limit states

7.1 General

(1) The rules for serviceability limit states given in Section 7 of EN 1993-1-1 ~~shall~~ should also be applied to cold-formed ~~thin gauge~~ members and sheeting.

(2) The properties of the effective cross-section for serviceability limit states obtained from Section 5.1 should be used in all serviceability limit state calculations for cold-formed ~~thin gauge~~ members and sheeting.

(3) The second moment of area may be calculated alternatively by interpolation of gross cross-section and effective cross-section using the expression

$$I_{fic} = I_{gr} - \frac{\sigma_{gr}}{\sigma}(I_{gr} - I(\sigma)_{eff}) \quad \ldots (7.1)$$

where

I_{gr} is second moment of area of the gross cross-section;

σ_{gr} is maximum compressive bending stress in the serviceability limit state, based on the gross cross-section (positive in formula);

$I(\sigma)_{eff}$ is the second moment of area of the effective cross-section with allowance for local buckling calculated for a maximum stress $\sigma \geq \sigma_{gr}$, in which the maximum stress is the largest absolute value of stresses within the calculation length considered.

(4) The effective second moment of area I_{eff} (or I_{fic}) may be taken as variable along the span according to most severe locations. Alternatively a uniform value may be used, based on the maximum absolute span moment due to serviceability loading.

7.2 Plastic deformation

(1) In case of plastic global analysis the combination of support moment and support reaction at an internal support should not exceed 0,9 times the combined design resistance, determined using $\gamma_{M,ser}$, see section 2(5).

(2) The combined design resistance may be determined from 6.1.11, but using the effective cross-section for serviceability limit states and $\gamma_{M,ser}$.

7.3 Deflections

(1) The deflections may be calculated assuming elastic behaviour.

(2) The influence of slip in the connections (for example in the case of continuous beam systems with sleeves and overlaps) should be considered in the calculation of deflections, forces and moments.

8 Design of joints

8.1 General

(1) For design assumptions and requirements of joints see EN 1993-1-8.

(2) The following rules apply to core vthickness $t_{cor} \leq 4$ mm, not covered by EN 1993-1-8.

8.2 Splices and end connections of members subject to compression

(1) Splices and end connections in members that are subject to compression, ~~shall~~ should either have at least

the same resistance as the cross-section of the member, or be designed to resist an additional bending moment due to the second-order effects within the member, in addition to the internal compressive force N_{Ed} and the internal moments $M_{y,Ed}$ and $M_{z,Ed}$ obtained from the global analysis.

(2) In the absence of a second-order analysis of the member, this additional moment ΔM_{Ed} should be taken as acting about the cross-sectional axis that gives the smallest value of the reduction factor χ for flexural buckling, see 6.2.2.1(2), with a value determined from:

$$\Delta M_{Ed} = N_{Ed} \left(\frac{1}{\chi} - 1 \right) \frac{W_{eff}}{A_{eff}} \sin \frac{\pi a}{l} \qquad \text{...(8.1a)}$$

where:

A_{eff} is the effective area of the cross-section;

a is the distance from the splice or end connection to the nearer point of contraflexure;

l is the buckling length of the member between points of contraflexure, for buckling about the relevant axis;

W_{eff} is the section modulus of the effective cross-section for bending about the relevant axis.

Splices and end connections should be designed to resist an additional internal shear force

$$\Delta V_{Ed} = \frac{\pi N_{Ed}}{l} \left(\frac{1}{\chi} - 1 \right) \frac{W_{eff}}{A_{eff}} \qquad \text{...(8.1b)}$$

(3) Splices and end connections should be designed in such a way that load may be transmitted to the effective portions of the cross-section.

(4) If the constructional details at the ends of a member are such that the line of action of the internal axial force cannot be clearly identified, a suitable eccentricity should be assumed and the resulting moments should be taken into account in the design of the member, the end connections and the splice, if there is one.

8.3 Connections with mechanical fasteners

(1) Connections with mechanical fasteners ~~shall~~ should be compact in shape. The positions of the fasteners ~~shall~~ should be arranged to provide sufficient room for satisfactory assembly and maintenance.

NOTE More information see Part 1-8 of EN 1993.

(2) The shear forces on individual mechanical fasteners in a connection may be assumed to be equal, provided that:

- the fasteners have sufficient ductility;
- shear is not the critical failure mode.

(3) For design by calculation the resistances of mechanical fasteners subject to predominantly static loads should be determined from:

- table 8.1 for blind rivets;
- table 8.2 for self-tapping screws;
- table 8.3 for cartridge fired pins;
- table 8.4 for bolts.

NOTE: For determining the design resistance of mechanical fasteners by testing see 9(4).

(4) In tables 8.1 to 8.4 the meanings of the symbols ~~shall~~ should be taken as follows:

A is the gross cross-sectional area of a bolt;

A_s is the tensile stress area of a bolt;

A_{net} is the net cross-sectional area of the connected part;

β_{Lf} is the reduction factor for long joints according to EN 1993-1-8;

d is the nominal diameter of the fastener;

d_o is the nominal diameter of the hole;

d_w is the diameter of the washer or the head of the fastener;

e_1 is the end distance from the centre of the fastener to the adjacent end of the connected part, in the direction of load transfer, see figure 8.1;

e_2 is the edge distance from the centre of the fastener to the adjacent edge of the connected part, in the direction perpendicular to the direction of load transfer, see figure 8.1;

f_{ub} is the ultimate tensile strength of the bolt material;

$f_{u,sup}$ is the ultimate tensile strength of the supporting member into which a screw is fixed;

n is the number of sheets that are fixed to the supporting member by the same screw or pin;

n_f is the number of mechanical fasteners in one connection;

p_1 is the spacing centre-to-centre of fasteners in the direction of load transfer, see figure 8.1;

p_2 is the spacing centre-to-centre of fasteners in the direction perpendicular to the direction of load transfer, see figure 8.1;

t is the thickness of the thinner connected part or sheet;

t_1 is the thickness of the thicker connected part or sheet;

t_{sup} is the thickness of the supporting member into which a screw or a pin is fixed.

(5) The partial factor γ_M for calculating the design resistances of mechanical fasteners ~~shall~~ should be taken as γ_{M2}:

NOTE The value γ_{M2} may be given in the National Annex. The value $\gamma_{M2} = 1,25$ is recommended.

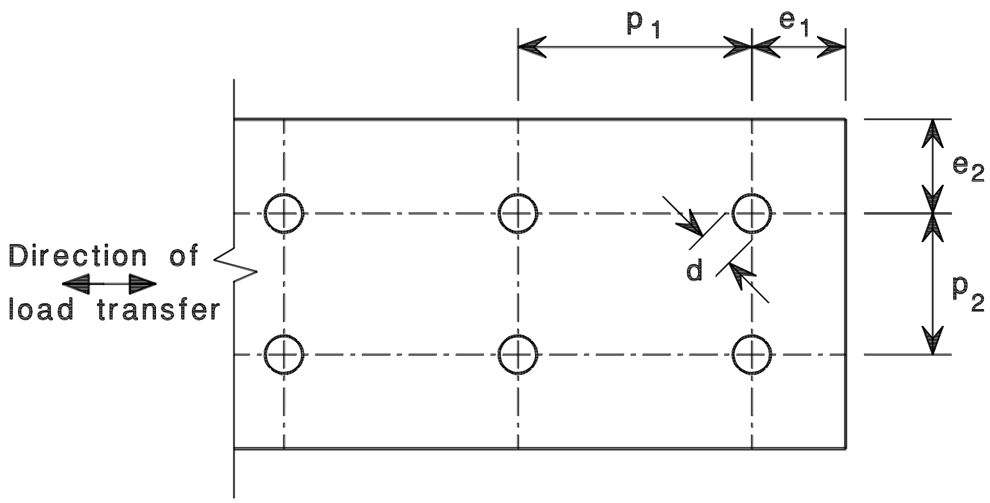


Figure 8.1: End distance, edge distance and spacings for fasteners and spot welds

(6) If the pull-out resistance $F_{o,Rd}$ of a fastener is smaller than its pull-through resistance $F_{p,Rd}$ the deformation capacity should be determined from tests.

(7) The pull-through resistances given in tables 8.2 and 8.3 for self-tapping screws and cartridge fired pins should be reduced if the fasteners are not located centrally in the troughs of the sheeting. If attachment is at a quarter point, the design resistance should be reduced to $0{,}9F_{p,Rd}$ and if there are fasteners at both quarter points, the resistance should be taken as $0{,}7F_{p,Rd}$ per fastener, see figure 8.2.

(8) For a fastener loaded in combined shear and tension, provided that both $F_{t,Rd}$ and $F_{v,Rd}$ are determined by calculation on the basis of tables 8.1 to 8.4, the resistance of the fastener to combined shear and tension may be verified using:

$$\frac{F_{t,Ed}}{\min(F_{p,Rd}, F_{o,Rd})} + \frac{F_{v,Ed}}{\min(F_{b,Rd}, F_{n,Rd})} \leq 1 \qquad \text{... (8.2)}$$

(9) The gross section distortion may be neglected if the design resistance is obtained from tables 8.1 to 8.4, provided that the fastening is through a flange not more than 150 mm wide.

(10)The diameter of holes for screws should be in accordance with the manufacturer's guidelines. These guidelines should be based on following criteria:

- the applied torque should be just higher than the threading torque;
- the applied torque should be lower than the thread stripping torque or head-shearing torque;
- the threading torque should be smaller than 2/3 of the head-shearing torque.

(11)For long joints a reduction factor β_{Lf} should be taken into account according to EN 1993-1-8.

(12)The design rules for blind rivets are valid only if the diameter of the hole is not 0,1 mm larger than the diameter of the rivet.

(13)For the bolts M12 and M14 with the hole diameters 2 mm larger than the bolt diameter, reference is made to EN 1993-1-8.

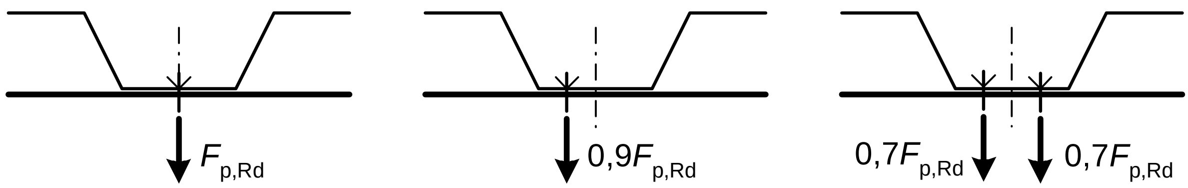


Figure 8.2: Reduction of ~~tension~~ pull through resistance due to the position of fasteners

Table 8.1: Design resistances for blind rivets [1)]

Rivets loaded in shear:
Bearing resistance: $F_{b,Rd} = \alpha f_u d t / \gamma_{M2}$ but $F_{b,Rd} \le f_u e_1 t / (1,2\ \gamma_{M2})$ In which α is given by the following: - if $t = t_1$: $\alpha = 3,6\sqrt{t/d}$ but $\alpha \le 2,1$ - if $t_1 \ge 2,5t$: $\alpha = 2,1$ - if $t < t_1 < 2,5t$: obtain α by linear interpolation. Net-section resistance: $F_{n,Rd} = A_{net} f_u / \gamma_{M2}$ Shear resistance: Shear resistance $F_{v,Rd}$ to be determined by testing *[1)] and $F_{v,Rd} = F_{v,Rk} / \gamma_{M2}$
Conditions: [4)] $F_{v,Rd} \ge 1,2 F_{b,Rd} / (n_f \beta_{Lf})$ or $F_{v,Rd} \ge 1,2 F_{n,Rd}$
Rivets loaded in tension: [2)]
Pull-through resistance: Pull-through resistance $F_{p,Rd}$ to be determined by testing *[1)]. Pull-out resistance: Not relevant for rivets. Tension resistance: Tension resistance $F_{t,Rd}$ to be determined by testing *[1)]
Conditions: $F_{t,Rd} \ge n\Sigma F_{p,Rd}$
Range of validity: [3)]
$e_1 \ge 1,5d$ $p_1 \ge 3d$ $2,6\text{ mm} \le d \le 6,4\text{ mm}$ $e_2 \ge 1,5d$ $p_2 \ge 3d$ $f_u \le 550\text{ MPa}$
[1)] In this table it is assumed that the thinnest sheet is next to the preformed head of the blind rivet. [2)] Blind rivets are not usually used in tension. [3)] Blind rivets may be used beyond this range of validity if the resistance is determined from the results of tests. [4)] The required conditions should be fulfilled when deformation capacity of the connection is needed. When these conditions are not fulfilled there should be proved that the needed deformation capacity will be provided by other parts of the structure.

NOTE*[1)] The National Annex may give further information on shear resistance of blind rivets loaded in shear and pull-through resistance and tension resistance of blind rivets loaded in tension.

Table 8.2: Design resistances for self-tapping screws [1)]

Screws loaded in shear:

Bearing resistance: $F_{b,Rd} = \alpha f_u d t / \gamma_{M2}$

In which α is given by the following:

- if $t = t_1$: $\alpha = 3,2 \sqrt{t/d}$ but $\alpha \leq 2,1$
- if $t_1 \geq 2,5t$ and $t < 1,0$ mm: $\alpha = 3,2 \sqrt{t/d}$ but $\alpha \leq 2,1$
- if $t_1 \geq 2,5t$ and $t \geq 1,0$ mm: $\alpha = 2,1$
- if $t < t_1 < 2,5t$: obtain α by linear interpolation.

Net-section resistance: $F_{n,Rd} = A_{net} f_u / \gamma_{M2}$

Shear resistance: Shear resistance $F_{v,Rd}$ to be determined by testing *[2)]

$F_{v,Rd} = F_{v,Rk} / \gamma_{M2}$

Conditions: [4)] $F_{v,Rd} \geq 1,2 F_{b,Rd} \cancel{/(n_f \beta_{Lf})}$ or $\Sigma F_{v,Rd} \geq 1,2 F_{n,Rd}$

Screws loaded in tension:

Pull-through resistance: [2)]

- for static loads: $F_{p,Rd} = d_w t f_u / \gamma_{M2}$
- for screws subject to wind loads and combination of wind loads and static loads: $F_{p,Rd} = 0,5 d_w t f_u / \gamma_{M2}$

Pull-out resistance: If $t_{sup} / s < 1$: $F_{o,Rd} = 0,45 d t_{sup} f_{u,sup} / \gamma_{M2}$ (s is the thread pitch)

If $t_{sup} / s \geq 1$: $F_{o,Rd} = 0,65 d t_{sup} f_{u,sup} / \gamma_{M2}$

Tension resistance: Tension resistance $F_{t,Rd}$ to be determined by testing *[2)].

Conditions: [4)] $F_{t,Rd} \geq n \Sigma F_{p,Rd}$ or $F_{t,Rd} \geq F_{o,Rd}$

Range of validity: [3)]

Generally: $e_1 \geq 3d$ $p_1 \geq 3d$ $3,0 \text{ mm} \leq d \leq 8,0 \text{ mm}$

$e_2 \geq 1,5d$ $p_2 \geq 3d$

For tension: $0,5 \text{ mm} \leq t \leq 1,5 \text{ mm}$ and $t_1 \geq 0,9 \text{ mm}$

$f_u \leq 550$ MPa

[1)] In this table it is assumed that the thinnest sheet is next to the head of the screw.
[2)] These values assume that the washer has sufficient rigidity to prevent it from being deformed appreciably or pulled over the head of the fastener.
[3)] Self-tapping screws may be used beyond this range of validity if the resistance is determined from the results of tests.
[4)] The required conditions should be fulfilled when deformation capacity of the connection is needed. When these conditions are not fulfilled there should be proved that the needed deformation capacity will be provided by other parts of the structure.

NOTE*[2)] The National Annex may give further information on shear resistance of self-tapping skrews loaded in shear and tension resistance of self-tapping skrews loaded in tension.

Table 8.3: Design resistances for cartridge fired pins

Pins loaded in shear:

Bearing resistance:

$F_{b,Rd} = 3{,}2 f_u d t / \gamma_{M2}$

Net-section resistance: $F_{n,Rd} = A_{net} f_u / \gamma_{M2}$

Shear resistance: Shear resistance $F_{v,Rd}$ to be determined by testing *[3)]

$F_{v,Rd} = F_{v,Rk} / \gamma_{M2}$

Conditions: [3)] $F_{v,Rd} \geq 1{,}5 \Sigma F_{b,Rd} \cancel{/(n_f \beta_{Lf})}$ or $\Sigma F_{v,Rd} \geq 1{,}5 F_{n,Rd}$

Pins loaded in tension:

Pull-through resistance: [1)]

- for static loads: $F_{p,Rd} = d_w t f_u / \gamma_{M2}$
- for wind loads and combination of wind loads and static loads: $F_{p,Rd} = 0{,}5 d_w t f_u / \gamma_{M2}$

Pull-out resistance:

Pull-out resistance $F_{o,Rd}$ to be determined by testing *[3)]

Tension resistance:

Tension resistance $F_{t,Rd}$ to be determined by testing *[3)]

Conditions: [3)] $F_{o,Rd} \geq \cancel{n} \Sigma F_{p,Rd}$ or $F_{t,Rd} \geq F_{o,Rd}$

Range of validity: [2)]

Generally:	$e_1 \geq 4{,}5 d$	$3{,}7 \text{ mm} \leq d \leq 6{,}0 \text{ mm}$
	$e_2 \geq 4{,}5 d$	for $d = 3{,}7$ mm: $t_{sup} \geq 4{,}0$ mm
	$p_1 \geq 4{,}5 d$	for $d = 4{,}5$ mm: $t_{sup} \geq 6{,}0$ mm
	$p_2 \geq 4{,}5 d$	for $d = 5{,}2$ mm: $t_{sup} \geq 8{,}0$ mm
	$f_u \leq 550$ MPa	
For tension:	$0{,}5 \text{ mm} \leq t \leq 1{,}5 \text{ mm}$	$t_{sup} \geq 6{,}0$ mm

[1)] These values assume that the washer has sufficient rigidity to prevent it from being deformed appreciably or pulled over the head of the fastener.

[2)] Cartridge fired pins may be used beyond this range of validity if the resistance is determined from the results of tests.

[3)] The required conditions should be fulfilled when deformation capacity of the connection is needed. When these conditions are not fulfilled there should be proved that the needed deformation capacity will be provided by other parts of the structure.

NOTE*[3)] The National Annex may give further information on shear resistance of cartrige fired pins loaded in shear and pull-out resistance and tension resistance of cartridge fired pins loaded in tension.

Table 8.4: Design resistances for bolts

Bolts loaded in shear:

Bearing resistance: [2)]

$F_{b,Rd} = 2{,}5 \alpha_b \, k_t \, f_u \, d \, t \, / \, \gamma_{M2}$ with α_b is the smallest of 1,0 or $e_1 / (3d)$ and

$k_t = (0{,}8\, t + 1{,}5) / 2{,}5$ for 0,75 mm $\leq t \leq$ 1,25 mm; $k_t = 1{,}0$ for $t >$ 1,25 mm

Net-section resistance:

$F_{n,Rd} = (1 + 3\, r\, (d_o / u - 0{,}3))\, A_{net}\, f_u / \gamma_{M2}$ but $F_{n,Rd} \leq A_{net}\, f_u / \gamma_{M2}$

with:

r = [number of bolts at the cross-section] / [total number of bolts in the connection]

$u = 2\, e_2$ but $u \leq p_2$

Shear resistance:

- for strength grades 4.6, 5.6 and 8.8:

$F_{v,Rd} = 0{,}6\, f_{ub}\, A_s / \gamma_{M2}$

- for strength grades 4.8, 5.8, 6.8 and 10.9:

$F_{v,Rd} = 0{,}5\, f_{ub}\, A_s / \gamma_{M2}$

Conditions: [3)] $F_{v,Rd} \geq 1{,}2\, \Sigma\, F_{b,Rd} \cancel{/(n_f \beta_{Lf})}$ or $\Sigma F_{v,Rd} \geq 1{,}2\, F_{n,Rd}$

Bolts loaded in tension:

Pull-through resistance: Pull-through resistance $F_{p,Rd}$ to be determined by testing *[4)].

Pull-out resistance: Not relevant for bolts.

Tension resistance: $F_{t,Rd} = 0{,}9\, f_{ub}\, A_s / \gamma_{M2}$

Conditions: [3)] $F_{t,Rd} \geq n \Sigma F_{p,Rd}$

Range of validity: [1)]

$e_1 \geq 1{,}0\, d$	$p_1 \geq 3\, d$	3 mm $> t \geq$ 0,75 mm	Minimum bolt size: M 6
$e_2 \geq 1{,}5\, d$	$p_2 \geq 3\, d$		Strength grades: 4.6 - 10.9
$f_u \leq 550$ N/mm^2			

[1)] Bolts may be used beyond this range of validity if the resistance is determined from the results of tests.

[2)] For thickness larger than or equal to 3 mm the rules for bolts in EN 1993-1-8 should be used.

[3)] The required conditions should be fulfilled when deformation capacity of the connection is needed. When these conditions are not fulfilled there should be proved that the needed deformation capacity will be provided by other parts of the structure.

NOTE*[4)] The National Annex may give further information on pull-through resistance of bolts loaded in tension.

8.4 Spot welds

(1) Spot welds may be used with as-rolled or galvanized parent material up to 4,0 mm thick, provided that the thinner connected part is not more than 3,0 mm thick.

(2) Spot welds may be either resistance welded or fusion welded.

(3) The design resistance $F_{v,Rd}$ of a spot weld loaded in shear should be determined using table 8.5.

(4) In table 8.5 the meanings of the symbols should be taken as follows:

A_{net}	is	the net cross-sectional area of the connected part;
n_w	is	the number of spot welds in one connection;
t	is	the thickness of the thinner connected part or sheet [mm];
t_1	is	the thickness of the thicker connected part or sheet;

and the end and edge distances e_1 and e_2 and the spacings p_1 and p_2 are as defined in 8.4(5).

(5) The partial factor γ_M for calculating the design resistances of spot welds ~~shall~~ should be taken as γ_{M2}.

NOTE The National Annex may chose the value of γ_{M2}. The value γ_{M2} = 1,25 is recommended.

Table 8.5: Design resistances for spot welds

Spot welds loaded in shear:
Tearing and bearing resistance: - if $t \le t_1 \le 2{,}5t$: $F_{tb,Rd} = 2{,}7\sqrt{t}\, d_s f_u / \gamma_{M2}$ [with t in mm] - if $t_1 > 2{,}5t$: $F_{tb,Rd} = 2{,}7\sqrt{t}\, d_s f_u / \gamma_{M2}$ but $F_{tb,Rd} \le 0{,}7\, d_s^2\, f_u / \gamma_{M2}$ and $F_{tb,Rd} \le 3{,}1\, t\, d_s\, f_u / \gamma_{M2}$ End resistance: $F_{e,Rd} = 1{,}4\, t\, e_1 f_u / \gamma_{M2}$ Net section resistance: $F_{n,Rd} = A_{net} f_u / \gamma_{M2}$ Shear resistance: $F_{V,Rd} = \frac{\pi}{4} d_s^2\, f_u / \gamma_{M2}$
Conditions: $F_{v,Rd} \ge 1{,}25 F_{tb,Rd}$ or $F_{v,Rd} \ge 1{,}25 F_{e,Rd}$ or $\Sigma F_{v,Rd} \ge 1{,}25 F_{n,Rd} \cancel{/ n_w}$
Range of validity:
$2d_s \le e_1 \le 6d_s$ $3d_s \le p_1 \le 8d_s$ $e_2 \le 4d_s$ $3d_s \le p_2 \le 6d_s$

(6) The interface diameter d_s of a spot weld should be determined from the following:

- for fusion welding: $d_s = 0{,}5t + 5$ mm ... (8.3a)

- for resistance welding: $d_s = 5\sqrt{t}$ [with t in mm] ... (8.3b)

(7) The value of d_s actually produced by the welding procedure should be verified by shear tests in accordance with Section 9, using single-lap test specimens as shown in figure 8.3. The thickness t of the specimen should be the same as that used in practice.

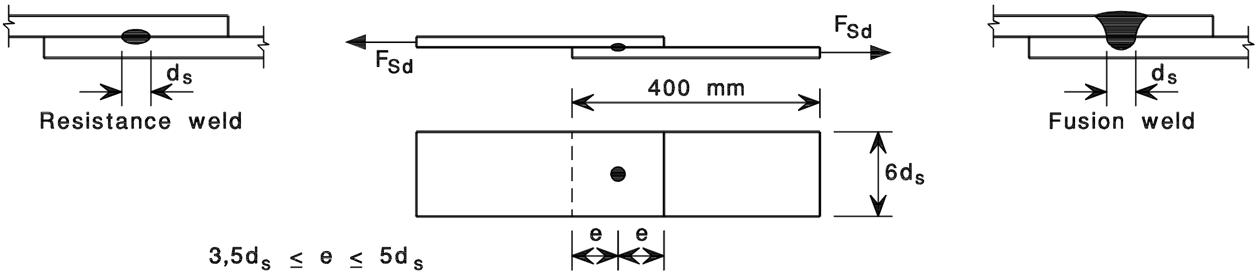


Figure 8.3: Test specimen for shear tests of spot welds

8.5 Lap welds

8.5.1 General

(1) This clause 8.6 ~~shall~~ should be used for the design of arc-welded lap welds where the parent material is 4,0 mm thick or less. For thicker parent material, lap welds ~~shall~~ should be designed using EN 1993-1-1.

(2) The weld size ~~shall~~ should be chosen such that the resistance of the connection is governed by the thickness of the connected part or sheet, rather than the weld.

(3) The requirement in (2) may be assumed to be satisfied if the throat size of the weld is at least equal to the thickness of the connected part or sheet.

(4) The partial factor γ_M for calculating the design resistances of lap welds ~~shall~~ should be taken as γ_{M2}.

NOTE The N~~A~~ational Annex may give a choice of γ_{M2}. The value $\gamma_{M2} = 1{,}25$ is recommended.

8.5.2 Fillet welds

(1) The design resistance $F_{w.Rd}$ of a fillet-welded connection should be determined from the following:

- for a side fillet that comprises one of a pair of side fillets:

$$F_{w,Rd} = t L_{w,s} (0{,}9 - 0{,}45 L_{w,s} / b) f_u / \gamma_{M2} \qquad \text{if } L_{w,s} \le b \qquad \text{... (8.4a)}$$

$$F_{w,Rd} = 0{,}45 t\, b\, f_u / \gamma_{M2} \qquad \text{if } L_{w,s} > b \qquad \text{... (8.4b)}$$

- for an end fillet:

$$F_{w,Rd} = t L_{w,e} (1 - 0{,}3 L_{w,e} / b) f_u / \gamma_{M2} \qquad [\text{for one weld and if } L_{w,s} \le b] \qquad \text{... (8.4c)}$$

where:

b is the width of the connected part or sheet, see figure 8.4;

$L_{w,e}$ is the effective length of the end fillet weld, see figure 8.4;

$L_{w,s}$ is the effective length of a side fillet weld, see figure 8.4.

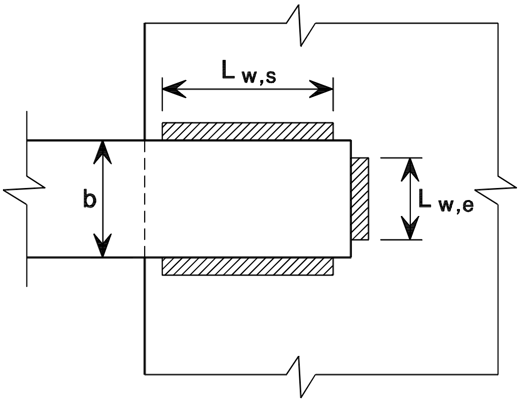


Figure 8.4: Fillet welded lap connection

(2) If a combination of end fillets and side fillets is used in the same connection, its total resistance should be taken as equal to the sum of the resistances of the end fillets and the side fillets. The position of the centroid and realistic assumption of the distribution of forces should be taken into account.

(3) The effective length L_w of a fillet weld should be taken as the overall length of the full-size fillet, including end returns. Provided that the weld is full size throughout this length, no reduction in effective length need be made for either the start or termination of the weld.

(4) Fillet welds with effective lengths less than 8 times the thickness of the thinner connected part should not be designed to transmit any forces.

8.5.3 Arc spot welds

(1) Arc spot welds ~~shall~~ should not be designed to transmit any forces other than in shear.

(2) Arc spot welds should not be used through connected parts or sheets with a total thickness Σt of more than 4 mm.

(3) Arc spot welds should have an interface diameter d_s of not less than 10 mm.

(4) If the connected part or sheet is less than 0,7 mm thick, a weld washer should be used, see figure 8.5.

(5) Arc spot welds should have adequate end and edge distances as given in the following:

(i) The minimum distance measured parallel to the direction of force transfer, from the centreline of an arc spot weld to the nearest edge of an adjacent weld or to the end of the connected part towards which the force is directed, should not be less than the value of e_{min} given by the following:

if $f_u / f_y \leq 1{,}15$

$$e_{min} = 1{,}8 \frac{F_{w,Sd}}{t f_u / \gamma_{M2}}$$

if $f_u / f_y \geq 1{,}15$

$$e_{min} = 2{,}1 \frac{F_{w,Sd}}{t f_u / \gamma_{M2}}$$

(ii) The minimum distance from the centreline of a circular arc spot weld to the end or edge of the connected sheet should not be less than $1{,}5 d_w$ where d_w is the visible diameter of the arc spot weld.

(iii) The minimum clear distance between an elongated arc spot weld and the end of the sheet and between the weld and the edge of the sheet should not be less than $1{,}0\ d_w$.

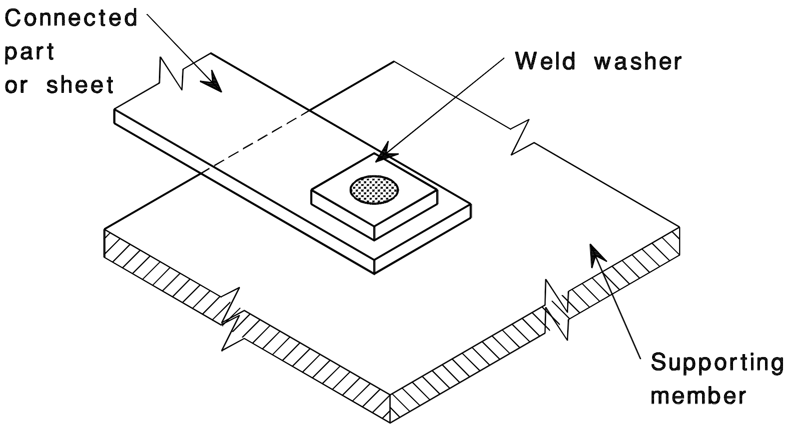


Figure 8.5: Arc spot weld with weld washer

(6) The design shear resistance $F_{w,Rd}$ of a circular arc spot weld should be determined as follows:

$$F_{w,Rd} = (\pi/4)\, d_s^2 \times 0{,}625 f_{uw} / \gamma_{M2} \quad \text{... (8.5a)}$$

where:

f_{uw} is the ultimate tensile strength of the welding electrodes;

but $F_{w,Rd}$ should not be taken as more than the peripheral resistance given by the following:

- if $d_p / \Sigma t \leq 18 (420 / f_u)^{0,5}$:

$$F_{w,Rd} = 1{,}5\, d_p \Sigma t \; f_u / \gamma_{M2} \quad \text{... (8.5b)}$$

- if $18 (420 / f_u)^{0,5} < d_p / \Sigma t < 30 (420 / f_u)^{0,5}$:

$$F_{w,Rd} = 27 (420 / f_u)^{0,5} (\Sigma t)^2 \; f_u / \gamma_{M2} \quad \text{... (8.5c)}$$

- if $d_p / \Sigma t \geq 30 (420 / f_u)^{0,5}$:

$$F_{w,Rd} = 0{,}9\, d_p \Sigma t \; f_u / \gamma_{M2} \quad \text{... (8.5d)}$$

(7) The interface diameter d_s of an arc spot weld, see figure 8.6, should be obtained from:

$$d_s = 0{,}7 d_w - 1{,}5 \Sigma t \quad \text{but} \quad d_s \geq 0{,}55\, d_w \quad \text{... (8.6)}$$

where:

d_w is the visible diameter of the arc spot weld, see figure 8.6.

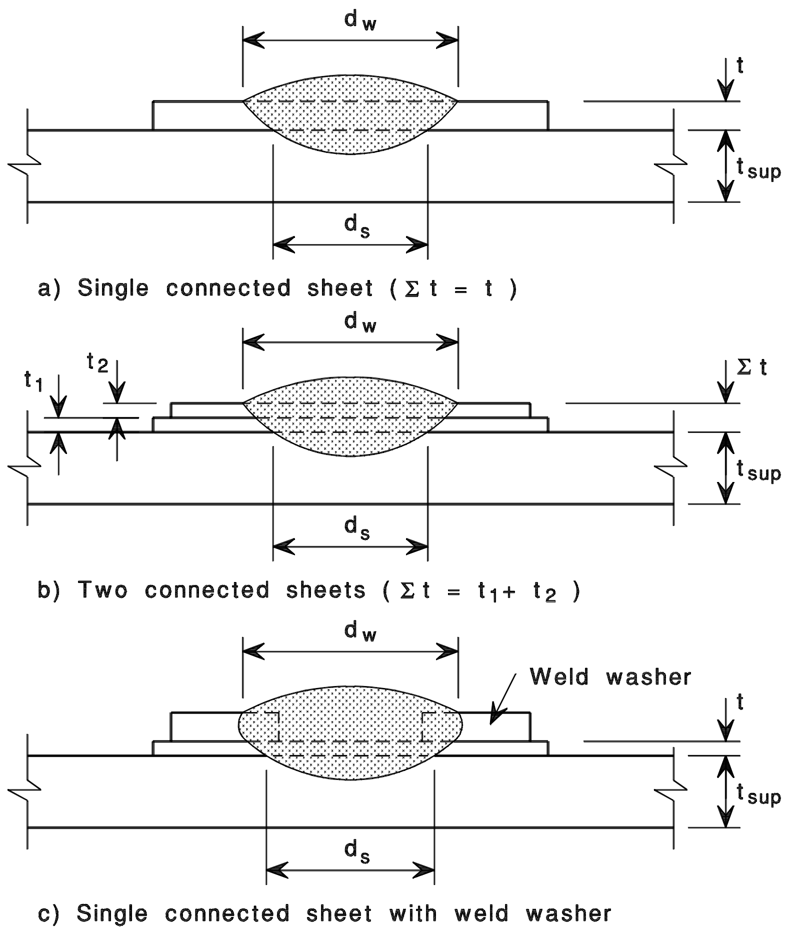


Figure 8.6: Arc spot welds

(8) The effective peripheral diameter d_p of an arc spot weld should be obtained as follows:

- for a single connected sheet or part of thickness t:

$$d_p = d_w - t \quad \text{... (8.7a)}$$

- for multiple connected sheets or parts of total thickness Σt:

$$d_p = d_w - 2\Sigma t \quad \text{... (8.7b)}$$

(9) The design shear resistance $F_{w,Rd}$ of an elongated arc spot weld should be determined from:

$$F_{w,Rd} = [(\pi/4) d_s^2 + L_w d_s] \times 0{,}625 f_{uw} / \gamma_{M2} \quad \text{... (8.8a)}$$

but $F_{w,Rd}$ should not be taken as more than the peripheral resistance given by:

$$F_{w,Rd} = (0{,}5 L_w + 1{,}67 d_p) \Sigma t f_u / \gamma_{M2} \quad \text{... (8.8b)}$$

where:

L_w is the length of the elongated arc spot weld, measured as shown in figure 8.7.

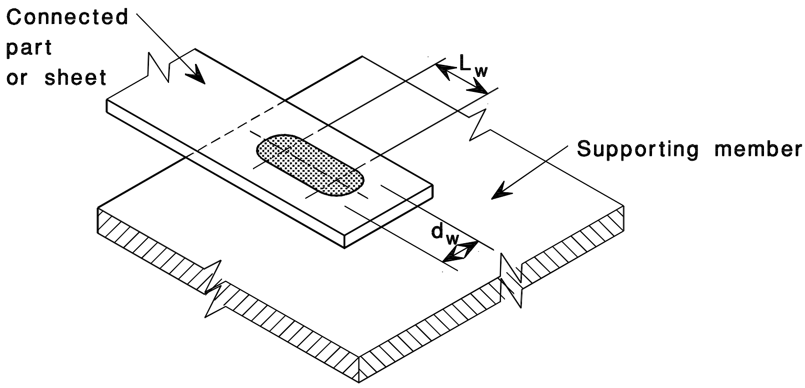


Figure 8.7: Elongated arc spot weld

9 Design assisted by testing

(1) This Section 9 may be used to apply the principles for design assisted by testing given in EN 1990 and in Section 2.5. of EN 1993-1-1, with the additional specific requirements of cold-formed ~~thin gauge~~ members and sheeting.

(2) Testing should ~~be in compliance with~~ apply the principles given in Annex A.

NOTE: The National Annex may give informations on testing.

NOTE: Annex A gives standardised procedures for:

- tests on profiled sheets and liner trays;

- tests on cold-formed members;

- tests on structures and portions of structures;

- tests on beams torsionally restrained by sheeting;

- evaluation of test results to determine design values.

(3) Tensile testing of steel should be carried out in accordance with EN 10002-1. Testing of other steel properties should be carried out in accordance with the relevant European Standards.

(4) Testing of fasteners and connections should be carried out in accordance with the relevant European Standard or International Standard.

NOTE: Pending availability of an appropriate European or International Standard, i~~m~~nformation on testing procedures for fasteners may be obtained from:

ECCS Publication No. 21 (1983): *European recommendations for steel construction: the design and testing of connections in steel sheeting and sections*;

ECCS Publication No. 42 (1983): *European recommendations for steel construction: mechanical fasteners for use in steel sheeting and sections*.

10 Special considerations for purlins, liner trays and sheetings

10.1 Beams restrained by sheeting

10.1.1 General

(1) The provisions given in this clause 10.1 may be applied to beams (called purlins in this Section) of *Z*, *C*, Σ, *U*, *Zed* and *Hat* cross-section with $h / t < 233$, $c / t \leq 20$ for single fold and $d / t \leq 20$ for double edge fold~~(other limits as in table 5.1 and clause 5.2(5) and with continuous full lateral restraint to one flange~~.

NOTE Other limits are possible if verified by tersting. The National Annex may give informations on tests. Standard tests as given in Annex A are recommended.

(2) These provisions may be used for structural systems of purlins with anti-sag bars, continuous, sleeved and overlapped systems.

(3) These provisions may also be applied to cold-formed members used as side rails, floor beams and other similar types of beam that are similarly restrained by sheeting.

(4) Side rails may be designed on the basis that wind pressure has a similar effect on them to gravity loading on purlins, and that wind suction acts on them in a similar way to uplift loading on purlins.

(5) Full continuous lateral restraint may be supplied by trapezoidal steel sheeting or other profiled steel sheeting with sufficient stiffness, continuously connected to the flange of the purlin through the troughs of the sheets. The purlin at the connection to trapetzoidal sheeting may be regarded as laterally restrained, if clause 10.1.1(6) is fulfilled. In other cases (for example, fastening through the crests of the sheets) the degree of restraint should either be validated by experience, or determined from tests.

NOTE For tests see Annex A.

(6) If the trapetzoidal sheeting is connected to a purlin and the condition expressed by the equation (10.1a) is met, the purlin at the connection may be regarded as being laterally restrained in the plane of the sheeting:

$$S \geq \left(EI_{\mathrm{w}} \frac{\pi^2}{L^2} + GI_{\mathrm{t}} + EI_{\mathrm{z}} \frac{\pi^2}{L^2} 0{,}25 h^2 \right) \frac{70}{h^2} \qquad \ldots(10.1a)$$

where

- S is the portion of the shear stiffness provided by the sheeting for the examined member connected to the sheeting at each rib (If the sheeting is connected to a purlin every second rib only, then S should be substituted by $0{,}20\,S$);
- I_{w} is the warping constant of the purlin;
- I_{t} is the torsion constant of the purlin;
- I_{z} is the second moment of area of the cross-section about the minor axis of the cross-section of the purlin;
- L is the span of the purlin;
- h is the height of the purlin.

NOTE 1: The equation (10.1a) may also be used to determine the lateral stability of member flanges used in combination with other types of cladding than trapetzoidal sheeting, provided that the connections are of suitable design.

NOTE 2: The shear stiffness S may be calculated using ECCS guidance (see NOTE in 9.1(5)) or determined by tests.

(7) Unless alternative support arrangements may be justified from the results of tests the purlin should have support details, such as cleats, that prevent rotation and lateral displacement at its supports. The effects of forces in the plane of the sheeting, that are transmitted to the supports of the purlin, should be taken into account in the design of the support details.

(8) The behaviour of a laterally restrained purlin should be modelled as outlined in figure 10.1. The connection of the purlin to the sheeting may be assumed to partially restrain the twisting of the purlin. This partial torsional restraint may be represented by a rotational spring with a spring stiffness C_{D}. The stresses in

the free flange, not directly connected to the sheeting, should then be calculated by superposing the effects of in-plane bending and the effects of torsion, including lateral bending due to cross-sectional distortion. The rotational restraint given by the sheeting should be determined following 10.1.5.

(9) Where the free flange of a single span purlin is in compression under uplift loading, allowance should also be made for the amplification of the stresses due to torsion and distortion.

(10) The shear stiffness of trapetzoidal sheeting connected to the purlin at each rib and connected in every side overlap may be calculated as

$$S = 1000\sqrt{t^3}\,(50 + 10\sqrt[3]{b_{roof}})\,\frac{s}{h_w} \qquad \text{(N)}, \qquad t \text{ and } b_{roof} \text{ in mm} \qquad \ldots(10.1b)$$

where t is the design thickness of sheeting, b_{roof} is the width of the roof, s is the distance between the purlins and h_w is the profile depth of sheeting. All dimensions are given in mm. For liner trays the shear stiffness is S_v times distance between purlins, where S_v is calculated according to 10.3.5(6).

10.1.2 Calculation methods

(1) Unless a second order analysis is carried out, the method given in 10.1.3 and 10.1.4 should be used to allow for the tendency of the free flange to move laterally (thus inducing additional stresses) by treating it as a beam subject to a lateral load $q_{h,Ed}$, see figure 10.1.

(2) For use in this method, the rotational spring should be replaced by an equivalent lateral linear spring of stiffness K. In determining K the effects of cross-sectional distortion should also be allowed for. For this purpose, the free flange may be treated as a compression member subject to a non-uniform axial force, with a continuous lateral spring support of stiffness K.

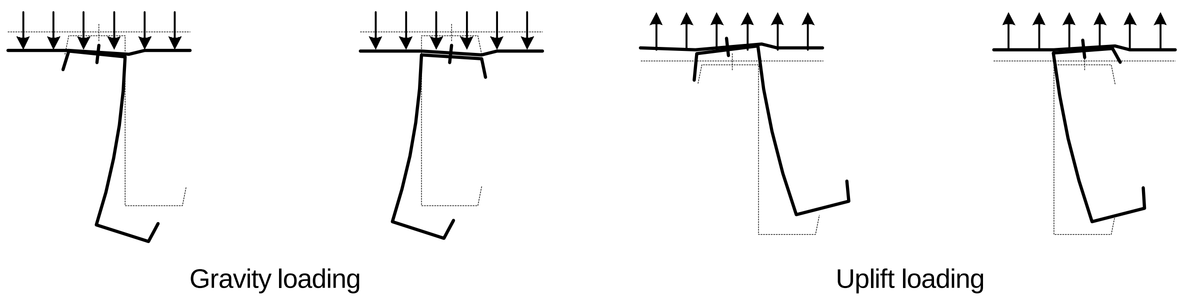
(3) If the free flange of a purlin is in compression due to in-plane bending (for example, due to uplift loading in a single span purlin), the resistance of the free flange to lateral buckling should also be verified.

(4) For a more precise calculation, a numerical analysis should be carried out, using values of the rotational spring stiffness C_D obtained from 10.1.5.2. Allowance should be made for the effects of an initial bow imperfection of (e_o) in the free flange, defined as in 5.3. The initial imperfection should be compatible with the shape of the relevant buckling mode, determined by the eigen-vectors obtained from the elastic first order buckling analysis.

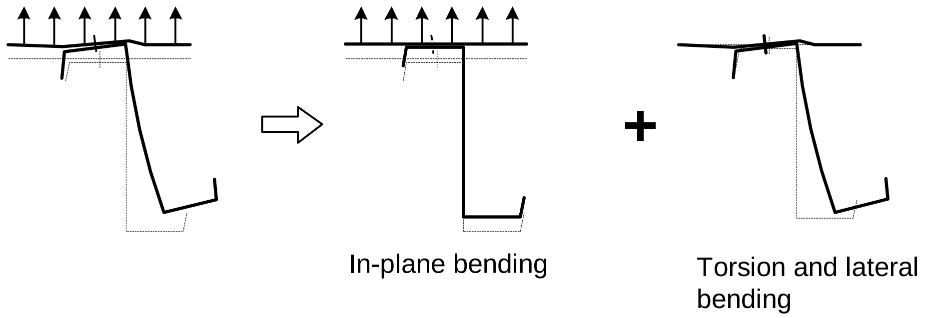
(5) A numerical analysis using the rotational spring stiffness C_D obtained from 10.1.5.2 may also be used if lateral restraint is not supplied or if its effectiveness cannot be proved. When the numerical analysis is carried out, it ~~shall~~ should take into account the bending in two directions, torsional St Venant stiffness and warping stiffness about the imposed rotation axis.

(6) If a 2nd order analysis is carried out, effective sections and stiffness, due to local buckling, ~~shall~~ should be taken into account.

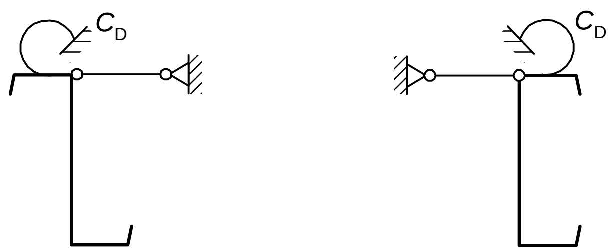
NOTE: For a simplified design of purlins made of C-, Z- and Σ- cross sections see Annex E.



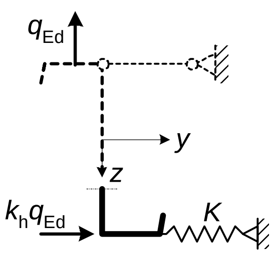
a) Z and C section purlin with upper flange connected to sheeting

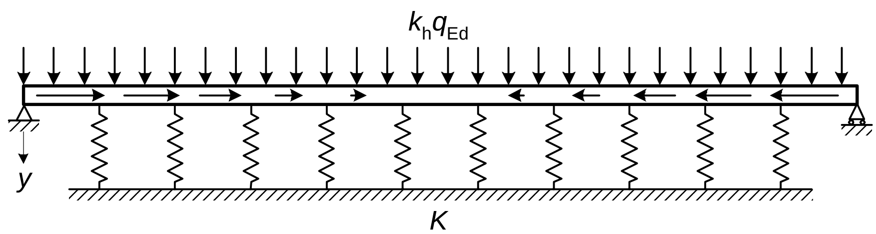


b) Total deformation split into two parts



c) Model purlin as laterally braced with rotationally spring restraint C_D from sheeting





d) As a simplification replace the rotational spring C_D by a lateral spring stiffness K

e) Free flange of purlin modelled as beam on elastic foundation. Model representing effect of torsion and lateral bending (including cross section distortion) on single span with uplift loading.

Figure 10.1: Modelling laterally braced purlins rotationally restrained by sheeting

10.1.3 Design criteria

10.1.3.1 Single span purlins

(1) For gravity loading, a single span purlin should satisfy the criteria for cross-section resistance given in 10.1.4.1. If it is subject to axial compression, it should also satisfy the criteria for stability of the free flange given in 10.1.4.2.

(2) For uplift loading, a single span purlin should satisfy the criteria for cross-section resistance given in 10.1.4.1 and the criteria for stability of the free flange given in 10.1.4.2.

10.1.3.2 ~~Purlins continuous over t~~Two~~-~~spans continuous purlins with gravity load

(1) The moments due to gravity loading in a purlin that is physically continuous over two spans without overlaps or sleeves, may either be obtained by calculation or based on the results of tests.

(2) If the moments are calculated they should be determined using elastic global analysis. The purlin should satisfy the criteria for cross-section resistance given in 10.1.4.1. For the moment at the internal support, the criteria for stability of the free flange given in 10.1.4.2 should also be satisfied. For mid-support should be checked also for bending moment + support reaction (web crippling if cleats are not used) and for bending moment + shear forces depending on the case under consideration.

(3) Alternatively the moments may be determined using the results of tests in accordance with Section 9 and Annex A.5 on the moment-rotation behaviour of the purlin over the internal support.

NOTE: Appropriate testing procedures are given in Annex A.

(4) The design value of the resistance moment at the supports $M_{sup,Rd}$ for a given value of the load per unit length q_{Ed}, should be obtained from the intersection of two curves representing the design values of:

- the moment-rotation characteristic at the support, obtained by testing in accordance with Section 9 and Annex A.5;

- the theoretical relationship between the support moment $M_{sup,Ed}$ and the corresponding plastic hinge rotation ϕ_{Ed} in the purlin over the support.

To determine the final design value of the support moment $M_{sup,Ed}$ allowance should be made for the effect of the lateral load in the free flange and/or the buckling stability of that free flange around the mid-support, which are not fully taken into account by the internal support test as given in clause A.5.2. When the free flange is physically continued at the support and if the distance between the support and the nearest anti-sag bar is larger than 0,5 s, the lateral load $q_{h,Ed}$ according to 10.1.4.2 should be taken into account in verification of the resistance at mid-support. Alternatively, full-scale tests for two or multi-span purlins may be used to determine the effect of the lateral load in the free flange and/or the buckling stability of that free flange around the mid-support.

(5) The span moments should then be calculated from the value of the support moment.

(6) The following expressions may be used for a purlin with two equal spans:

$$\phi_{Ed} = \frac{L}{12\,E\,I_{eff}}\left[\, q_{Ed}\,L^2 - 8\,M_{sup,Ed} \,\right] \qquad \text{... (10.2a)}$$

$$M_{spn,Ed} = \frac{\left(\, q_{Ed}\,L^2 - 2\,M_{sup,Rd}\,\right)^2}{8\,q_{Ed}\,L^2} \qquad \text{... (10.2b)}$$

where:

I_{eff} is the effective second moment of area for the moment $M_{spn,Ed}$;

L is the span;

$M_{spn,Ed}$ is the maximum moment in the span.

(7) The expressions for a purlin with two unequal spans, and for non-uniform loading (e.g. snow accumulation), and for other similar cases, the formulas (10.2a) and (10.2b) are not valid and approriate analysis should be made for these cases.

(8) The maximum span moment $M_{spn,Ed}$ in the purlin should satisfy the criteria for cross-section resistance given in 10.1.4.1. Alternatively the resistance moment in the span may be determined by testing. A ~~using~~ single span test~~s~~ may be used with a span comparable to the distance between the points of contraflexure in the span.

10.1.3.3 Two-span continuous purlins with uplift loading

(1) The moments due to uplift loading in a purlin that is physically continuous over two spans without overlaps or sleeves, should be determined using elastic global analysis.

(2) The moment over the internal support should satisfy the criteria for cross-section resistance given in 10.1.4.1. Because the support reaction is a tensile force, no account need be taken of its interaction with the support moment. The mid-support should be checked also for bending moment + shear forces.

(3) The moments in the spans should satisfy the criteria for stability of the free flange given in 10.1.4.2.

10.1.3.4 Purlins with semi-continuity given by overlaps or sleeves

(1) The moments in purlins in which continuity over two or more spans is given by overlaps or sleeves at internal supports, should be determined taking into account the effective section properties of the cross-section and the effects of the overlaps or sleeves.

(2) Tests may be carried out on the support details to determine:

- the flexural stiffness of the overlapped or sleeved part;
- the moment-rotation characteristic for the overlapped or sleeved part. Note, that only when the failure occurs at the support with cleat or similar preventing lateral displacements at the support, then the plastic redistribution of bending moments may be used for sleeved and overlapped systems;
- the resistance of the overlapped or sleeved part to combined support reaction and moment;
- the resistance of the non-overlapped unsleeved part to combined shear force and bending moment.

Alternatively the characteristics of the mid-support details may be determined by numerical methods if the design procedure is at least validated by a relevant numbers of tests.

(3) For gravity loading, the purlin should satisfy the following criteria:

- at internal supports, the resistance to combined support reaction and moment determined e.g. by calculation assisted by testing;
- near supports, the resistance to combined shear force and bending moment determined e.g. by calculation assisted by testing;
- in the spans, the criteria for cross-section resistance given in 10.1.4.1;
- if the purlin is subject to axial compression, the criteria for stability of the free flange given in 10.1.4.2.

(4) For uplift loading, the purlin should satisfy the following criteria:

- at internal supports, the resistance to combined support reaction and moment determined e.g. by calculation assisted by testing, taking into account the fact that the support reaction is a tensile force in this case;
- near supports, the resistance to combined shear force and bending moment determined e.g. by calculation assisted by testing;
- in the spans, the criteria for stability of the free flange given in 10.1.4.2;
- if the purlin is subjected to axial compression, the criteria for stability of the free flange is given in 10.1.4.2.

10.1.3.5 Serviceability criteria

(1) The serviceability criteria relevant to purlins should also be satisfied.

10.1.4 Design resistance

10.1.4.1 Resistance of cross-sections

(1) For a purlin subject to axial force and transverse load the resistance of the cross-section should be verified as indicated in figure 10.2 by superposing the stresses due to:

- the in-plane bending moment $M_{y,Ed}$;

- the axial force N_{Ed};

- an equivalent lateral load $q_{h,Ed}$ acting on the free flange, due to torsion and lateral bending, see (3).

(2) The maximum stresses in the cross-section should satisfy the following:

- restrained flange:

$$\sigma_{max,Ed} = \frac{M_{y,Ed}}{W_{eff,y}} + \frac{N_{Ed}}{A_{eff}} \leq f_y / \gamma_M \qquad \text{... (10.3a)}$$

- free flange:

$$\sigma_{max,Ed} = \frac{M_{y,Ed}}{W_{eff,y}} + \frac{N_{Ed}}{A_{eff}} + \frac{M_{fz,Ed}}{W_{fz}} \leq f_y / \gamma_M \qquad \text{... (10.3b)}$$

where:

A_{eff} is the effective area of the cross-section for only uniform compression;

f_y is the yield strength as defined in 3.2.1(5);

$M_{fz,Ed}$ is the bending moment in the free flange due to the lateral load $q_{h,Ed}$;

$W_{eff,y}$ is the effective section modulus of the cross-section for only bending about the y - y axis;

W_{fz} is the gross elastic section modulus of the free flange plus ~~0,27 of the web height for the point of web-flange intersection, for bending about the z - z axis~~ the contributing part of the web for bending about the z-z axis; unless a more sophisticated analysis is carried out the contributing part of the web may be taken equal to 1/5 of the web height from the point of web-flange intersection in case of C- and Z-sections and 1/6 of the web height in case of Σ-section , see Figure 10.2;

and $\gamma_M = \gamma_{M0}$ if $A_{eff} = A_g$ or if $W_{eff,y} = W_{el,y}$ and $N_{Ed} = 0$, otherwise $\gamma_M = \gamma_{M1}$.

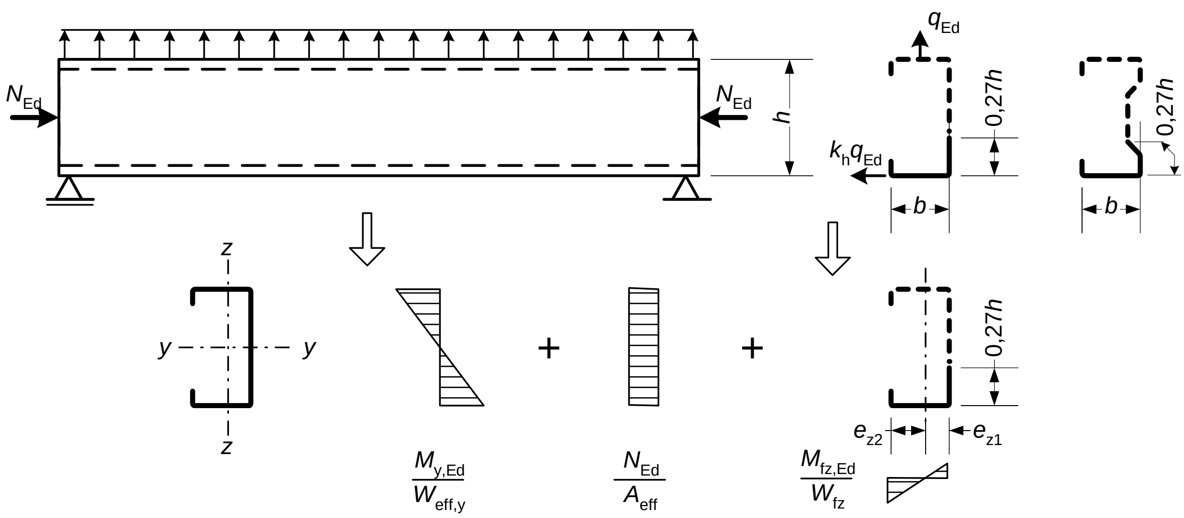


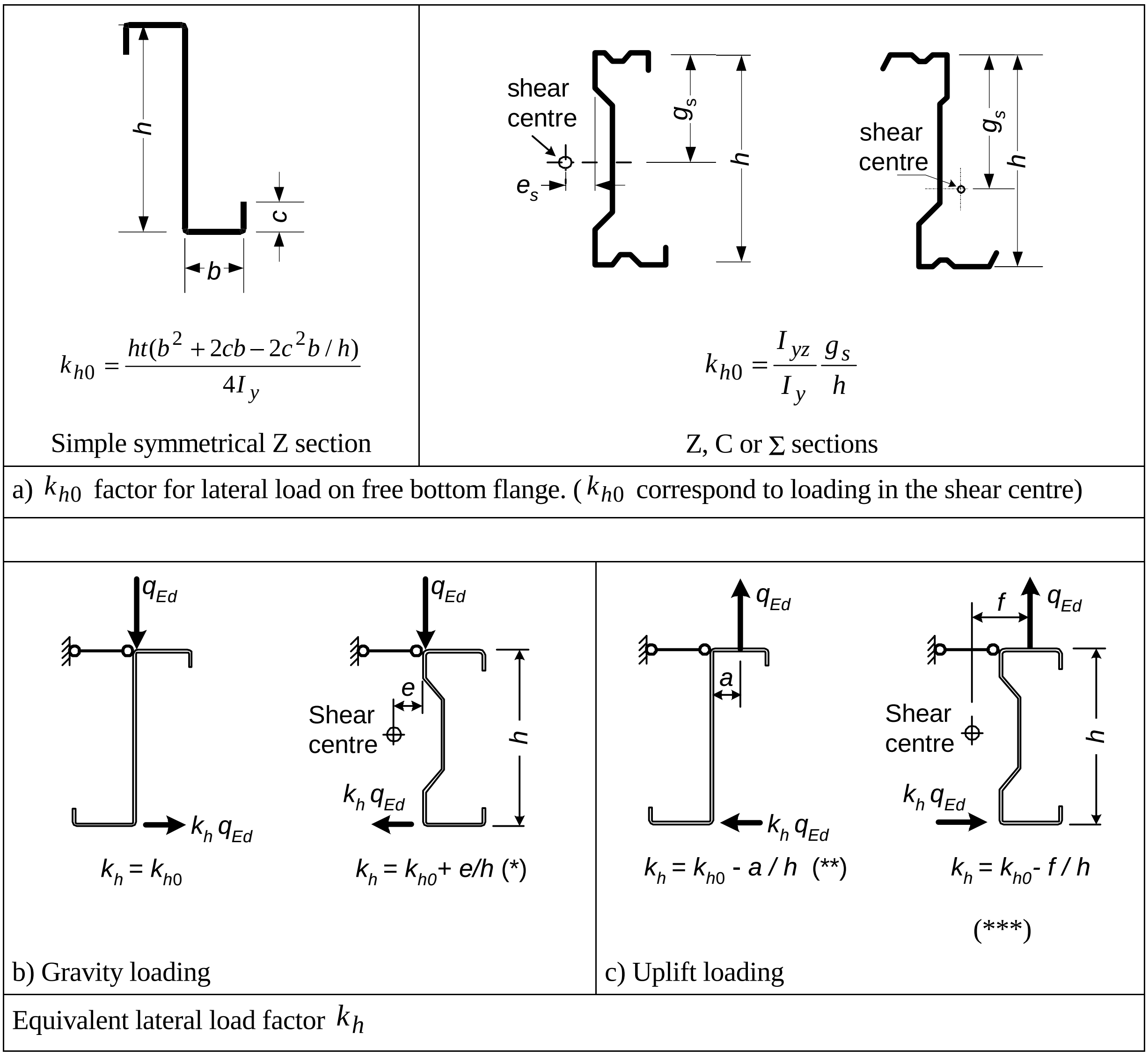
Figure 10.2: Superposition of stresses

((Figure to be modified: 0,27h to be replaced by 1/5h resp. 1/6h))

(3) The equivalent lateral load $q_{h,Ed}$ acting on the free flange, due to torsion and lateral bending, should be obtained from:

$$q_{h,Ed} = k_h \, q_{Ed} \qquad \text{... (10.4)}$$

(4) The coefficient k_h should be obtained as indicated in figure 10.3 for common types of cross-section.



a) k_{h0} factor for lateral load on free bottom flange. (k_{h0} correspond to loading in the shear centre)

b) Gravity loading

c) Uplift loading

Equivalent lateral load factor k_h

(*) If the shear centre is at the right hand side of the load q_{Ed} then the load is acting in the opposite direction.

(**) If $a/h > k_{h0}$ then the load is acting in the opposite direction.

(***) The value of f is limited to the position of the load q_{Ed} between the edges of the top flange.

Figure 10.3: Conversion of torsion and lateral bending into an equivalent lateral load $k_h \, q_{Ed}$

(5) The lateral bending moment $M_{fz,Ed}$ ~~should~~ may be determined from expression (10.5) except for a beam with the free flange in tension, where, due to positive influence of flange curling and second order effect moment $M_{fz,Ed}$ may be taken equal to zero:

$$M_{fz,Ed} = \kappa_R \, M_{0,fz,Ed} \qquad \text{... (10.5)}$$

where:

$M_{0,fz,Ed}$ is the initial lateral bending moment in the free flange without any spring support;

κ_R is a correction factor for the effective spring support.

(6) The initial lateral bending moment in the free flange $M_{0,fz,Ed}$ ~~should~~ may be determined from table 10.1 for the critical locations in the span, at supports, at anti-sag bars and between anti-sag bars. The validity of the table 10.1 is limited to the range $R \leq 40$.

(7) The correction factor κ_R for the relevant location and boundary conditions, ~~should~~ may be determined from table 10.1 (or using the theory of beams on the elastic Winkler foundation), using the value of the coefficient R of the spring support given by:

$$R = \frac{K L_a^4}{\pi^4 E I_{fz}} \quad \text{... (10.6)}$$

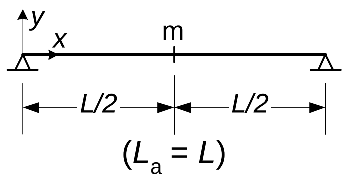
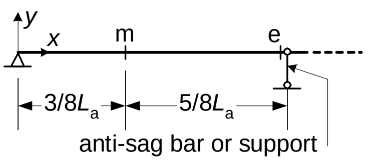
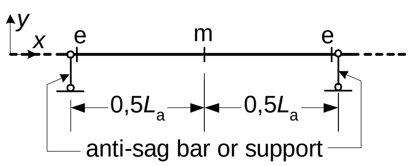
where:

I_{fz} is the second moment of area of the gross cross-section of the free flange plus ~~0,27 of the web height, for bending about the z - z axis, when numerical analysis is carried out, see 10.1.2(5);~~ the contributing part of the web for bending about the z-z axis, see 10.1.4.1(2); when numerical analysis is carried out, see 10.1.2(5);

K is the lateral spring stiffness per unit length from 10.1.5.1;

L_a is the distance between anti-sag bars, or if none are present, the span L of the purlin.

Table 10.1: Values of initial moment $M_{0,fz,Ed}$ and correction factor κ_R

System	Location	$M_{0,fz,Ed}$	κ_R
	m	$\frac{1}{8} q_{h,Ed} L_a^2$	$\kappa_R = \frac{1-0{,}0225R}{1+1{,}013R}$
	m	$\frac{9}{128} q_{h,Ed} L_a^2$	$\kappa_R = \frac{1-0{,}0141R}{1+0{,}416R}$
	e	$-\frac{1}{8} q_{h,Ed} L_a^2$	$\kappa_R = \frac{1+0{,}0314R}{1+0{,}396R}$
	m	$\frac{1}{24} q_{h,Ed} L_a^2$	$\kappa_R = \frac{1-0{,}0125R}{1+0{,}198R}$
	e	$-\frac{1}{12} q_{h,Ed} L_a^2$	$\kappa_R = \frac{1+0{,}0178R}{1+0{,}191R}$

10.1.4.2 Buckling resistance of free flange

(1) If the free flange is in compression, its buckling resistance should be verified using:

$$\frac{1}{\chi_{LT}}\left(\frac{M_{y,Ed}}{W_{eff,y}}+\frac{N_{Ed}}{A_{eff}}\right)+\frac{M_{fz,Ed}}{W_{fz}} \leq f_{yb}/\gamma_{M1} \quad \text{... (10.7)}$$

in which χ_{LT} is the reduction factor for lateral torsional buckling (flexural buckling of the free flange).

NOTE ~~The use of the χ_{LT}-value may be chosen in the Natiuonal Annex. The use of EN 1993-1-1, 6.3.2.3~~ using ~~obtained from 6.2.3. using~~ buckling curve b (~~imperfection factor~~ α_{LT} = 0,34 ; $\bar{\lambda}_{LT,0} = 0{,}4$; $\beta = 0{,}75$) is recommended for the relative slenderness $\bar{\lambda}_{fz}$ given in (2). In the case of an axial compression force N_{Ed}, when the reduction factor for buckling around the strong axis is smaller than the reduction factor for lateral flange buckling, e.g. in the case of many anti-sag bars, this failure mode should also be checked following clause 6.2.2 and 6.2.4.

(2) The relative slenderness $\bar{\lambda}_{fz}$ for flexural buckling of the free flange should be determined from:

$$\bar{\lambda}_{fz} = \frac{l_{fz}/i_{fz}}{\lambda_1} \quad \text{... (10.8)}$$

with:

$$\lambda_1 = \pi\left[E/f_{yb}\right]^{0,5}$$

where:

l_{fz} is the buckling length for the free flange from (3) to (7);

i_{fz} is the radius of gyration of the gross cross-section of the free flange plus ~~0,27 of the web height, about the z - z axis~~ the contributing part of the web for bending about the z-z axis, see 10.1.4.1(2).

(3) For gravity loading, provided that $0 \leq R \leq 200$, the buckling length of the free flange for a variation of the compressive stress over the length L as shown in figure 10.4 may be obtained from:

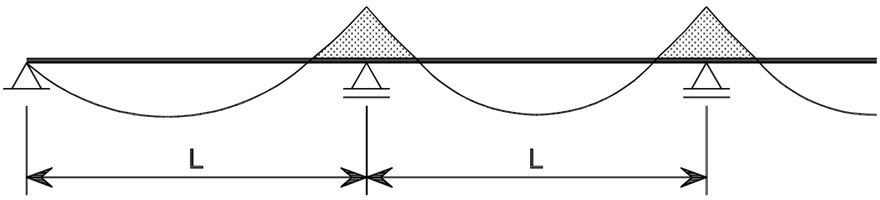
$$l_{fz} = \eta_1 L_a \left(1+\eta_2 R^{\eta_3}\right)^{\eta_4} \quad \text{... (10.9)}$$

where:

L_a is the distance between anti-sag bars, or if none are present, the span L of the purlin;

R is as given in 10.1.4.1(7);

and η_1 to η_4 are coefficients that depend on the number of anti-sag bars, as given in table 10.2a.. The tables 10.2a and 10.2b are valid only for equal spans uniformly loaded beam systems without overlap or sleeve and with anti-sag bars that provide lateral rigid support for the free flange. Due to rotations in overlap or sleeve connection, the field moment may be much larger than ~~support moment~~ without rotation which results also in longer buckling lengths in span. Neglecting the real moment distribution may lead to unsafe design. The tables may be used for systems with sleeves and overlaps provided that the connection system may be considered as fully continuous. In other cases the buckling length should be determined by more appropriate calculations or, except cantilevers, the values of the table 10.2a for the case of 3 anti-sag bars per field may be used.



[Dotted areas show regions in compression]

Figure 10.4: Varying compressive stress in free flange for gravity load cases

Table 10.2a : Coefficients η_i for down load with 0, 1, 2, 3, 4 anti-sag bars

Situation	Anti sag-bar Number	η_1	η_2	η_3	η_4
End span	0	0.414	1.72	1.11	-0.178
Intermediate span		0.657	8.17	2.22	-0.107
End span	1	0.515	1.26	0.868	-0.242
Intermediate span		0.596	2.33	1.15	-0.192
End and intermediate span	2	0.596	2.33	1.15	-0.192
End and intermediate span	3 and 4	0.694	5.45	1.27	-0.168

Table 10.2b : Coefficients η_i for uplift load with 0, 1, 2, 3, 4 anti-sag bars

Situation	Anti sag-bar Number	η_1	η_2	η_3	η_4
Simple span	0	0.694	5.45	1.27	-0.168
End span		0.515	1.26	0.868	-0.242
Intermediate span		0.306	0.232	0.742	-0.279
Simple and end spans	1	0.800	6.75	1.49	-0.155
Intermediate span		0.515	1.26	0.868	-0.242
Simple span	2	0.902	8.55	2.18	-0.111
End and intermediate spans		0.800	6.75	1.49	-0.155
Simple and end spans	3 and 4	0.902	8.55	2.18	-0.111
Intermediate span		0.800	6.75	1.49	-0.155

(4) For gravity loading, if there are more than three equally spaced anti-sag bars, under other conditions specified in (3), the buckling length need not be taken as greater than the value for two anti-sag bars, with $L_a = L/3$. This clause is valid only if there is no axial compressive force.

(5) If the compressive stress over the length L is almost constant, due to the application of a relatively large axial force, the buckling length should be determined using the values of η_i from table 10.2a for the case shown as more than three anti-sag bars per span, but the actual spacing L_a.

(6) For uplift loading, when anti-sag bars are not used, provided that $0 \leq R_0 \leq 200$, the buckling length of the free flange for variations of the compressive stress over the length L_0 as shown in figure 10.5, may be obtained from:

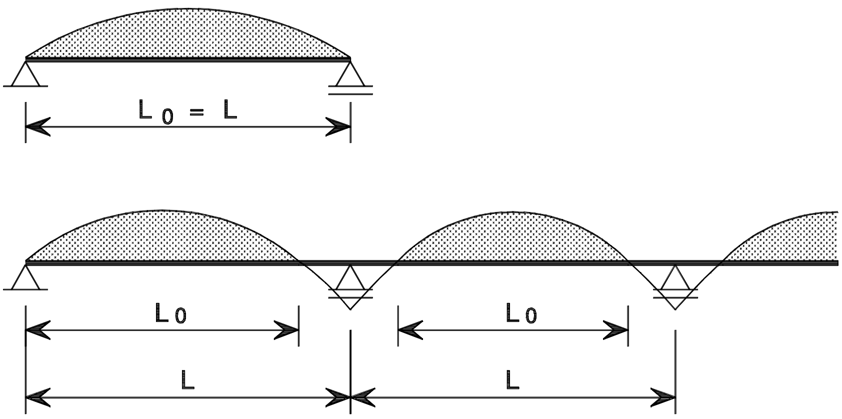
$$l_{fz} = 0{,}7\, L_0 \,(1 + 13{,}1\ R_0^{1{,}6})^{-0{,}125} \qquad \text{... (10.10a)}$$

with:

$$R_0 = \frac{K L_0^4}{\pi^4 E I_{fz}} \quad \text{... (10.10b)}$$

in which I_{fz} and K are as defined in 10.1.4.1(7). Alternatively, the buckling length of the free flange may be determined using the table 10.2b in combination with the equation given in 10.1.4.2(3).

(7) For uplift loading, if the free flange is effectively held in position laterally at intervals by anti-sag bars, the buckling length may conservatively be taken as that for a uniform moment, determined as in (5).The formula (10.10a) may be applied under conditions specified in (3). If there are no appropriate calculations, reference should be made to 10.1.4.2(5).



[Dotted areas show regions in compression]

Figure 10.5: Varying compressive stress in free flange for uplift cases

10.1.5 Rotational restraint given by the sheeting

10.1.5.1 Lateral spring stiffness

(1) The lateral spring support given to the free flange of the purlin by the sheeting should be modelled as a lateral spring acting at the free flange, see figure 10.1. The total lateral spring stiffness K per unit length should be determined from:

$$\frac{1}{K} = \frac{1}{K_A} + \frac{1}{K_B} + \frac{1}{K_C} \quad \text{... (10.11)}$$

where:

K_A is the lateral stiffness corresponding to the rotational stiffness of the connection between the sheeting and the purlin;

K_B is the lateral stiffness due to distortion of the cross-section of the purlin;

K_C is the lateral stiffness due to the flexural stiffness of the sheeting.

(2) Normally it may be assumed to be safe as well as acceptable to neglect $1/K_C$ because K_C is very large compared to K_A and K_B. The value of K should then be obtained from:

$$K = \frac{1}{(1/K_A + 1/K_B)} \quad \text{... (10.12)}$$

(3) The value of $(1/K_A + 1/K_B)$ may be obtained either by testing or by calculation.

NOTE: Appropriate testing procedures are given in Annex A.

(4) The lateral spring stiffness K per unit length may be determined by calculation using:

$$\frac{1}{K} = \frac{4\,(1-\nu^2)\;h^2\;(h_d + b_{mod})}{E\,t^3} + \frac{h^2}{C_D} \qquad \text{... (10.13)}$$

in which the dimension b_{mod} is determined as follows:

- for cases where the equivalent lateral force bringing the purlin into contact with the sheeting at the purlin web:

$b_{mod} = \quad a$

- for cases where the equivalent lateral force bringing the purlin into contact with the sheeting at the tip of the purlin flange:

$b_{mod} = \quad 2a + b$

where:

a is the distance from the sheet-to-purlin fastener to the purlin web, see figure 10.6;

b is the width of the purlin flange connected to the sheeting, see figure 10.6;

C_D is the total rotational spring stiffness from 10.1.5.2;

h is the overall height of the purlin;

h_d is the developed height of the purlin web, see figure 10.6.

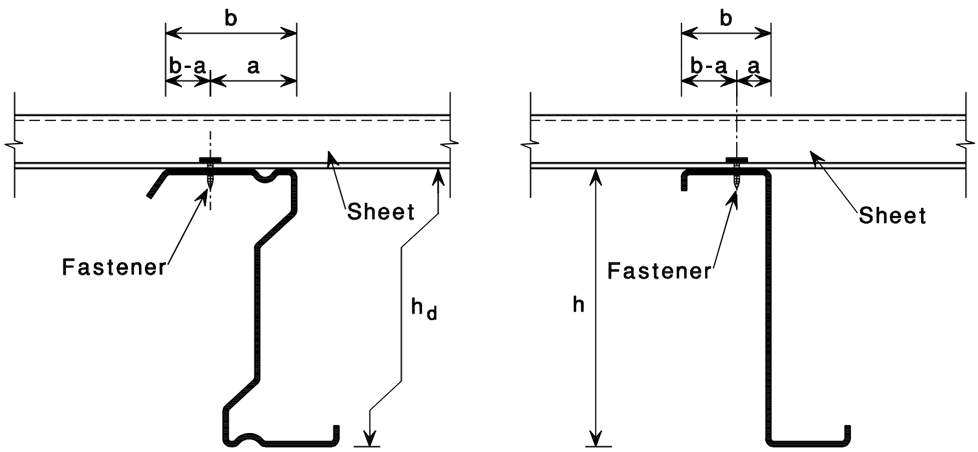


Figure 10.6: Purlin and attached sheeting

10.1.5.2 Rotational spring stiffness

(1) The rotational restraint given to the purlin by the sheeting that is connected to its top flange, should be modelled as a rotational spring acting at the top flange of the purlin, see figure 10.1. The total rotational spring stiffness C_D should be determined from:

$$C_D = \frac{1}{\left(1 / C_{D,A} + 1/C_{D,C}\right)} \qquad \text{...(10.14)}$$

where:

$C_{D,A}$ is the rotational stiffness of the connection between the sheeting and the purlin;

$C_{D,C}$ is the rotational stiffness corresponding to the flexural stiffness of the sheeting.

(2) Generally $C_{D,A}$ may be calculated as given in (5) and (7). Alternatively $C_{D,A}$ may be obtained by testing, see (9).

(3) The value of $C_{D,C}$ may be taken as the minimum value obtained from calculational models of the type shown in figure 10.7, taking account of the rotations of the adjacent purlins and the degree of continuity of the sheeting, using:

$$C_{D,C} = m / \theta \qquad \text{... (10.15)}$$

where:

~~I_{eff} is the effective second moment of area per unit width of the sheeting;~~

m is the applied moment per unit width of sheeting, applied as indicated in figure 10.7;

θ is the resulting rotation, measured as indicated in figure 10.7 [radians].

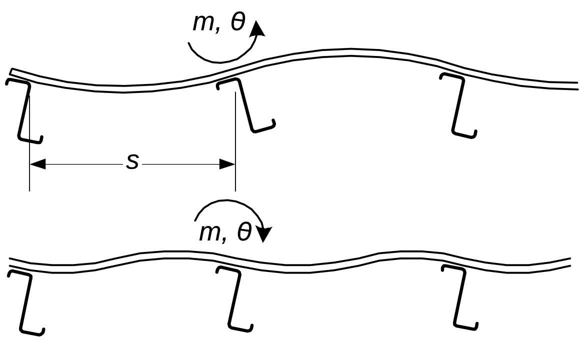


Figure 10.7: Model for calculating $C_{D,C}$

(4) Alternatively a conservative value of $C_{D,C}$ may be obtained from:

$$C_{D,C} = \frac{k\ E\ I_{eff}}{s} \qquad \text{... (10.16)}$$

in which k is a numerical coefficient, with values as follows:

- end, upper case of figure 10.7 $k = 2$;
- end, lower case of figure 10.7 $k = 3$;
- mid, upper case of figure 10.7 $k = 4$;
- mid, lower case of figure 10.7 $k = 6$;

where:

~~I_{eff} is the effective second moment of area per unit width of the sheeting;~~

s is the spacing of the purlins.

(5) Provided that the sheet-to-purlin fasteners are positioned centrally on the flange of the purlin, the value of $C_{D,A}$ for trapezoidal sheeting connected to the top flange of the purlin may be determined as follows (see table 10.3):

$$C_{D,A} = C_{100} \cdot k_{ba} \cdot k_t \cdot k_{bR} \cdot k_A \cdot k_{bT} \qquad \text{...(10.17)}$$

where

$k_{ba} = (b_a / 100)^2$ if $b_a < 125\text{mm}$;

$k_{ba} = 1{,}25(b_a / 100)$ if $125\text{mm} \le b_a < 200\text{mm}$;

$k_t = (t_{nom} / 0{,}75)^{1{,}1}$ if $t_{nom} \geq 0{,}75\text{mm}$; positive position;

$k_t = (t_{nom} / 0{,}75)^{1{,}5}$ if $t_{nom} \geq 0{,}75\text{mm}$; negative position;

$k_t = (t_{nom} / 0{,}75)^{1{,}5}$ if $t_{nom} < 0{,}75\text{mm}$;

$k_{bR} = 1{,}0$ if $b_R \leq 185\text{mm}$;

$k_{bR} = 185 / b_R$ if $b_R > 185\text{mm}$;

for gravity load:

$k_A = 1{,}0 + (A - 1{,}0) \cdot 0{,}08$ if $t_{nom} = 0{,}75\text{mm}$; positive position;

$k_A = 1{,}0 + (A - 1{,}0) \cdot 0{,}16$ if $t_{nom} = 0{,}75\text{mm}$; negative position;

$k_A = 1{,}0 + (A - 1{,}0) \cdot 0{,}095$ if $t_{nom} = 1{,}00\text{mm}$; positive position;

$k_A = 1{,}0 + (A - 1{,}0) \cdot 0{,}095$ if $t_{nom} = 1{,}00\text{mm}$; negative position;

for uplift load:

$k_A = 1{,}0$;

$k_{bT} = \sqrt{\dfrac{b_{T,max}}{b_T}}$ if $b_T > b_{T,max}$, otherwise $k_{bT} = 1$;

$A \leq 12\,kN/m$ load introduced from sheeting to beam;

where:

b_a is the width of the purlin flange [in mm];

b_R is the corrugation width [in mm];

b_T is the width of the sheeting flange through which it is fastened to the purlin;

C_{100} is a rotation coefficient, representing the value of $C_{D,A}$ if b_a = 100 mm.

(6) Provided that there is no insulation between the sheeting and the purlins, the value of the rotation coefficient C_{100} may be obtained from table 10.3.

(7) Alternatively $C_{D,A}$ may be taken as equal to $130\,p$ [Nm/m], where p is the number of sheet-to-purlin fasteners per metre length of purlin (but not more than one per rib of sheeting), provided that:

- the flange width b of the sheeting through which it is fastened does not exceed 120 mm;

- the nominal thickness t of the sheeting is at least 0,66 mm;

- the distance a or b - a between the centreline of the fastener and the centre of rotation of the purlin (depending on the direction of rotation), as shown in figure 10.6, is at least 25 mm.

(8) If the effects of cross-section distortion have to be taken into account, see 10.1.5.1, it may be assumed to be realistic to neglect $C_{D,C,}$ because the spring stiffness is mainly influenced by the value of $C_{D,A}$ and the cross-section distortion.

(9) Alternatively, values of $C_{D,A}$ may be obtained from a combination of testing and calculation.

(10)If the value of $(1 / K_A + 1 / K_B)$ is obtained by testing (in *mm/N* in accordance with A.5.3(3)), the values of

$C_{D,A}$ for gravity loading and for uplift loading should be determined from:

$$C_{D,A} = \frac{h^2 / l_A}{(1/K_A + 1/K_B) - 4(1-\nu^2)\,h^2(h_d + b_{mod})/(E t^3 l_B)} \qquad \text{... (10.18)}$$

in which b_{mod}, h and h_d are as defined in 10.1.5.1(4) and l_A is the modular width of tested sheeting and l_B is the length of tested beam.

NOTE For testing see Annex A.5.3(3).

Table 10.3: Rotation coefficient C_{100} for trapezoidal steel sheeting

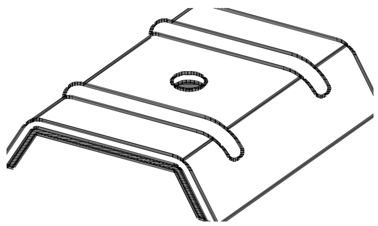
Positioning of sheeting		Sheet fastened through		Pitch of fasteners		Washer diameter [mm]	C_{100}	$b_{T,max}$
Positive 1)	Negative1)	Trough	Crest	$e = b_R$	$e = 2b_R$		[kNm/m]	[mm]
For gravity loading:								
×		×		×		22	5,2	40
×		×			×	22	3,1	40
	×		×	×		K_a	10,0	40
	×		×		×	K_a	5,2	40
	×	×		×		22	3,1	120
	×	×			×	22	2,0	120
For uplift loading:								
×		×		×		16	2,6	40
×		×			×	16	1,7	40

Key:

b_R is the corrugation width;

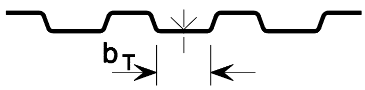
b_T is the width of the sheeting flange through which it is fastened to the purlin.

K_a indicates a steel saddle washer as shown below with $t \geq 0,75$ mm

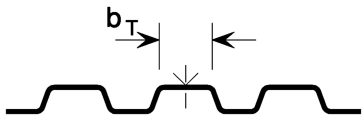


Sheet fastened:

- through the trough:



- through the crest:



The values in this table are valid for:

- sheet fastener screws of diameter: $\phi = 6,3$ mm;
- steel washers of thickness: $t_w \geq 1,0$ mm.

1) The position of sheeting in positive when the narrow flange is on the purlin and negative when the wide flange is on the purlin.

10.1.6 Forces in sheet/purlin fasteners and reaction forces

(1) Fasteners fixing the sheeting to the purlin ~~shall~~ should be checked for a combination of shear force $q_s\, e$, perpendicular to the flange, and tension force $q_t\, e$ where q_s and q_t may be calculated using table 10.4 and e is the pitch of the fasteners. Shear force due to stabilising effect, see EN1993-1-1, ~~shall~~ should be added to the shear force. Furthermore, shear force due to diaphragm action, acting parallel to the flange, ~~shall~~ should be

vectorially added to q_s.

Table 10.4 Shear force and tensile force in fastener along the beam

Beam and loading	Shear force per unite length q_s	Tensile force per unit length q_t
Z-beam, gravity loading	$(1+\xi)k_h q_{Ed}$, may be taken as 0	0
Z-beam, uplift loading	$(1+\xi)(k_h - a/h)q_{Ed}$	$\lvert \xi k_h q_{Ed} h/a \rvert + q_{Ed}$ $(a \cong b/2)$
C-beam, gravity loading	$(1-\xi)k_h q_{Ed}$	$\xi k_h q_{Ed} h/a$
C-beam, uplift loading	$(1-\xi)(k_h - a/h)q_{Ed}$	$\xi k_h q_{Ed} h/(b-a) + q_{Ed}$

(2) The fasteners fixing the purlins to the supports ~~shall~~ should be checked for the reaction force R_w in the plane of the web and the transverse reaction forces R_1 and R_2 in the plane of the flanges, see figure 10.8. Forces R_1 and R_2 may be calculated using table 10.5. Force R2 ~~shall~~ should also include loads parallel to the roof for sloped roofs. If R_1 is positive there is no tension force on the fastener. R_2 should be transferred from the sheeting to the top flange of the purlin and further on to the rafter (main beam) through the purlin/rafter connection (support cleat) or via special shear connectors or directly to the base or similar element. The reaction forces at an inner support of a continuous purlin may be taken as 2,2 times the values given in table 10.5.

NOTE: For sloped roofs the transversal loads to the purlins are the perpendicular (to the roof plane) components of the vertical loads and parallel components of the vertical loads are acting on the roof plane.

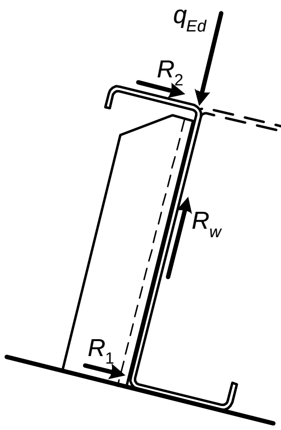


Figure 10.8: Reaction forces at support

Table 10.5 Reaction force at support for simply supported beam

Beam and loading	Reaction force on bottom flange R_1	Reaction force on top flange R_2
Z-beam, gravity loading	$(1-\varsigma)k_h q_{Ed} L/2$	$(1+\varsigma)k_h q_{Ed} L/2$
Z-beam, uplift loading	$-(1-\varsigma)k_h q_{Ed} L/2$	$-(1+\varsigma)k_h q_{Ed} L/2$
C-beam, gravity loading	$-(1-\varsigma)k_h q_{Ed} L/2$	$(1-\varsigma)k_h q_{Ed} L/2$
C-beam, uplift loading	$(1-\varsigma)k_h q_{Ed} L/2$	$-(1-\varsigma)k_h q_{Ed} L/2$

(3) The factor ζ may be taken as $\zeta = \sqrt[3]{\kappa_R}$, where κ_R = correction factor given in table 10.1, and the factor ξ may be taken as $\xi = \sqrt[3]{\zeta}$.

10.2 Liner trays restrained by sheeting

10.2.1 General

(1) Liner trays should be large channel-type sections, with two narrow flanges, two webs and one wide flange, generally as shown in figure 10.9. The two narrow flanges should be laterally restrained by attached profiled steel sheeting.

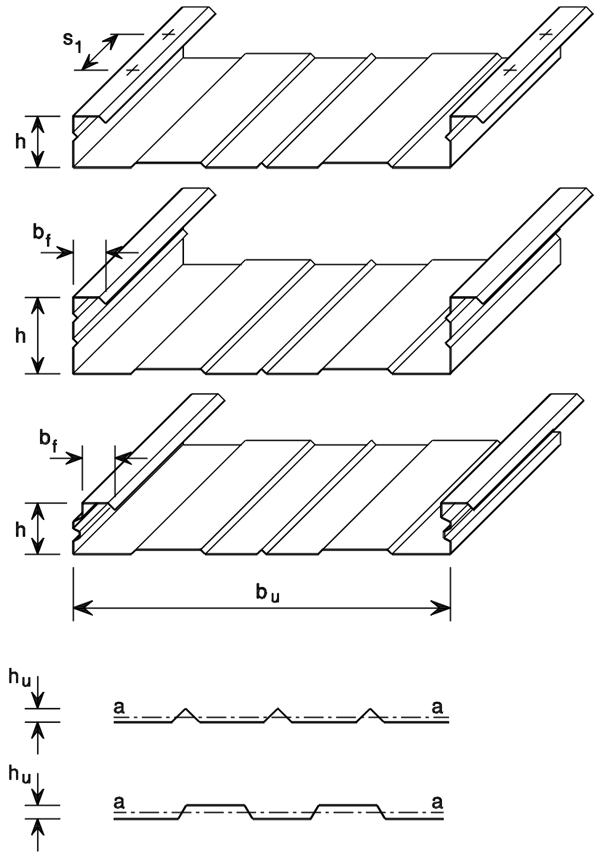


Figure 10.9: Typical geometry for liner trays

(2) The resistance of the webs of liner trays to shear forces and to local transverse forces should be obtained using 6.1.5 to 6.1.11, but using the value of $M_{c,Rd}$ given by (3) or (4).

(3) The moment resistance $M_{c,Rd}$ of a liner tray may be obtained using 10.2.2 provided that:

- the geometrical properties are within the range given in table 10.6;

- the depth h_u of the corrugations of the wide flange does not exceed $h/8$, where h is the overall depth of the liner tray.

(4) Alternatively the moment resistance of a liner tray may be determined by testing provided that it is ensured that the local behaviour of the liner tray is not affected by the testing equipment.

NOTE: Appropriate testing procedures are given in annex A.

Table 10.6: Range of validity of 10.2.2

0,75 mm	$\leq$	t_{nom}	$\leq$	1,5 mm
30 mm	$\leq$	b_f	$\leq$	60 mm
60 mm	$\leq$	h	$\leq$	200 mm
300 mm	$\leq$	b_u	$\leq$	600 mm
		I_a / b_u	$\leq$	10 mm^4 / mm
		s_1	$\leq$	1000 mm

10.2.2 Moment resistance

10.2.2.1 Wide flange in compression

(1) The moment resistance of a liner tray with its wide flange in compression should be determined using the step-by-step procedure outlined in figure 10.10 as follows:

- **Step 1:** Determine the effective areas of all compression elements of the cross-section, based on values of the stress ratio $\psi = \sigma_2 / \sigma_1$ obtained using the effective widths of the compression flanges but the gross areas of the webs;
- **Step 2:** Find the centroid of the effective cross-section, then obtain the moment resistance $M_{c,Rd}$ from:

$$M_{c,Rd} = 0,8\, W_{eff,min} f_{yb} / \gamma_{M0} \qquad \text{... (10.19)}$$

with:

$$W_{eff,min} = I_{y,eff} / z_c \quad \text{but} \quad W_{eff,min} \leq I_{y,eff} / z_t;$$

where z_c and z_t are as indicated in figure 10.10.

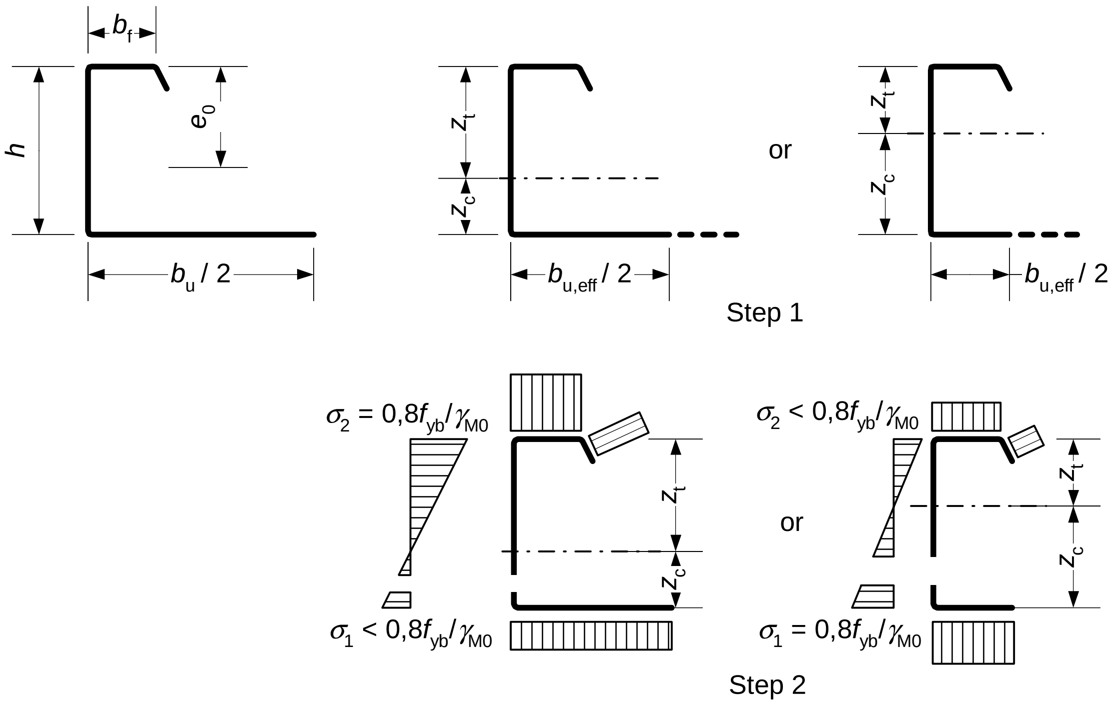


Figure 10.10: Determination of moment resistance — wide flange in compression

10.2.2.2 Wide flange in tension

(1) The moment resistance of a liner tray with its wide flange in tension should be determined using the step-by-step procedure outlined in figure 10.11 as follows:

- **Step 1:** Locate the centroid of the gross cross-section;

- **Step 2:** Obtain the effective width of the wide flange $b_{u,eff}$, allowing for possible flange curling, from:

$$b_{u,eff} = \frac{53{,}3 \cdot 10^{10}\ e_o^{\ 2}\ t^3\ t_{eq}}{h\ L\ b_u^{\ 3}} \quad \text{... (10.20)}$$

where:

b_u is the overall width of the wide flange;

e_o is the distance from the centroidal axis of the gross cross-section to the centroidal axis of the narrow flanges;

h is the overall depth of the liner tray;

L is the span of the liner tray;

t_{eq} is the equivalent thickness of the wide flange, given by:

$$t_{eq} = (12\, I_a / b_u)^{1/3}$$

I_a is the second moment of area of the wide flange, about its own centroid, see figure 10.9.

- **Step 3:** Determine the effective areas of all the compression elements, based on values of the stress ratio $\psi = \sigma_2 / \sigma_1$ obtained using the effective widths of the flanges but the gross areas of the webs;
- **Step 4:** Find the centroid of the effective cross-section, then obtain the buckling resistance moment $M_{b,Rd}$ using:

$$M_{b,Rd} = 0{,}8\, \beta_b\, W_{eff,com}\, f_{yb} / \gamma_{M0} \quad \text{but} \quad M_{b,Rd} \leq 0{,}8\, W_{eff,t}\, f_{yb} / \gamma_{M0} \quad \text{... (10.21)}$$

with:

$$W_{eff,com} = I_{y,eff} / z_c$$

$$W_{eff,t} = I_{y,eff} / z_t$$

in which the correlation factor β_b is given by the following:

- if $s_1 \leq 300$ mm:

$$\beta_b = 1{,}0$$

- if $300 \text{ mm} \leq s_1 \leq 1000$ mm:

$$\beta_b = 1{,}15 - s_1 / 2000$$

where:

s_1 is the longitudinal spacing of fasteners supplying lateral restraint to the narrow flanges, see figure 10.9.

(2) The effects of shear lag need not be considered if $L / b_{u,eff} \geq$ 25~~20~~. Otherwise a reduced value of ρ should be determined as specified in 6.1.4.3.

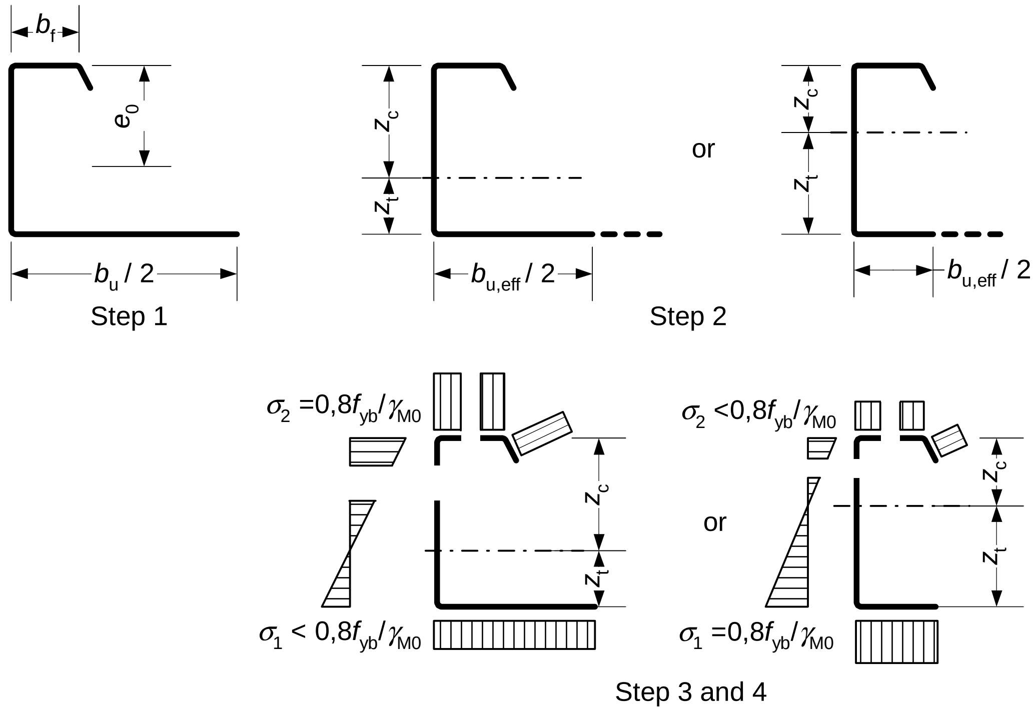


Figure 10.11: Determination of moment resistance — wide flange in tension

(3) Flange curling need not be taken into account in determining deflections at serviceability limit states.

(4) As a simplified alternative, the moment resistance of a liner tray with an unstiffened wide flange may be approximated by taking the same effective area for the wide flange in tension as for the two narrow flanges in compression combined.

10.3 Stressed skin design

10.3.1 General

(1) The interaction between structural members and sheeting panels that are designed to act together as parts of a combined structural system, may be allowed for as described in this clause 10.3.

(2) The provisions given in this clause ~~shall~~ should be applied only to sheet diaphragms that are made of steel.

(3) Diaphragms may be formed from profiled sheeting used as roof or wall cladding or for floors. They may also be formed from wall or roof structures based upon liner trays.

NOTE: Information on the verification of such diaphragms may be obtained from:

ECCS Publication No. 88 (1995): *European recommendations for the application of metal sheeting acting as a diaphragm.*

10.3.2 Diaphragm action

(1) In stressed skin design, advantage may be taken of the contribution that diaphragms of sheeting used as roofing, flooring or wall cladding make to the overall stiffness and strength of the structural frame, by means of their stiffness and strength in shear.

(2) Roofs and floors may be treated as deep plate girders extending throughout the length of a building, resisting transverse in-plane loads and transmitting them to end gables, or to intermediate stiffened frames. The panel of sheeting may be treated as a web that resists in-plane transverse loads in shear, with the edge members acting as flanges that resist axial tension and compression forces, see figures 10.12 and 10.13.

(3) Similarly, rectangular wall panels may be treated as bracing systems that act as shear diaphragms to resist in-plane forces.

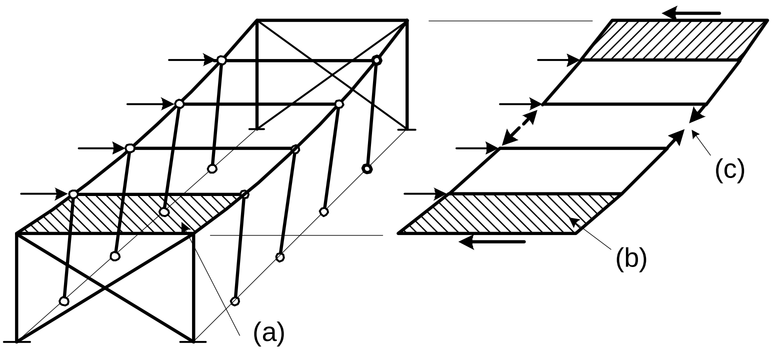


Figure 10.12: Stressed skin action in a flat-roof building

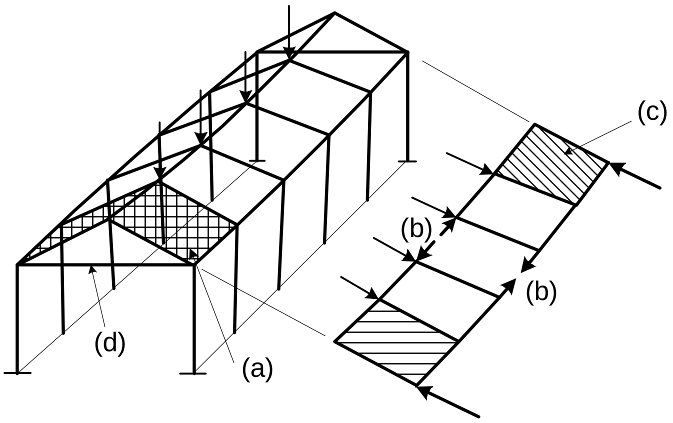
10.3.3 Necessary conditions

(1) Methods of stressed skin design that utilize sheeting as an integral part of a structure, may be used only under the following conditions:

- the use made of the sheeting, in addition to its primary purpose, is limited to the formation of shear diaphragms to resist structural displacement in the plane of that sheeting;
- the diaphragms have longitudinal edge members to carry flange forces arising from diaphragm action;
- the diaphragm forces in the plane of a roof or floor are transmitted to the foundations by means of braced frames, further stressed-skin diaphragms, or other methods of sway resistance;
- suitable structural connections are used to transmit diaphragm forces to the main steel framework and to join the edge members acting as flanges;
- the sheeting is treated as a structural component that cannot be removed without proper consideration;
- the project specification, including the calculations and drawings, draws attention to the fact that the building is designed to utilize stressed skin action;
- in sheeting with the corrugation oriented in the longitudinal direction of the roof the flange forces due to diaphragm action may be taken up by the sheeting.

(2) Stressed skin design may be used predominantly in low-rise buildings, or in the floors and facades of high-rise buildings.

(3) Stressed skin diaphragms may be used predominantly to resist wind loads, snow loads and other loads that are applied through the sheeting itself. They may also be used to resist small transient loads, such as surge from light overhead cranes or hoists on runway beams, but may not be used to resist permanent external loads, such as those from plant.



(a) Sheeting

(b) Flange forces in edge members

(c) Shear field in sheeting

(d) Gable tie required to resist forces in roof sheeting

Figure 10.13: Stressed skin action in a pitched roof building

10.3.4 Profiled steel sheet diaphragms

(1) In a profiled steel sheet diaphragm, see figure 10.14, both ends of the sheets ~~shall~~ should be attached to the supporting members by means of self-tapping screws, cartridge fired pins, welding, bolts or other fasteners of a type that will not work loose in service, pull out, or fail in shear before causing tearing of the sheeting. All such fasteners ~~shall~~ should be fixed directly through the sheeting into the supporting member, for example through the troughs of profiled sheets, unless special measures are taken to ensure that the connections effectively transmit the forces assumed in the design.

(2) The seams between adjacent sheets should be fastened by rivets, self-drilling screws, welds, or other fasteners of a type that will not work loose in service, pull out, or fail in shear before causing tearing of the sheeting. The spacing of such fasteners should not exceed 500 mm.

(3) The distances from all fasteners to the edges and ends of the sheets ~~shall~~ should be adequate to prevent premature tearing of the sheets.

(4) Small randomly arranged openings, up to 3% of the relevant area, may be introduced without special calculation, provided that the total number of fasteners is not reduced. Openings up to 15% of the relevant area (the area of the surface of the diaphragm taken into account for the calculations) may be introduced if justified by detailed calculations. Areas that contain larger openings should be split into smaller areas, each with full diaphragm action.

(5) All sheeting that also forms part of a stressed-skin diaphragm ~~shall~~ should first be designed for its primary purpose in bending. To ensure that any deterioration of the sheeting would be apparent in bending before the resistance to stressed skin action is affected, it should then be verified that the shear stress due to diaphragm action does not exceed $0,25 f_{yb}/\gamma_{M1}$.

(6) The shear resistance of a stressed-skin diaphragm ~~shall~~ should be based on the least tearing strength of the seam fasteners or the sheet-to-member fasteners parallel to the corrugations or, for diaphragms fastened only to longitudinal edge members, the end sheet-to-member fasteners. The calculated shear resistance for any other type of failure should exceed this minimum value by at least the following:

- for failure of the sheet-to-purlin fasteners under combined shear and wind uplift, by at least 40%;

- for any other type of failure, by at least 25%.

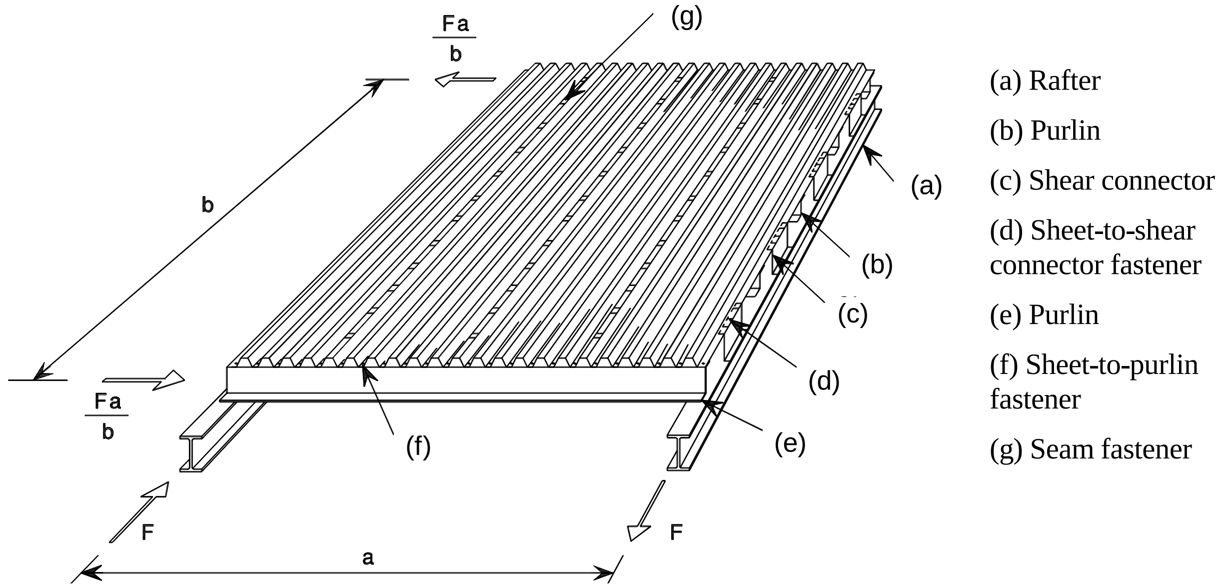


Figure 10.14: Arrangement of an individual panel

10.3.5 Steel liner tray diaphragms

(1) Liner trays used to form shear diaphragms should have stiffened wide flanges.

(2) Liner trays in shear diaphragms should be inter-connected by seam fasteners through the web at a spacing e_s of not more than 300 mm by seam fasteners (normally blind rivets) located at a distance e_u from the wide flange of not more than 30 mm, all as shown in figure 10.15.

(3) An accurate evaluation of deflections due to fasteners may be made using a similar procedure to that for trapezoidal profiled sheeting.

(4) The shear flow $T_{v,Ed}$ due to ultimate limit states design loads should not exceed $T_{v,Rd}$ given by:

$$T_{V,Rd} = 8{,}43\ E \sqrt[4]{I_a \left(t / b_u \right)^9} \qquad \text{... (10.22)}$$

where:

I_a is the second moment of area of the wide flange about it own centroid, see figure 10.9;

b_u is the overall width of the wide flange.

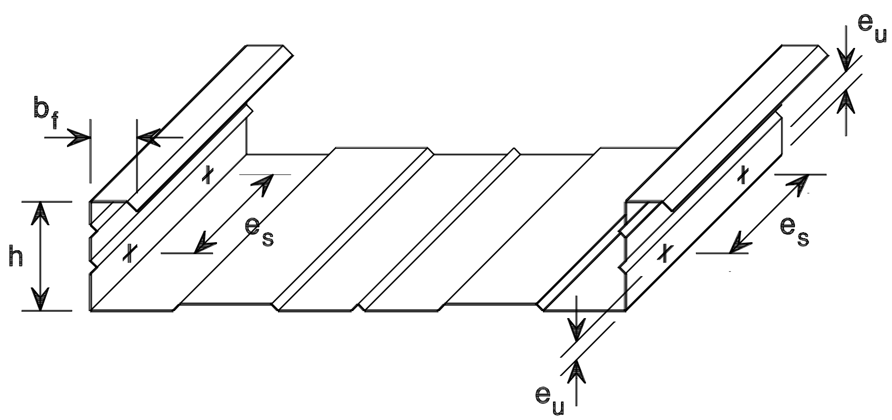


Figure 10.15: Location of seam fasteners

(5) The shear flow $T_{v,ser}$ due to serviceability design loads should not exceed $T_{v,Cd}$ given by:

$$T_{v,Cd} = S_v / 375 \quad \text{... (10.23)}$$

where:

S_v is the shear stiffness of the diaphragm, per unit length of the span of the liner trays.

(6) The shear stiffness S_v per unit length may be obtained from:

$$S_V = \frac{\alpha\ L\ b_u}{e_s\ (b - b_u)} \quad \text{... (10.24)}$$

where:

L is the overall length of the shear diaphragm (in the direction of the span of the liner trays);

b is the overall width of the shear diaphragm ($b = \Sigma\, b_u$);

α is the stiffness factor.

(7) The stiffness factor α may be conservatively be taken as equal to 2000 N/mm unless more accurate values are derived from tests.

10.4 Perforated sheeting

(1) Perforated sheeting may be designed by calculation, provided that the rules for non-perforated sheeting are modified by introducing the effective thicknesses given below.

NOTE: These calculation rules tend to give rather conservative values. More economical solutions might be obtained from design assisted by testing, see Section 9.

(2) Provided that $0,2 \leq d / a \leq 0,8$ gross section properties may be calculated using 5.1.2, but replacing t by $t_{a,eff}$ obtained from:

$$t_{a,eff} = 1{,}18\, t\ \left(1 - 0{,}9\, d\ / a\right) \quad \text{... (10.25)}$$

where:

d is the diameter of the perforations;

a is the spacing between the centres of the perforations.

(3) Provided that $0,2 \leq d / a \leq 1,0$ effective section properties may be calculated using Section 4, but replacing t by $t_{b,eff}$ obtained from:

$$t_{b,eff} = t\ \sqrt[3]{1{,}18\left(1 - d\ / a\right)} \quad \text{... (10.26)}$$

(4) The resistance of a single web to local transverse forces may be calculated using 6.1.9, but replacing t by $t_{c,eff}$ obtained from:

$$t_{c,eff} = t\ \left[1 - \left(d\ / a\right)^2 s_{per}\ / s_w\right]^{3/2} \quad \text{... (10.27)}$$

where:

s_{per} is the slant height of the perforated portion of the web;

s_w is the total slant height of the web.

Annex A [normative] – Testing procedures

A.1 General

(1) This annex A gives appropriate standardized testing and evaluation procedures for a number of tests that are required in design.

NOTE 1: In the field of cold-formed members and sheeting, many standard products are commonly used for which design by calculation might not lead to economical solutions, so it is frequently desirable to use design assisted by testing.

NOTE 2: The National Annex may give further information on testing.

NOTE 3: The National Annex may give conversion factors for existing test results to be equivalent to the outcome of standardised tests according to this annex.

(2) This annex covers:

- tests on profiled sheets and liner trays, see A.2;
- tests on cold-formed members, see A.3;
- tests on structures and portions of structures, see A.4;
- tests on torsionally restrained beams, see A.5;
- evaluation of test results to determine design values, see A.6.

A.2 Tests on profiled sheets and liner trays

A.2.1 General

(1) Although these test procedures are presented in terms of profiled sheets, similar test procedures based on the same principles may also be used for liner trays and other types of sheeting (e.g. sheeting mentioned in EN 508).

(2) Loading may be applied through air bags or in a vacuum chamber or by steel or timber cross beams arranged to approximate uniformly distributed loading.

(3) To prevent spreading of corrugations, transverse ties or other appropriate test accessories such as timber blocks may be applied to the test specimen. Some examples are given in figure A.1.

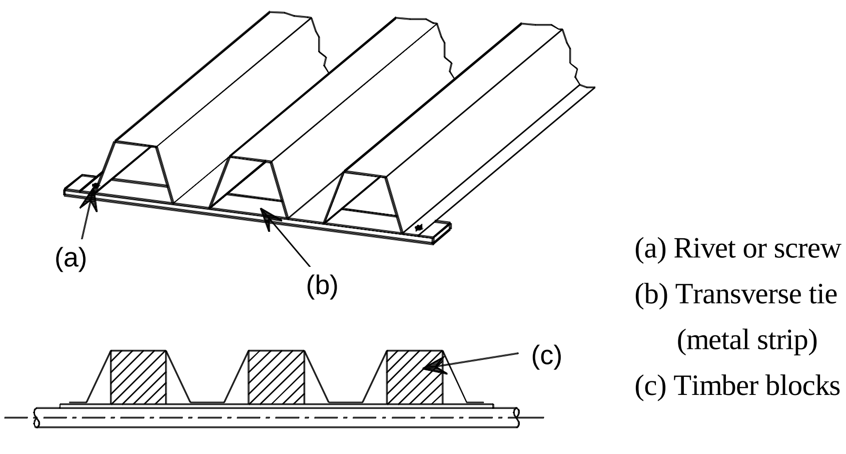


Figure A.1: Examples of appropriate test accessories

(4) For uplift tests, the test set-up should realistically simulate the behaviour of the sheeting under practical

conditions. The type of connections between the sheet and the supports should be the same as in the connections to be used in practice.

(5) To give the results a wide range of applicability, hinged and roller supports should preferably be used, to avoid any influence of torsional restraint at the supports on the test results.

(6) It should be ensured that the direction of the loading remains perpendicular to the initial plane of the sheet throughout the test procedure.

(7) To eliminate the deformations of the supports, the deflections at both ends of the test specimen should also be measured.

(8) The test result should be taken as the maximum value of the loading applied to the specimen either coincident with failure or immediately prior to failure as appropriate.

A.2.2 Single span test

(1) A test set-up equivalent to that shown in figure A.2 may be used to determine the midspan moment resistance (in the absence of shear force) and the effective flexural stiffness.

(2) The span should be chosen such that the test results represent the moment resistance of the sheet.

(3) The moment resistance should be determined from the test result.

(4) The flexural stiffness should be determined from a plot of the load-deflection behaviour.

A.2.3 Double span test

(1) The test set-up shown in figure A.3 may be used to determine the resistance of a sheet that is continuous over two or more spans to combinations of moment and shear at internal supports, and its resistance to combined moment and support reaction for a given support width.

(2) The loading should preferably be uniformly distributed (applied using an air bag or a vacuum chamber, for example).

(3) Alternatively any number of line loads (transverse to the span) may be used, arranged to produce internal moments and forces that are appropriate to represent the effects of uniformly distributed loading. Some examples of suitable arrangements are shown in figure A.4.

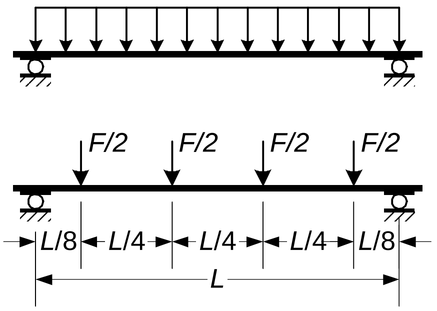
A.2.4 Internal support test

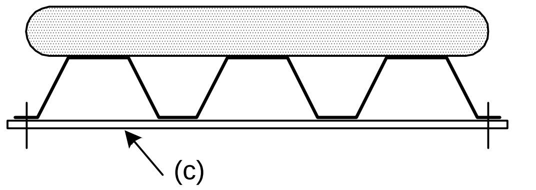
(1) As an alternative to A.2.3, the test set-up shown in figure A.5 may be used to determine the resistance of a sheet that is continuous over two or more spans to combinations of moment and shear at internal supports, and its resistance to combined moment and support reaction for a given support width.

(2) The test span s used to represent the portion of the sheet between the points of contraflexure each side of the internal support, in a sheet continuous over two equal spans L may be obtained from:

$$s = 0{,}4L \qquad \text{... (A.1)}$$

(3) If plastic redistribution of the support moment is expected, the test span s should be reduced to represent the appropriate ratio of support moment to shear force.

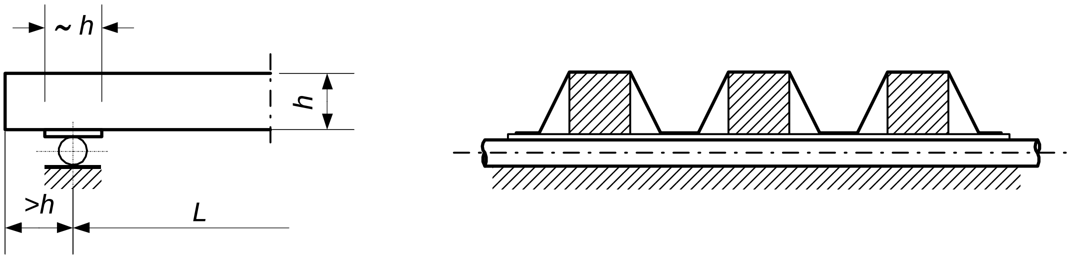




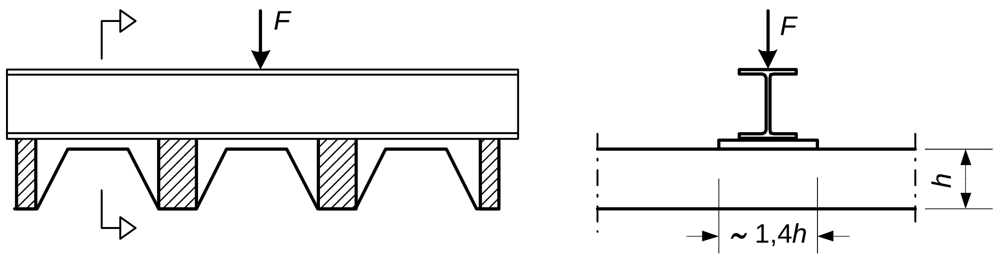
a) Uniformly distributed loading and an example of alternative equivalent line loads

b) Distributed loading applied by an airbag (alternatively by a vacuum test rig)

(c) Transverse tie



c) Example of support arrangements for preventing distortion



d) Example of method of applying a line load

Figure A.2: Test set-up for single span tests

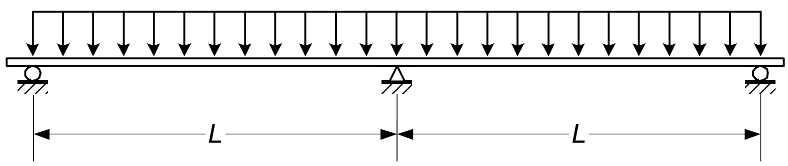


Figure A.3: Test setup for double span tests

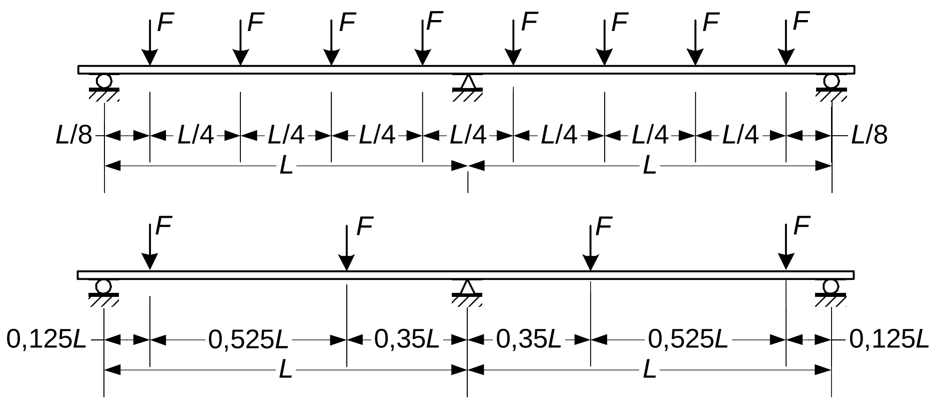


Figure A.4: Examples of suitable arrangements of alternative line loads

(4) The width b_B of the beam used to apply the test load should be selected to represent the actual support width to be used in practice.

(5) Each test result may be used to represent the resistance to combined bending moment and support reaction (or shear force) for a given span and a given support width. To obtain information about the interaction of bending moment and support reaction, tests should be carried out for several different spans.

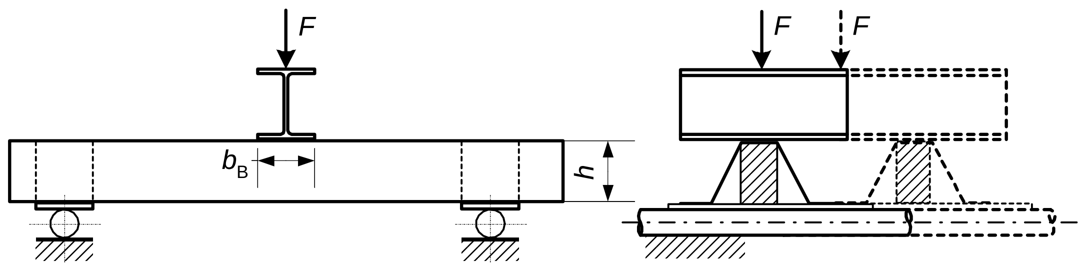
(6) Interpretation of test results, see A.5.2.3.

A.2.5 End support test

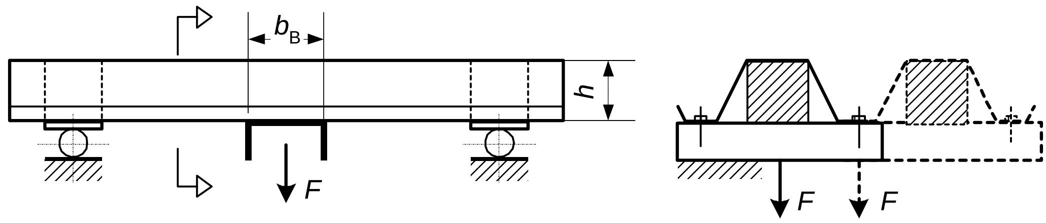
(1) The test set-up shown in figure A.6 may be used to determine the shear resistance of a sheet at an end support.

(2) Separate tests should be carried out to determine the shear resistance of the sheet for different lengths u from the contact point at the inner edge of the end support, to the actual end of the sheet, see figure A.6.

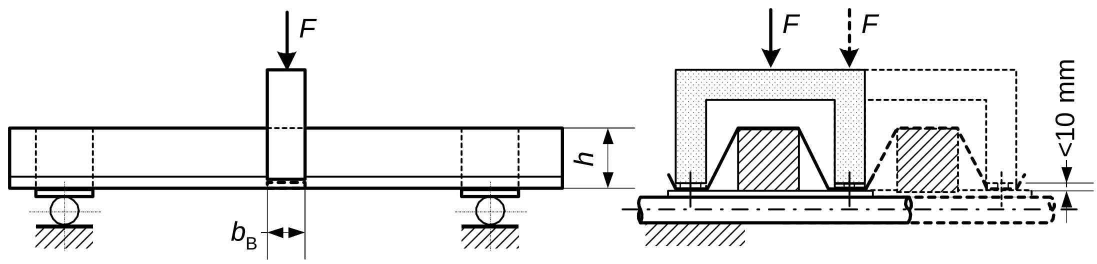
NOTE: Value of maximum support reaction measured during a bending test may be used as a lower bound for section resistance to both shear and local transverse force.



a) Internal support under gravity loading

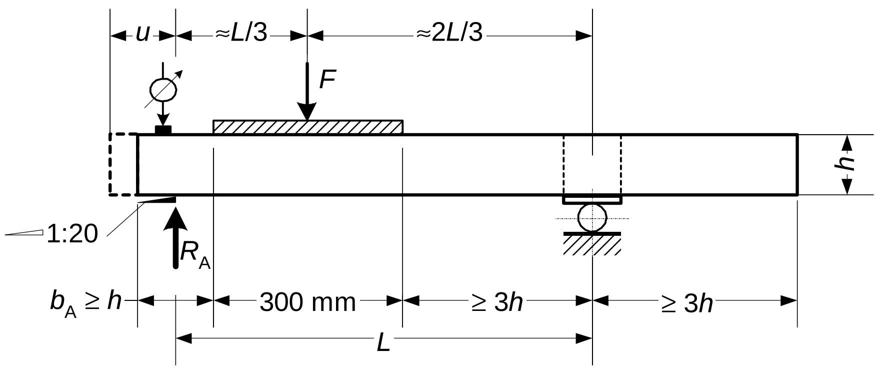


b) Internal support under uplift loading



c) Internal support with loading applied to tension flange

Figure A.5: Test set-up for internal support test



Key:

b_A = support length

u = length from internal edge of end support to end of sheet

Figure A.6: Test set-up for end support tests

A.3 Tests on cold-formed members

A.3.1 General

(1) Each test specimen should be similar in all respects to the component or structure that it represents.

(2) The supporting devices used for tests should preferably provide end conditions that closely reproduce those supplied by the connections to be used in service. Where this cannot be achieved, less favourable end conditions that decrease the load carrying capacity or increase the flexibility should be used, as relevant.

(3) The devices used to apply the test loads should reproduce the way that the loads would be applied in service. It should be ensured that they do not offer more resistance to transverse deformations of the cross-section than would be available in the event of an overload in service. It should also be ensured that they do not localize the applied forces onto the lines of greatest resistance.

(4) If the given load combination includes forces on more than one line of action, each increment of the test loading should be applied proportionately to each of these forces.

(5) At each stage of the loading, the displacements or strains should be measured at one or more principal locations on the structure. Readings of displacements or strains should not be taken until the structure has completely stabilized after a load increment.

(6) Failure of a test specimen should be considered to have occurred in any of the following cases:

- at collapse or fracture;
- if a crack begins to spread in a vital part of the specimen;
- if the displacement is excessive.

(7) The test result should be taken as the maximum value of the loading applied to the specimen either coincident with failure or immediately prior to failure as appropriate.

(8) The accuracy of all measurements should be compatible with the magnitude of the measurement concerned and should in any case not exceed $\pm 1\%$ of the value to be determined. The following magnitudes (in clause (9)) must also be fulfilled.

(9) The measurements of the cross-sectional geometry of the test specimen should include:

- the overall dimensions (width, depth and length) to an accuracy of $\pm 1{,}0$ mm;
- widths of plane elements of the cross-section to an accuracy of $\pm 1{,}0$ mm;
- radii of bends to an accuracy of $\pm 1{,}0$ mm;
- inclinations of plane elements to an accuracy of $\pm 2{,}0°$;
- angles between flat surfaces to an accuracy of $\pm 2{,}0°$;
- locations and dimensions of intermediate stiffeners to an accuracy of $\pm 1{,}0$ mm;
- the thickness of the material to an accuracy of $\pm 0{,}01$ mm;
- accuracy of all measurements of the cross-section has to be taken as equal to maximum $0{,}5\ \%$ of the nominal values.

(10)All other relevant parameters should also be measured, such as:

- locations of components relative to each other;
- locations of fasteners;
- the values of torques etc. used to tighten fasteners.

A.3.2 Full cross-section compression tests

A.3.2.1 Stub column test

(1) Stub column tests may be used to allow for the effects of local buckling in thin gauge cross-sections, by determining the value of the ratio $\beta_A = A_{eff} / A_g$ and the location of the effective centroidal axis.

(2) If local buckling of the plane elements governs the resistance of the cross-section, the specimen should have a length of at least 3 times the width of the widest plate element.

(3) The lengths of specimens with perforated cross-sections should include at least 5 pitches of the perforations, and should be such that the specimen is cut to length midway between two perforations.

(4) In the case of a cross-section with edge or intermediate stiffeners, it should be ensured that the length of the specimen is not less than the expected buckling lengths of the stiffeners.

(5) If the overall length of the specimen exceeds 20 times the least radius of gyration of its gross cross-section i_{min}, intermediate lateral restraints should be supplied at a spacing of not more than $20\, i_{min}$.

(6) Before testing, the tolerances of the cross-sectional dimensions of the specimen should be checked to ensure that they are within the permitted deviations.

(7) The cut ends of the specimen should be flat, and should be perpendicular to its longitudinal axis.

(8) An axial compressive force should be applied to each end of the specimen through pressure pads at least 30 mm thick, that protrude at least 10 mm beyond the perimeter of the cross-section.

(9) The test specimen should be placed in the testing machine with a ball bearing at each end. There should be small drilled indentations in the pressure pads to receive the ball bearings. The ball bearings should be located in line with the centroid of the calculated effective cross-section. If the calculated location of this effective centroid proves not to be correct, it may be adjusted within the test series.

(10)In the case of open cross-sections, possible spring-back may be corrected.

(11)Stub column tests may be used to determine the compression resistance of a cross-section. In interpreting the test results, the following parameters should be treated as variables:

- the thickness;
- the ratio b_p / t;
- the ratio f_u / f_{yb};
- the ultimate strength f_u and the yield strength f_{yb};
- the location of the centroid of the effective cross-section;
- imperfections in the shape of the elements of the cross-section;
- the method of cold forming (for example increasing the yield strength by introducing a deformation that is subsequently removed).

A.3.2.2 Member buckling test

(1) Member buckling tests may be used to determine the resistance of compression members with thin gauge cross-sections to overall buckling (including flexural buckling, torsional buckling and torsional-flexural buckling) and the interaction between local buckling and overall buckling.

(2) The method of carrying out the test should be generally as given for stub column tests in A.3.2.1.

(3) A series of tests on axially loaded specimens may be used to determine the appropriate buckling curve for a given type of cross-section and a given grade of steel, produced by a specific process. The values of relative slenderness $\overline{\lambda}$ to be tested and the minimum number of tests n at each value, should be as given in table A.1.

Table A.1: Relative slenderness values and numbers of tests

$\bar{\lambda}$	0,2	0,5	0,7	1,0	1,3	1,6	2,0	3,0
N	3	5	5	5	5	5	5	5

(4) Similar tests may also be used to determine the effect of introducing intermediate restraints on the torsional buckling resistance of a member.

(5) For the interpretation of the test results the following parameters should be taken into account:

- the parameters listed for stub column tests in A.3.2.1(11);

- overall lack of straightness imperfections compared to standard production output, see (6);

- type of end or intermediate restraint (flexural, torsional or both).

(6) Overall lack of straighness may be taken into account as follows:

a) Determine the critical compression load of the member by an appropriate analysis with initial bow equal to test sample: $F_{cr,bow,test}$

b) As a) but with an initial bow equal to the maximum allowed according to the product specification: $F_{cr,bow,max,nom}$

c) Additional correction factor: $F_{cr,bow,max,nom}$ / $F_{cr,bow,test}$

A.3.3 Full cross-section tension test

(1) This test may be used to determine the average yield strength f_{ya} of the cross-section.

(2) The specimen should have a length of at least 5 times the width of the widest plane element in the cross-section.

(3) The load should be applied through end supports that ensure a uniform stress distribution across the cross-section.

(4) The failure zone should occur at a distance from the end supports of not less than the width of the widest plane element in the cross-section.

A.3.4 Full cross-section bending test

(1) This test may be used to determine the moment resistance and rotation capacity of a cross-section.

(2) The specimen should have a length of at least 15 times its greatest transversal dimension. The spacing of lateral restraints to the compression flange should not be less than the spacing to be used in service.

(3) A pair of point loads should be applied to the specimen to produce a length under uniform bending moment at midspan of at least 0,2 $\times$ (span) but not more than 0,33 $\times$ (span). These loads should be applied through the shear centre of the cross-section. The section should be torsionally restrained at the load points. If necessary, local buckling of the specimen should be prevented at the points of load application, to ensure that failure occurs within the central portion of the span. The deflection should be measured at the load positions, at midspan and at the ends of the specimen.

(4) In interpreting the test results, the following parameters should be treated as variables:

- the thickness;

- the ratio b_p / t;

- the ratio f_u / f_{yb};

- the ultimate strength f_u and the yield strength f_{yb};
- differences between restraints used in the test and those available in service;
- the support conditions.

A.4 Tests on structures and portions of structures

A.4.1 Acceptance test

(1) This acceptance test may be used as a non-destructive test to confirm the structural performance of a structure or portion of a structure.

(2) The test load for an acceptance test should be taken as equal to the sum of:

- 1,0 × (the actual self-weight present during the test);
- 1,15 × (the remainder of the permanent load);
- 1,25 × (the variable loads).

but need not be taken as more than the mean of the total ultimate limit state design load and the total serviceability limit state design load for the characteristic (rare) load combination.

(3) Before carrying out the acceptance test, preliminary bedding down loading (not exceeding the characteristic values of the loads) may optionally be applied, and then removed.

(4) The structure should first be loaded up to a load equal to the total characteristic load. Under this load it should demonstrate substantially elastic behaviour. On removal of this load the residual deflection should not exceed 20% of the maximum recorded. If these criteria are not satisfied this part of the test procedure should be repeat. In this repeat load cycle, the structure should demonstrate substantially linear behaviour up to the characteristic load and the residual deflection should not exceed 10% of the maximum recorded.

(5) During the acceptance test, the loads should be applied in a number of regular increments at regular time intervals and the principal deflections should be measured at each stage. When the deflections show significant non-linearity, the load increments should be reduced.

(6) On the attainment of the acceptance test load, the load should be maintained for being no changes between a set of adjacent readings and deflection measurements should be taken to establish whether the structure is subject to any time-dependent deformations, such as deformations of fasteners or deformations arising from creep in the zinc layer.

(7) Unloading should be completed in regular decrements, with deflection readings taken at each stage.

(8) The structure should prove capable of sustaining the acceptance test load, and there should be no significant local distortion or defects likely to render the structure unserviceable after the test.

A.4.2 Strength test

(1) This strength test may be used to confirm the calculated load carrying capacity of a structure or portion of a structure. Where a number of similar items are to be constructed to a common design, and one or more prototypes have been submitted to and met all the requirements of this strength test, the others may be accepted without further testing provided that they are similar in all relevant respects to the prototypes.

(2) Before carrying out a strength test the specimen should first pass the acceptance test detailed in A.4.1.

(3) The load should then be increased in increments up to the strength test load and the principal deflections should be measured at each stage. The strength test load should be maintained for at least one hour and deflection measurements should be taken to establish whether the structure is subject to creep.

(4) Unloading should be completed in regular decrements with deflection readings taken at each stage.

(5) The total test load (including self-weight) for a strength test F_{str} should be determined from the total design load F_{Ed} specified for ultimate limit state verifications by calculation, using:

$$F_{str} = \gamma_M \mu_F F_{Ed} \quad \text{... (A.2)}$$

in which μ_F is the load adjustment coefficient and γ_M is the partial coefficient of the ultimate limit state.

(6) The load adjustment coefficient μ_F should take account of variations in the load carrying capacity of the structure, or portion of a structure, due to the effects of variation in the material yield strength, local buckling, overall buckling and any other relevant parameters or considerations.

(7) Where a realistic assessment of the load carrying capacity of the structure, or portion of a structure, may be made using the provisions of this Part 1-3 of EN 1993 for design by calculation, or another proven method of analysis that takes account of all buckling effects, the load adjustment coefficient μ_F may be taken as equal to the ratio of (the value of the assessed load carrying capacity based on the averaged basic yield strength f_{ym}) compared to (the corresponding value based on the nominal basic yield strength f_{yb}).

(8) The value of f_{ym} should be determined from the measured basic strength $f_{yb,obs}$ of the various components of the structure, or portion of a structure, with due regard to their relative importance.

(9) If realistic theoretical assessments of the load carrying capacity cannot be made, the load adjustment coefficient μ_F should be taken as equal to the resistance adjustment coefficient μ_R defined in A.6.2.

(10)Under the test load there should be no failure by buckling or rupture in any part of the specimen.

(11)On removal of the test load, the deflection should be reduced by at least 20%.

A.4.3 Prototype failure test

(1) A test to failure may be used to determine the real mode of failure and the true load carrying capacity of a structure or assembly. If the prototype is not required for use, it may optionally be used to obtain this additional information after completing the strength test described in A.4.2.

(2) Alternatively a test to failure may be carried out to determine the true design load carrying capacity from the ultimate test load. As the acceptance and strength test procedures should preferably be carried out first, an estimate should be made of the anticipated design load carrying capacity as a basis for such tests.

(3) Before carrying out a test to failure, the specimen should first pass the strength test described in A.4.2. Its estimated design load carrying capacity may then be adjusted based on its behaviour in the strength test.

(4) During a test to failure, the loading should first be applied in increments up to the strength test load. Subsequent load increments should then be based on an examination of the plot of the principal deflections.

(5) The ultimate load carrying capacity should be taken as the value of the test load at that point at which the structure or assembly is unable to sustain any further increase in load.

NOTE: At this point gross permanent distortion is likely to have occurred. In some cases gross deformation might define the test limit.

A.4.4 Calibration test

(1) A calibration test may be used to:

- verify load bearing behaviour relative to analytical design models;
- quantify parameters derived from design models, such as strength or stiffness of members or joints.

A.5 Tests on torsionally restrained beams

A.5.1 General

(1) These test procedures may be used for beams that are partially restrained against torsional displacement, by means of trapezoidal profiled steel sheeting or other suitable cladding.

(2) These procedures may be used for purlins, side rails, floor beams and other similar types of beams that have relevant restraint conditions.

A.5.2 Internal support test

A.5.2.1 Test set-up

(1) The test set-up shown in figure A.7 may be used to determine the resistance of a beam that is continuous over two or more spans, to combinations of bending moment and shear force at internal supports.

NOTE: The same test set-up may be used for sleeved and overlap systems.

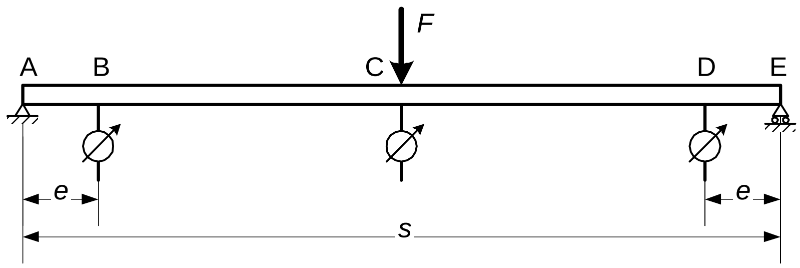


Figure A.7: Test set-up for end support tests

(2) The supports at **A** and **E** should be hinged and roller supports respectively. At these supports, rotation about the longitudinal axis of the beam may be prevented, for example by means of cleats.

(3) The method of applying the load at **C** should correspond with the method to be used in service.

NOTE: In many cases this will mean that lateral displacement of both flanges is prevented at **C**.

(4) The displacement measurements at points **B** and **D** located at a distance e from each support, see figure A.7, should be recorded to allow these displacements to be eliminated from the results analysis

(5) The test span s should be chosen to produce combinations of bending moment and shear force that represent those expected to occur in practical application under the design load for the relevant limit state.

(6) For double span beams of span L subject to uniformly distributed loads, the test span s should normally be taken as equal to $0,4L$. However, if plastic redistribution of the support moment is expected, the test span s should be reduced to represent the appropriate ratio of support moment to shear force.

A.5.2.2 Execution of tests

(1) In addition to the general rules for testing, the following specific aspects should be taken into account.

(2) Testing should continue beyond the peak load and the recording of the deflections should be continued either until the applied load has reduced to between 10% and 15% of its peak value or until the deflection has reached a value 6 times the maximum elastic displacement.

A.5.2.3 Interpretation of test results

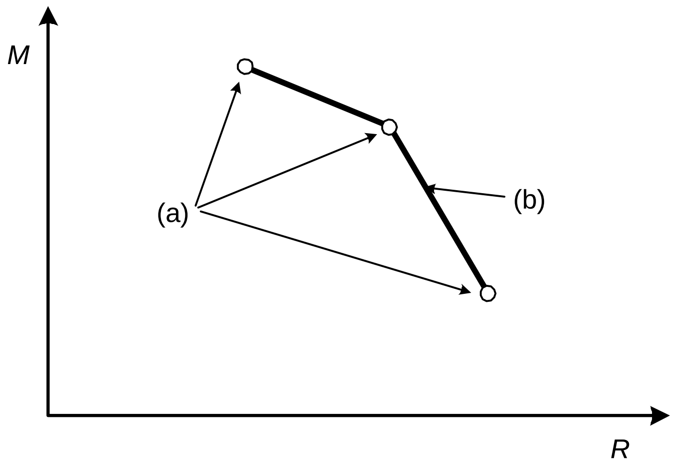
(1) The actual measured test results $R_{obs,i}$ should be adjusted as specified in A.6.2 to obtain adjusted values $R_{adj,i}$ related to the nominal basic yield strength f_{yb} and design thickness t of the steel, see 3.2.4.

(2) For each value of the test span s the support reaction R should be taken as the mean of the adjusted values of the peak load F_{max} for that value of s. The corresponding value of the support moment M should then be determined from:

$$M = \frac{s\ R}{4} \qquad \text{... (A.3)}$$

Generally the influence of the dead load should be added when calculating the value of moment M following the expression (A.3).

(3) The pairs of values of M and R for each value of s should be plotted as shown in figure A.8. Pairs of values for intermediate combinations of M and R may then be determined by linear interpolation.



(a) test results for different test spans *s*, (b) linear interpolation

Figure A.8: Relation between support moment M and support reaction R

(4) The net deflection at the point of load application **C** in figure A.7 should be obtained from the gross measured values by deducting the mean of the corresponding deflections measured at the points **B** and **D** located at a distance e from the support points **A** and **E**, see figure A.7.

(5) For each test the applied load should be plotted against the corresponding net deflection, see figure A.9. From this plot, the rotation θ should be obtained for a range of values of the applied load using:

$$\theta = \frac{2(\delta_{pl} - \delta_e - \delta_{el})}{0{,}5\ s\ -\ e} \qquad \text{... (A.4a)}$$

$$\theta = \frac{2(\delta_{pl} - \delta_e - \delta_{lin})}{0{,}5\ s\ -\ e} \qquad \text{... (A.4b)}$$

where:

δ_{el} is the net deflection for a given load on the rising part of the curve, before F_{max};

δ_{pl} is the net deflection for the same load on the falling part of the curve, after F_{max};

δ_{lin} is the fictive net deflection for a given load, that would be obtained with a linear behaviour, see figure A.9;

δ_e is the average deflection measured at a distance e from the support, see figure A.7;

s is the test span;

e is the distance between a deflection measurement point and a support, see figure A.7.

The expression (A.4a) is used when analyses are done based on the effective cross-section. The expression (A.4b) is used when analyses are done based on the gross cross-section.

(6) The relationship between M and θ should then be plotted for each test at a given test span s corresponding to a given value of beam span L as shown in figure A.10. The design M - θ characteristic for the moment resistance of the beam over an internal support should then be taken as equal to 0,9 times the mean value of M for all the tests corresponding to that value of the beam span L.

NOTE: Smaller value than 0,9 for reduction should be used, if the full-scale tests are used to determine effect of lateral load and buckling of free flange around the mid-support, see 10.1.3.2(4).

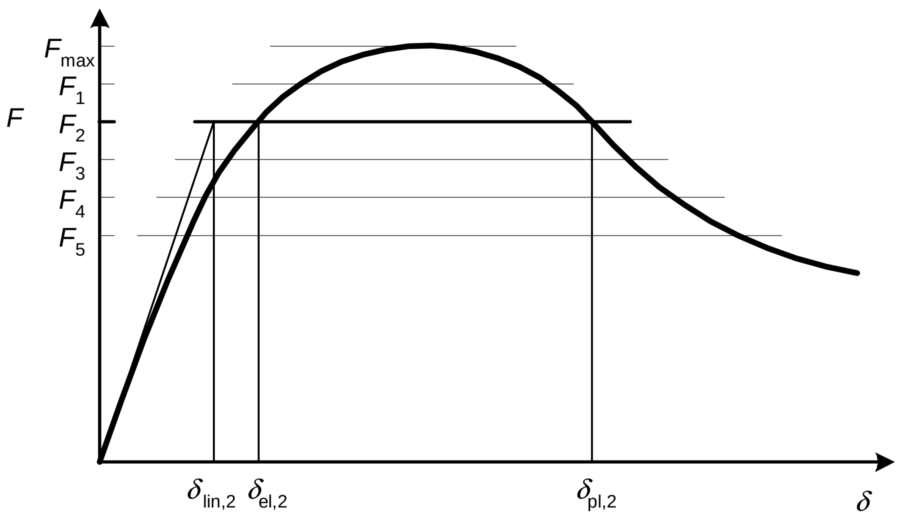
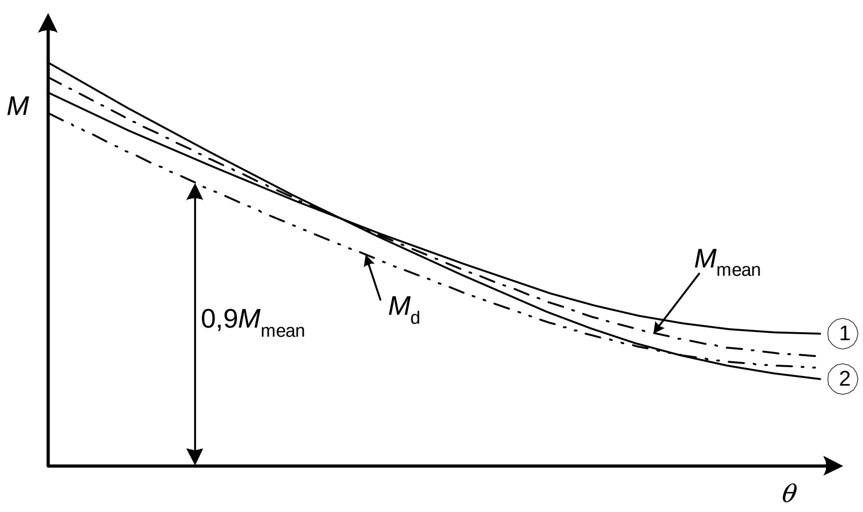


Figure A.9: Relation between load F and net deflection δ



M_{mean} = mean value, M_d = design value

Figure A.10: Derivation of moment-rotation (M - θ) characteristic

A.5.3 Determination of torsional restraint

(1) The test set-up shown in figure A.11 may be used to determine the amount of torsional restraint given by

adequately fastened sheeting or by another member perpendicular to the span of the beam.

(2) This test set-up covers two different contributions to the total amount of restraint as follows:

a) The lateral stiffness K_A per unit length corresponding to the rotational stiffness of the connection between the sheeting and the beam;

b) The lateral stiffness K_B per unit length due to distortion of the cross-section of the purlin.

(3) The combined restraint per unit length may be determined from:

$$(1 / K_A + 1 / K_B) = \delta / F \quad \text{... (A.5)}$$

where:

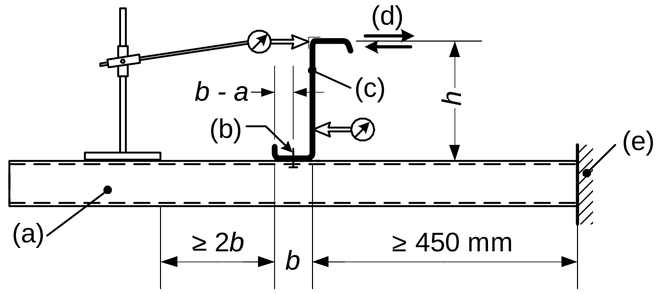
F is the load per unit length of the test specimen to produce a lateral deflection of $h/10$;

h is the overall depth of the specimen;

δ is the lateral displacement of the top flange in the direction of the load F.

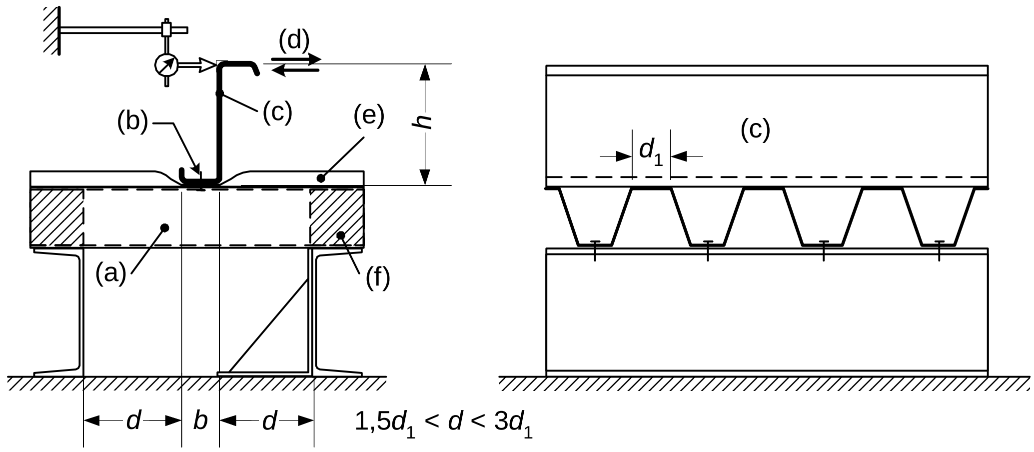
(4) In interpreting the test results, the following parameters should be treated as variables:

- the number of fasteners per unit length of the specimen;
- the type of fasteners;
- the flexural stiffness of the beam, relative to its thickness;
- the flexural stiffness of the bottom flange of the sheeting, relative to its thickness;
- the positions of the fasteners in the flange of the sheeting;
- the distance from the fasteners to the centre of rotation of the beam;
- the overall depth of the beam;
- the possible presence of insulation between the beam and the sheeting.



(a) sheeting, (b) fastener, (c) profile, (d) load, (e) clamped support

a) Alternative 1



(a) sheeting, (b) fastener, (c) profile, (d) load, (e) insulation if available, (f) timber blocks

b) Alternative 2

Figure A.11: Experimental determination of spring stiffness K_A and K_B

A.6 Evaluation of test results

A.6.1 General

(1) A specimen under test should be regarded as having failed if the applied test loads reach their maximum values, or if the gross deformations exceed specified limits.

(2) The gross deformations of members should generally satisfy:

$$\delta \leq L/50 \qquad \text{... (A.6)}$$

$$\phi \leq 1/50 \qquad \text{... (A.7)}$$

where:

δ is the maximum deflection of a beam of span L;

ϕ is the sway angle of a structure.

(3) In the testing of connections, or of components in which the examination of large deformations is necessary for accurate assessment (for example, in evaluating the moment-rotation characteristics of sleeves), no limit need be placed on the gross deformation during the test.

(4) An appropriate margin of safety should be available between a ductile failure mode and possible brittle failure modes. As brittle failure modes do not usually appear in large scale tests, additional detail tests should

be carried out where necessary.

NOTE: This is often the case for connections.

A.6.2 Adjustment of test results

(1) Test results should be appropriately adjusted to allow for variations between the actual measured properties of the test specimens and their nominal values.

(2) The actual measured basic yield strength $f_{yb,obs}$ should not deviate by more than -25% from the nominal basic yield strength f_{yb} i.e. $f_{yb,obs} \geq 0{,}75\, f_{yb}$.

(3) The actual measured thickness t_{obs} should not exceed the nominal material thickness t_{nom} (see 3.2.4) by more than 12%.

(4) Adjustments should be made in respect of the actual measured values of the core material thickness $t_{obs,cor}$ and the basic yield strength $f_{yb,obs}$ for all tests, except if values measured in tests are used to calibrate a design model then provisions of (5) need not be applied.

(5) The adjusted value $R_{adj,i}$ of the test result for test i should be determined from the actual measured test result $R_{obs,i}$ using:

$$R_{adj,i} = R_{obs,i} / \mu_R \qquad \text{... (A.8)}$$

in which μ_R is the resistance adjustment coefficient given by:

$$\mu_R = \left(\frac{f_{yb,obs}}{f_{yb}}\right)^{\alpha} \left(\frac{t_{obs,cor}}{t_{cor}}\right)^{\beta} \qquad \text{... (A.9)}$$

(6) The exponent α for use in expression (A.9) should be obtained as follows:

- if $f_{yb,obs} \leq f_{yb}$: $\alpha = 0$
- if $f_{yb,obs} > f_{yb}$:
  - generally: $\alpha = 1$

For profiled sheets or liner trays in which compression elements have such large b_p / t ratios that local buckling is clearly the failure mode: $\alpha = 0{,}5$.

(7) The exponent β for use in expression (A.9) should be obtained as follows:

- if $t_{obs,cor} \leq t_{cor}$: $\beta = 1$
- if $t_{obs,cor} > t_{cor}$:
  - for tests on profiled sheets or liner trays: $\beta = 2$
- for tests on members, structures or portions of structures:
  - if $b_p / t \leq (b_p / t)_{lim}$: $\beta = 1$
  - if $b_p / t > 1{,}5(b_p / t)_{lim}$: $\beta = 2$
  - if $(b_p / t)_{lim} < b_p / t < 1{,}5(b_p / t)_{lim}$: obtain β by linear interpolation.

in which the limiting width-to thickness ratio $(b_p / t)_{lim}$ given by:

$$(b_p / t)_{lim} = 0{,}64 \sqrt{\frac{E\, k_\sigma}{f_{yb}}} \cdot \sqrt{\frac{f_{yb} / \gamma_{M1}}{\sigma_{com,Ed}}} \cong 19{,}1\,\varepsilon \sqrt{k_\sigma} \cdot \sqrt{\frac{f_{yb} / \gamma_{M1}}{\sigma_{com,Ed}}} \qquad \text{... (A.10)}$$

where:

b_p is the notional flat width of a plane element;

k_σ is the relevant buckling factor from table 5.3 or 5.4;

$\sigma_{com,Ed}$ is the largest calculated compressive stress in that element, when the resistance of the cross-section is reached.

NOTE: In the case of available test report concerning sheet specimens with $t_{obs,cor} / t_{cor} \leq 1{,}06$ readjustment of existing value not exceeding 1,02 times the $R_{adj,i}$ value according to A.6.2 may be ommitted.

A.6.3 Characteristic values

A.6.3.1 General

(1) Characteristic values may be determined statistically, provided that there are at least 4 test results.

NOTE: A larger number is generally preferable, particularly if the scatter is relatively wide.

(2) If the number of test results available is 3 or less, the method given in A.6.3.3 may be used.

(3) The characteristic minimum value should be determined using the following provisions. If the characteristic maximum value or the characteristic mean value is required, it should be determined by using appropriate adaptations of the provisions given for the characteristic minimum value.

(4) The characteristic value R_k determined on the basis of at least 4 tests may be obtained from:

$$R_k = R_m \pm k s \quad \text{... (A.11)}$$

where:

s is the standard deviation;

k is the appropriate coefficient from table A.2;

R_m is the mean value of the adjusted test results R_{adj};

The unfavourable sign "+" or "-" ~~shall~~ should be adopted for given considered value.

NOTE: As general rule, for resistance characteristic value, the sign "-" should be taken and e.g. for rotation characteristic value, both are to be considered.

(5) The standard deviation s may be determined using:

$$s = \left[\sum_{i=1}^{n} \left(R_{adj.i} - R_m \right)^2 / (n-1) \right]^{0,5} \equiv \left[\left[\sum_{i=1}^{n} \left(R_{adj.i} \right)^2 - (1/n) \left(\sum_{i=1}^{n} R_{adj.i} \right)^2 \right] \Big/ (n-1) \right]^{0,5} \quad \text{... (A.12)}$$

where:

$R_{adj,i}$ is the adjusted test result for test i;

n is the number of tests.

Table A.2: Values of the coefficient *k*

N	4	5	6	8	10	20	30	∞
k	2,63	2,33	2,18	2,00	1,92	1,76	1,73	1,64

A.6.3.2 Characteristic values for families of tests

(1) A series of tests carried out on a number of otherwise similar structures, portions of structures, members, sheets or other structural components, in which one or more parameters is varied, may be treated as a single

family of tests, provided that they all have the same failure mode. The parameters that are varied may include cross-sectional dimensions, spans, thicknesses and material strengths.

(2) The characteristic resistances of the members of a family may be determined on the basis of a suitable design expression that relates the test results to all the relevant parameters. This design expression may either be based on the appropriate equations of structural mechanics, or determined on an empirical basis.

(3) The design expression should be modified to predict the mean measured resistance as accurately as practicable, by adjusting the coefficients to optimize the correlation.

NOTE: Information on this process is given Annex D of EN 1990.

(4) In order to calculate the standard deviation s each test result should first be normalized by dividing it by the corresponding value predicted by the design expression. If the design expression has been modified as specified in (3), the mean value of the normalized test results will be unity. The number of tests n should be taken as equal to the total number of tests in the family.

(5) For a family of at least four tests, the characteristic resistance R_k should then be obtained from expression (A.11) by taking R_m as equal to the value predicted by the design expression, and using the value of k from table A.2 corresponding to a value of n equal to the total number of tests in the family.

A.6.3.3 Characteristic values based on a small number of tests

(1) If only one test is carried out, then the characteristic resistance R_k corresponding to this test should be obtained from the adjusted test result R_{adj} using:

$$R_k = 0,9\, \eta_k R_{adj} \qquad \text{... (A.13)}$$

in which η_k should be taken as follows, depending on the failure mode:

- yielding failure: $\eta_k = 0,9$;
- gross deformation: $\eta_k = 0,9$;
- local buckling: $\eta_k = 0,8 \text{ ... } 0,9$ depending on effects on global behaviour in tests ;
- overall instability: $\eta_k = 0,7$.

(2) For a family of two or three tests, provided that each adjusted test result $R_{adj,i}$ is within $\pm 10\%$ of the mean value R_m of the adjusted test results, the characteristic resistance R_k should be obtained using:

$$R_k = \eta_k R_m \qquad \text{... (A.14)}$$

(3) The characteristic values of stiffness properties (such as flexural or rotational stiffness) may be taken as the mean value of at least two tests, provided that each test result is within $\pm 10\%$ of the mean value.

(4) In the case of one single test the characteristic value of the stiffness is reduced by 0,95 for favourable value and increased by 1,05 for non-favourable value.

A.6.4 Design values

(1) The design value of a resistance R_d should be derived from the corresponding characteristic value R_k determined by testing, using:

$$R_d = \eta_{sys} \frac{R_k}{\gamma_M} \qquad \text{... (A.15)}$$

where:

γ_M is the partial factor for resistance;

η_{sys} is a conversion factor for differences in behaviour under test conditions and service conditions.

(2) The appropriate value for η_{sys} should be determined in dependance of the modelling for testing.

(3) For sheeting and for other well defined standard testing procedures (including A.3.2.1 stub column tests,

A.3.3 tension tests and A.3.4 bending tests) η_{sys} may be taken as equal to 1,0. For tests on torsionally restrained beams conformed to the section A.5, η_{sys} = 1,0 may also be taken.

(4) For other types of tests in which possible instability phenomena, or modes of behaviour, of structures or structural components might not be covered sufficiently by the tests, the value of η_{sys} should be assessed taking into account the actual testing conditions, in order to achieve the necessary reliability.

NOTE: The partial factor γ_M may be given in the National Annex. It is recommended to use the γ_M-values as chosen in the design by calculation given in section 2 or section 8 of this part unless other values result from the use of Annex D of EN 1990.

A.6.5 Serviceability

(1) The provisions given in Section 7 should be satisfied.

Annex B [informative] – Durability of fasteners

~~B.1 Durability of fasteners~~

(1) In Construction Classes I, II and III table B.1 may be applied.

Table B.1: Fastener material with regard to corrosion environment (and sheeting material only for information). Only the risk of corrosion is considered. Classification of environment according to EN ISO 12944-2.

Classification of environment	Sheet material	Material of fastener					
		Aluminium	Electro galvanized steel. Coat thickness > 7μm	Hot-dip zinc coated steel[b]. Coat thickness >45μm	Stainless steel, case hardened. 1.4006 [d]	Stainless steel, 1.4301 [d] 1.4436 [d]	Monel[a]
C1	A, B, C	X	X	X	X	X	X
	D, E, S	X	X	X	X	X	X
C2	A	X	-	X	X	X	X
	C, D, E	X	-	X	X	X	X
	S	X	-	X	X	X	X
C3	A	X	-	X	-	X	X
	C, E	X	-	X	(X)[C]	(X)[C]	-
	D	X	-	X	-	(X)[C]	X
	S	-	-	X	X	X	X
C4	A	X	-	(X)[C]	-	(X)[C]	-
	D	-	-	X	-	(X)[C]	-
	E	X	-	X	-	(X)[C]	-
	S	-	-	X	-	X	X
C5-I	A	X	-	-	-	(X)[C]	-
	D[f]	-	-	X	-	(X)[C]	-
	S	-	-	-	-	X	-
C5-M	A	X	-	-	-	(X)[C]	-
	D[f]	-	-	X	-	(X)[C]	-
	S	-	-	-	-	X	-

Anm. Fastener of steel without coating may be used in corrosion classification class C1.

A = Aluminium irrespective of surface finish

B = Un-coated steel sheet

C = Hot-dip zinc coated (Z275) or aluzink coated (AZ150) steel sheet

D = Hot-dip zinc coated steel sheet + coating of paint or plastics

E = Aluzink coated (AZ185) steel sheet

S = Stainless steel

X = Type of material recommended from the corrosion standpoint

(X) = Type of material recommended from the corrosion standpoint under the specified condition only

\- = Type of material not recommended from the corrosion standpoint

a Refers to rivets only

b Refers to screws and nuts only

c Insulating washer, of material resistant to ageing, between sheeting and fastener

d Stainless steel EN 10 088

e Risk of discoloration.

f Always check with sheet supplier

(2) The environmental classification following EN-ISO 12944-2 is presented in table B.2.

Table B.2: Atmospheric-corrosivity categories according to EN ISO 12944-2 and examples of typical environments

Corro-sivity category	Corro-sivity level	Examples of typical environments in a temperate climate (informative))	
		Exterior	Interior
C1	Very low	-	Heated buildings with clean atmospheres, e. g. offices, shops, schools and hotels.
C2	Low	Atmospheres with low level of pollution. Mostly rural areas	Unheated buildings where condensation may occur, e. g. depots, sport halls.
C3	Medium	Urban and industrial atmospheres, moderate sulphur dioxide pollution. Coastal areas with low salinity.	Production rooms with high humidity and some air pollution, e. g. food-processing plants, laundries, breweries and dairies.
C4	High	Industrial areas and coastal areas with moderate salinity.	Chemical plants, swimming pools, coastal ship- and boatyards.
C5-I	Very high (in-dustrial)	Industrial areas with high humidity and aggressive atmosphere.	Building or areas with almost permanent condensation and with high pollution.
C5-M	Very high (marine)	Coastal and offshore areas with high salinity.	Building or areas with almost permanent condensation and with high pollution.

Annex C [informative] – Cross section constants for thin-walled cross sections

~~**Drafting note:** Update of Annex C received from Prof. Höglund on 1 December 2003.~~

C.1 Open cross sections

(1) Divide the cross section into *n* parts. Number the parts 1 to *n*.

Insert nodes between the parts. Number the nodes 0 to *n*.

Part *i* is then defined by nodes *i* - 1 and *i*.

Give nodes, co-ordinates and (effective) thickness.

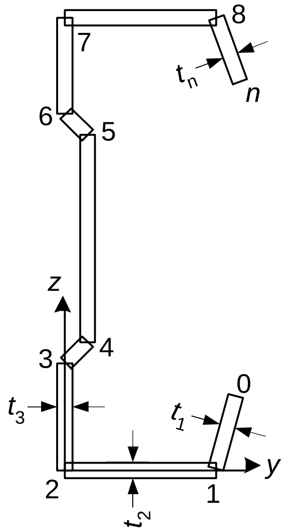


Figure C.1 Cross section nodes

Nodes and parts $\quad j = 0..n \quad i = 1..n$

Area of cross section parts

$$dA_i = \overrightarrow{\left[t_i \cdot \sqrt{(y_i - y_{i-1})^2 + (z_i - z_{i-1})^2} \right]}$$

Cross section area

$$A = \sum_{i=1}^{n} dA_i$$

First moment of area with respect to *y*-axis and coordinate for gravity centre

$$S_{y0} = \sum_{i=1}^{n} (z_i + z_{i-1}) \cdot \frac{dA_i}{2} \qquad z_{gc} = \frac{S_{y0}}{A}$$

Second moment of area with respect to original *y*-axis and new *y*-axis through gravity centre

$$I_{y0} = \sum_{i=1}^{n} \left[(z_i)^2 + (z_{i-1})^2 + z_i \cdot z_{i-1} \right] \cdot \frac{dA_i}{3} \qquad I_y = I_{y0} - A \cdot z_{gc}{}^2$$

First moment of area with respect to *z*-axis and gravity centre

$$S_{z0} = \sum_{i=1}^{n} (y_i + y_{i-1}) \cdot \frac{dA_i}{2} \qquad y_{gc} = \frac{S_{z0}}{A}$$

Second moment of area with respect to original *z*-axis and new *z*-axis through gravity centre

$$I_{z0} = \sum_{i=1}^{n} \left[(y_i)^2 + (y_{i-1})^2 + y_i \cdot y_{i-1} \right] \cdot \frac{dA_i}{3} \qquad I_z = I_{z0} - A \cdot y_{gc}{}^2$$

Product moment of area with respect of original *y*- and *z*-axis and new axes through gravity centre

$$I_{yz0} = \sum_{i=1}^{n} \left(2 \cdot y_{i-1} \cdot z_{i-1} + 2 \cdot y_i \cdot z_i + y_{i-1} \cdot z_i + y_i \cdot z_{i-1}\right) \cdot \frac{dA_i}{6} \quad I_{yz} = I_{yz0} - \frac{S_{y0} \cdot S_{z0}}{A}$$

Principal axis

$$\alpha = \frac{1}{2} \arctan\left(\frac{2I_{yz}}{I_z - I_y}\right) \text{ if } (I_z - I_y) \neq 0 \text{ otherwise } \alpha = 0$$

$$I_\xi = \frac{1}{2} \cdot \left[I_y + I_z + \sqrt{\left(I_z - I_y\right)^2 + 4 \cdot I_{yz}{}^2}\right]$$

$$I_\eta = \frac{1}{2} \cdot \left[I_y + I_z - \sqrt{\left(I_z - I_y\right)^2 + 4 \cdot I_{yz}{}^2}\right]$$

Sectorial co-ordinates

$$\omega_0 = 0 \qquad \omega_{0_i} = y_{i-1} \cdot z_i - y_i \cdot z_{i-1} \qquad \omega_i = \omega_{i-1} + \omega_{0_i}$$

Mean of sectorial coordinate

$$I_\omega = \sum_{i=1}^{n} \left(\omega_{i-1} + \omega_i\right) \cdot \frac{dA_i}{2} \qquad \omega_{mean} = \frac{I_\omega}{A}$$

Sectorial constants

$$I_{y\omega 0} = \sum_{i=1}^{n} \left(2 \cdot y_{i-1} \cdot \omega_{i-1} + 2 \cdot y_i \cdot \omega_i + y_{i-1} \cdot \omega_i + y_i \cdot \omega_{i-1}\right) \cdot \frac{dA_i}{6} \qquad I_{y\omega} = I_{y\omega 0} - \frac{S_{z0} \cdot I_\omega}{A}$$

$$I_{z\omega 0} = \sum_{i=1}^{n} \left(2 \cdot \omega_{i-1} \cdot z_{i-1} + 2 \cdot \omega_i \cdot z_i + \omega_{i-1} \cdot z_i + \omega_i \cdot z_{i-1}\right) \cdot \frac{dA_i}{6} \qquad I_{z\omega} = I_{z\omega 0} - \frac{S_{y0} \cdot I_\omega}{A}$$

$$I_{\omega\omega 0} = \sum_{i=1}^{n} \left[\left(\omega_i\right)^2 + \left(\omega_{i-1}\right)^2 + \omega_i \cdot \omega_{i-1}\right] \cdot \frac{dA_i}{3} \qquad I_{\omega\omega} = I_{\omega\omega 0} - \frac{I_\omega{}^2}{A}$$

Shear centre

$$y_{sc} = \frac{I_{z\omega} I_z - I_{y\omega} I_{yz}}{I_y \cdot I_z - I_{yz}{}^2} \qquad z_{sc} = \frac{-I_{y\omega} I_y + I_{z\omega} I_{yz}}{I_y \cdot I_z - I_{yz}{}^2} \qquad (I_y I_z - I_{yz}^2 \neq 0)$$

Warping constant

$$I_w = I_{\omega\omega} + z_{sc} \cdot I_{y\omega} - y_{sc} \cdot I_{z\omega}$$

Torsion constants

$$I_t = \sum_{i=1}^{n} dA_i \cdot \frac{\left(t_i\right)^2}{3} \qquad W_t = \frac{I_t}{min(t)}$$

Sectorial co-ordinate with respect to shear centre

$$\omega_{s_j} = \omega_j - \omega_{mean} + z_{sc} \cdot (y_j - y_{gc}) - y_{sc} \cdot (z_j - z_{gc})$$

Maximum sectorial co-ordinate and warping modulus

$$\omega_{max} = max(|\omega_s|) \qquad W_w = \frac{I_w}{\omega_{max}}$$

Distance between shear centre and gravity centre

$$y_s = y_{sc} - y_{gc} \qquad z_s = z_{sc} - z_{gc}$$

Polar moment of area with respect to shear centre

$$I_p = I_y + I_z + A(y_s^2 + z_s^2)$$

Non-symmetry factors z_j and y_j according to Annex F

$$z_j = z_s - \frac{0.5}{I_y} \cdot \sum_{i=1}^{n} \left[(z_{c_i})^3 + z_{c_i} \cdot \left[\frac{(z_i - z_{i-1})^2}{4} + (y_{c_i})^2 + \frac{(y_i - y_{i-1})^2}{12} \right] + y_{c_i} \cdot \frac{(y_i - y_{i-1}) \cdot (z_i - z_{i-1})}{6} \right] \cdot dA_i$$

$$y_j = y_s - \frac{0.5}{I_z} \cdot \sum_{i=1}^{n} \left[(y_{c_i})^3 + y_{c_i} \cdot \left[\frac{(y_i - y_{i-1})^2}{4} + (z_{c_i})^2 + \frac{(z_i - z_{i-1})^2}{12} \right] + z_{c_i} \cdot \frac{(z_i - z_{i-1}) \cdot (y_i - y_{i-1})}{6} \right] \cdot dA_i$$

where the coordinates for the centre of the cross section parts with respect to shear center are

$$y_{c_i} = \frac{y_i + y_{i-1}}{2} - y_{gc} \qquad z_{c_i} = \frac{z_i + z_{i-1}}{2} - z_{gc}$$

NOTE: $z_j = 0$ ($y_j = 0$) for cross sections with *y*-axis (*z*-axis) being axis of symmetry, see Figure C.1.

C.2 Cross section constants for open cross section with branches

(1) In cross sections with branches, formulae in C.1 can be used. However, follow the branching back (with thickness $t = 0$) to the next part with thickness $t \neq 0$, see branch 3 - 4 - 5 and 6 - 7 in Figure C.2. A section with branches is a section with points where more than two parts are joined together.

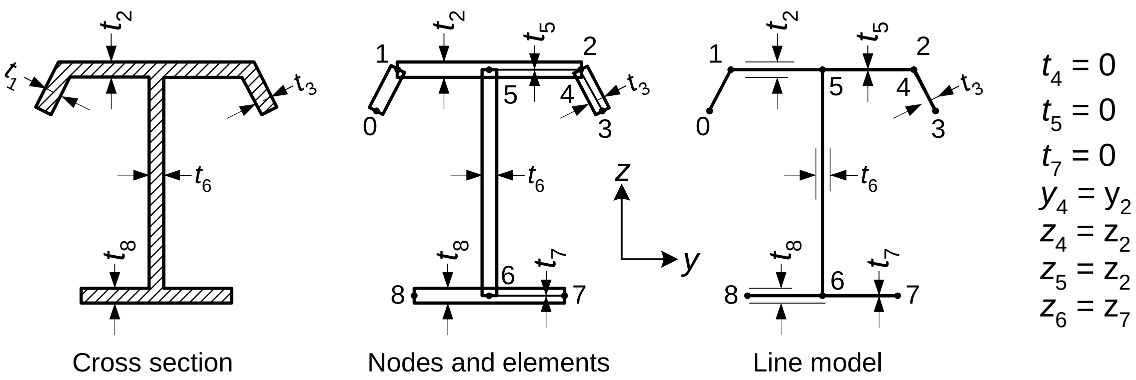


Figure C.2 Nodes and parts in a cross section with branches

C.3 Torsion constant and shear centre of cross section with closed part

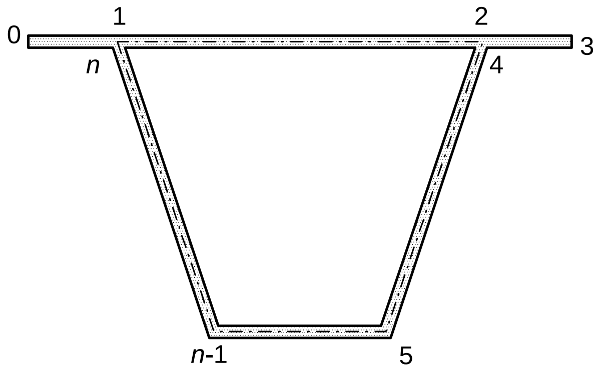


Figure C.3 Cross section with closed part

(1) For a symmetric or non-symmetric cross section with a closed part, Figure C.3, the torsion constant is given by

$$I_t = \frac{4A_t^2}{S_t} \text{ and } W_t = 2\,A_t \min(t_i)$$

where

$$A_t = 0{,}5 \sum_{i=2}^{n} (y_i - y_{i-1})(z_i + z_{i-1})$$

$$S_t = \sum_{i=2}^{n} \frac{\sqrt{(y_i - y_{i-1})^2 + (z_i - z_{i-1})^2}}{t_i} \qquad (t_i \neq 0)$$

Annex D [informative] – Mixed effective width/effective thickness method for outstand elements

~~**Drafting note:** Update of Annex D received from Prof. Höglund on 1 December 2003; to be checked with background material, see Prof. Höglund / Dr. Brune.~~

~~D.1 The method~~

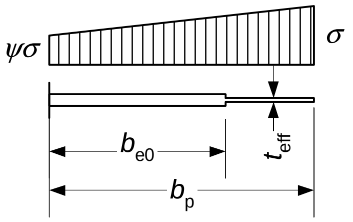
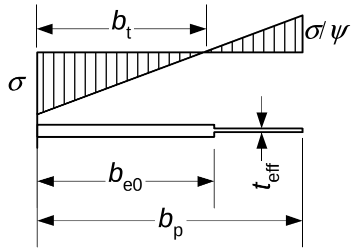
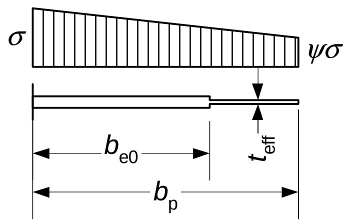
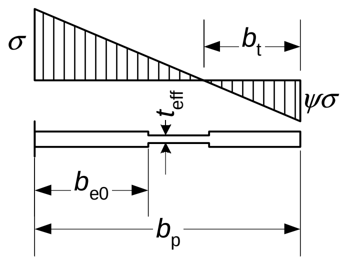
(1) This annex gives an alternative to the effective width method in 5.5.2 for outstand elements in compression. The effective area of the element is composed of the element thickness times an effective width b_{e0} and an effective thickness t_{eff} times the rest of the element width b_p. See Table C.1.

(2) The slenderness parameter $\bar{\lambda}_p$ and reduction factor ρ is found in 5.5.2 for the buckling factor k_σ in Table C.1.

(3) The stress relation factor ψ in the buckling factor k_σ may be based on the stress distribution for the gross cross section.

(4) The resistance of the section ~~shall~~ should be based on elastic stress distribution over the section.

Table D.1: Outstand compression elements

Maximum compression at free longitudinal edge		
Stress distribution	Effective width and thickness	Buckling factor
	$1 \geq \psi \geq 0$ $b_{e0} = 0{,}42 b_p$ $t_{eff} = (1{,}75\rho - 0{,}75)t$	$1 \geq \psi \geq -2$ $k_\sigma = \dfrac{1{,}7}{3+\psi}$
	$\psi < 0$ $b_{e0} = \dfrac{0{,}42 b_p}{(1-\psi)} + b_t < b_p$ $b_t = \dfrac{\psi b_p}{(\psi - 1)}$ $t_{eff} = (1{,}75\rho - 0{,}75 - 0{,}15\psi)t$	$-2 > \psi \geq -3$ $k_\sigma = 3{,}3(1+\psi) + 1{,}25\psi^2$ $\psi < -3$ $k_\sigma = 0{,}29(1-\psi)^2$
Maximum compression at supported longitudinal edge		
Stress distribution	Effective width and thickness	Buckling factor
	$1 \geq \psi \geq 0$ $b_{e0} = 0{,}42 b_p$ $t_{eff} = (1{,}75\rho - 0{,}75)t$	$1 \geq \psi \geq 0$ $k_\sigma = \dfrac{1{,}7}{1+3\psi}$
	$\psi < 0$ $b_{e0} = \dfrac{0{,}42 b_p}{(1-\psi)}$ $b_t = \dfrac{\psi b_p}{(\psi - 1)}$ $t_{eff} = (1{,}75\rho - 0{,}75)t$	$0 \geq \psi \geq -1$ $k_\sigma = 1{,}7 - 5\psi + 17{,}1\psi^2$ $\psi < -1$ $k_\sigma = 5{,}98(1-\psi)^2$

Annex E [Informative] – Simplified design for purlins

(1) Purlins with C-, Z- and Σ-cross-sections with or without additional stiffeners in web or flange may be designed due to (2) to (4) if the following conditions are fulfilled :

- the cross-section dimension are within the range of table 10.7;
- the purlins are horizontally restraint by trapezoidal sheeting where the horizontal restraint fulfill the conditions of the equation 10.1a~~E.1~~;
- the purlins are restraint rotationally by trapezoidal sheeting and the conditions of table E.1~~10.3~~ are met.
- the purlins have equal spans and uniform loading

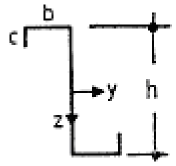
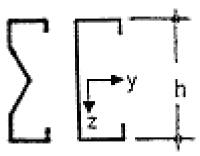
This method should not be used:

- for systems using anti-sag bars;
- for sleeve or overlapping systems;
- for application of axial forces N.

Note: The limitation and validity of this method may be given in the National Annex.

Table E.1: Limitations to be fulfilled if the simplified design method is used and other limits as in Table 5.1 and section 5.2

(the axis y and z are parallel respect rectangular to the top flange)

purlins	t [mm]	b/t	h/t	h/b	c/t	b/c	L/h
	$\geq 1{,}25$	≤ 55	≤ 160	$\leq 3{,}43$	≤ 20	$\leq 4{,}0$	≥ 15
	$\geq 1{,}25$	≤ 55	≤ 160	$\leq 3{,}43$	≤ 20	$\leq 4{,}0$	≥ 15

(2) The design value of the bending moment M_{Ed} should satisfy

$$\frac{M_{\mathrm{Ed}}}{M_{\mathrm{LT,Rd}}} \leq 1 \qquad \text{...(E.1)}$$

where

$$M_{\mathrm{LT,Rd}} = \left(\frac{f_{\mathrm{y}}}{\gamma_{\mathrm{M1}}}\right) W_{\mathrm{eff,y}} \frac{\chi_{\mathrm{LT}}}{k_{\mathrm{d}}} \qquad \text{...(E.2)}$$

and

$W_{\mathrm{eff,y}}$ is section modulus of the effective cross-section with regard to the axis y;

χ_{LT} is reduction factor for lateral torsional buckling in dependency of $\bar{\lambda}_{\mathrm{LT}}$ due to 6.2.3, where α_{LT} is substituted by $\alpha_{\mathrm{LT,eff}}$;

and

$$\bar{\lambda}_{\mathrm{LT}} = \sqrt{\frac{W_{\mathrm{eff,y}}\ f_{\mathrm{y}}}{M_{\mathrm{cr}}}} \qquad \text{…(E.3)}$$

$$\alpha_{\mathrm{LT,eff}} = \alpha_{\mathrm{LT}} \sqrt{\frac{W_{\mathrm{el,y}}}{W_{\mathrm{eff,y}}}} \qquad \text{…(E.4)}$$

and

α_{LT} is imperfection factor due to 6.2.3;

$W_{\mathrm{el,y}}$ is section modulus of the gross cross-section with regard to the axis y;

k_{d} is coefficient for consideration of the non restraint part of the purlin due to the equation (E.5) and table E.2;

$$k_{\mathrm{d}} = \left(a_1 - a_2\,\frac{L}{h}\right),\ \text{but} \ \geq 1.0 \qquad \text{…(E.5)}$$

a_1 , a_2 coefficients from table E.2;

L span of the purlin;

h overall depth of the purlin.

Table E.2: Coefficients a_1, a_2 for equation (E.5)

System	Z-purlins		C-purlins		Σ-purlins	
	a_1	a_2	a_1	a_2	a_1	a_2
single span beam gravity load	1.0	0	1.1	0.002	1.1	0.002
single span beam uplift load	1.3	0	3.5	0.050	1.9	0.020
continuous beam gravity load	1.0	0	1.6	0.020	1.6	0.020
continuous beam uplift load	1.4	0.010	2.7	0.040	1.0	0

(3) The reduction factor χ_{LT} may be chosen by equation (E.6), if a single span beam under gravity load is present or if equation (E.7) is met

$$\chi_{LT} = 1{,}0 \qquad \text{…(E.6)}$$

$$C_D \geq \frac{M_{el,u}^2}{E\,I_v} k_\vartheta \qquad \text{…(E.7)}$$

where

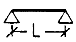
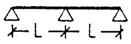
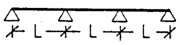
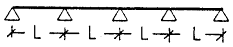
$M_{el,u} = W_{el,u}\, f_y$ elastic moment of the gross cross-section with regard to the major axis u; …(E.8)

I_v moment of inertia of the gross cross-section with regard to the minor axis v:

k_ϑ factor for considering the static system of the purlin due to table E.3.

NOTE: For equal flanged C-purlins and Σ-purlins $I_v = I_z$, $W_u = W_y$, and $M_{el,u} = M_{el,y}$. Conventions used for cross section axes are shown in Figure 1.7 and section 1.6.4.

Table E.3: Factors k_ϑ

Statical system	Gravity load	Uplift load
	-	0.210
	0.07	0.029
	0.15	0.066
	0.10	0.053

(4) The reduction factor χ_{LT} should be calculated by equation (6.36) using $\bar{\lambda}_{LT}$ and $\alpha_{LT,eff}$ in cases which are not met by (3). The elastic critical moment for lateral-torsional buckling M_{cr} may be calculated by the equation (E.9):

$$M_{cr} = \frac{k}{L}\sqrt{G\,I_t^*\,E\,I_v} \qquad \text{…(E.9)}$$

where

I_t^* is the fictitious St. Venant torsion constant considering the effective rotational restraint by equation (E.10) and (E.11):

$$I_t^* = I_t + C_D\,\frac{L^2}{\pi^2\,G} \qquad \text{…(E.10)}$$

I_t is St. Venant torsion constant of the purlin;

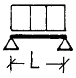
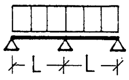
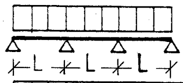
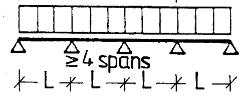
$$1/C_D = \frac{1}{C_{D,A}} + \frac{1}{C_{D,B}} + \frac{1}{C_{D,C}} \qquad \text{…(E.11)}$$

$C_{D,A}$, $C_{D,C}$ rotational stiffnesses due to 10.1.5.2;

$C_{D,B}$ rotational stiffnesses due to distorsion of the cross-section of the purlin due to 10.1.5.1, $C_{D,B} = K_B\,h^2$, where h = depth of the purlin and K_B according to 10.1.5.1;

k lateral torsional buckling coefficient due to table E.4.

Table E.4: Lateral torsional buckling coefficients *k* for beams restraint horizontally at the upper flange

Statical system	Gravity load	Uplift load
	∞	10.3
	17.7	27.7
	12.2	18.3
	14.6	20.5

EUROPEAN STANDARD **EN 1993-1-3 : 2004**
NORME EUROPÉENNE
EUROPÄISCHE NORM 1 March 2004

UDC

Descriptors:

English version

Eurocode 3 : Design of steel structures

Part 1-3: General rules
Supplementary rules for cold-formed members and sheeting

Eurocode 3: Calcul des structures en acier - Partie 1-3: Règles générales - Règles supplémentaires pour les profilés et plaques à parois minces formés à froid

Eurocode 3: Bemessung und Konstruktion von Stahlbauten - Teil 1-3: Allgemeine Regels - Ergänzende Regeln fur kaltgeformte dunnwandige Bauteile und Bleche

Stage 34

CEN

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

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

 Ref. No. EN 1993-1-3 : 2004 E

National annex for EN 1993-1-3

This standard gives alternative procedures, values and recommendations for classes with notes indicating where national choices may have to be made. Therefore the National Standard implementing EN 1993-1-3 should have a National Annex containing all Nationally Determined Parameters to be used for the design of steel structures to be constructed in the relevant country.

National choice is allowed in EN 1993-1-3 through clauses:

- 2(3)
- 2(5)
- 3.1(4)
- 3.2.4(1)
- 5.3(4)
- 8.3(5)
- 8.3(13) (4 times)
- 8.4(5)
- 8.5.1(4)
- 9(2)
- 10.1.1(1)
- A.1(1) (2 times)
- A.6.4(4)

1 Introduction

1.1 Scope

(1) Part 1-3 of EN 1993 gives design requirements for cold-formed thin gauge members and sheeting. It applies to cold-formed steel products made from coated or uncoated thin gauge hot or cold rolled sheet or strip, that have been cold-formed by such processes as cold-rolled forming or press-braking. It may also be used for the design of profiled steel sheeting for composite steel and concrete slabs at the construction stage, see EN 1994. The execution of steel structures made of cold-formed thin gauge members and sheeting is covered in EN 1090.

NOTE The rules in this part complement the rules in other parts of EN 1993-1.

(2) Methods are also given for stressed-skin design using steel sheeting as a structural diaphragm.

(3) This part does not apply to cold-formed circular and rectangular structural hollow sections supplied to EN 10219, for which reference should be made to EN 1993-1-1 and EN 1993-1-8.

(4) This Part 1-3 of EN 1993 gives methods for design by calculation and for design assisted by testing. The methods for design by calculation apply only within stated ranges of material properties and geometrical proportions for which sufficient experience and test evidence is available. These limitations do not apply to design assisted by testing.

(5) EN 1993-1-3 does not cover load arrangement for testing for loads during execution and maintenance.

1.2 Normative references

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

EN 1993 *Eurocode 3 – Design of steel structures*

Part 1: *General rules and rules for buildings*

EN 10002 *Metallic materials - Tensile testing:*

Part 1: *Method of test (at ambient temperature)*;

EN 10142 *Continuously hot-dip zinc coated mild steel strip and sheet for cold-forming - Technical delivery conditions;*

EN 10143 *Continuously hot-dip metal coated steel sheet and strip - Tolerances on dimensions and shape*;

EN 10147 *Specification for continuously hot-dip zinc coated structural steel sheet - Technical delivery conditions*;

EN 10149 *Hot rolled flat products made of high yield strength steels for cold-forming:*

Part 2: *Delivery conditions for normalized/normalized rolled steels*;

Part 3: *Delivery conditions for thermomechanical rolled steels*;

EN 10154 *Continuously hot-dip aluminium-silicon (AS) coated steel strip and sheet - Technical delivery conditions;*

prEN 10162 *Cold rolled steel sections - Technical delivery conditions - Dimensional and cross-sectional tolerance;*

EN 10204 *Metallic products. Types of inspection documents (includes amendment A 1:1995);*

EN 10214 *Continuously hot-dip zinc-aluminium (ZA) coated steel strip and sheet - Technical delivery conditions;*

EN 10215 *Continuously hot-dip aluminium-zinc (AZ) coated steel strip and sheet - Technical delivery conditions;*

EN 10219-1 *Cold formed welded structural hollow sections of non-alloy and fine grain steels - Technical delivery requirements;*

EN 10219-2 *Cold formed welded structural hollow sections of non-alloy and fine grain steels - Tolerances, dimensions and sectional properties;*

EN 10268 *Cold-rolled flat products made of high yield strength micro-alloyed steels for cold forming - General delivery conditions;*

EN 10292 *Continuously hot-dip coated strip and sheet of steels with higher yield strength for cold forming - Technical delivery conditions;*

EN-ISO 12944-2 *Paints and vanishes. Corrosion protection of steel structures by protective paint systems. Part 2: Classification of environments (ISO 12944-2:1998);*

EN 1090, Part 2 *Requirements for the execution of steel structures:*

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

EN ISO 1478 (ISO 1478:1983) *Tapping screws thread;*

EN ISO 1479 (ISO 1479:1983) *Hexagon head tapping screws;*

EN ISO 2702 (ISO 2702:1992) *Heat-treated steel tapping screws - Mechanical properties;*

EN ISO 7049 (ISO 7049:1983) *Cross recessed pan head tapping screws;*

ISO 1000

ISO 4997 *Cold reduced steel sheet of structural quality*;

EN 508-1 *Roofing products from metal sheet - Specification for self-supporting products of steel, aluminium or stainless steel sheet - Part 1: Steel;*

FEM 10.2.02 *Federation Europeenne de la manutention, Secion X, Equipment et proceedes de stockage, FEM 10.2.02, The design of static steel pallet racking, Racking design code, April 2001 Version 1.02.*

1.3 Definitions

Supplementary to EN 1993-1-1, for the purposes of this Part 1-3 of EN 1993, the following definitions apply:

1.3.1
basic material
The flat sheet steel material out of which cold-formed sections and profiled sheets are made by cold-forming.

1.3.2
basic yield strength
The tensile yield strength of the basic material.

1.3.3
diaphragm action
Structural behaviour involving in-plane shear in the sheeting.

1.3.4
liner tray
Profiled sheet with large lipped edge stiffeners, suitable for interlocking with adjacent liner trays to form a plane of ribbed sheeting that is capable of supporting a parallel plane of profiled sheeting spanning perpendicular to the span of the liner trays.

1.3.5
partial restraint
Restriction of the lateral or rotational movement, or the torsional or warping deformation, of a member or element, that increases its buckling resistance in a similar way to a spring support, but to a lesser extent than a rigid support.

1.3.6
relative slenderness
A normalized non-dimensional slenderness ratio.

1.3.7
restraint
Restriction of the lateral or rotational movement, or the torsional or warping deformation, of a member or element, that increases its buckling resistance to the same extent as a rigid support.

1.3.8
stressed-skin design
A design method that allows for the contribution made by diaphragm action in the sheeting to the stiffness and strength of a structure.

1.3.9
support
A location at which a member is able to transfer forces or moments to a foundation, or to another member or other structural component.

1.3.10
nominal thickness
A target average thickness inclusive zinc and other metallic coating layers when present rolled and defined by the steel supplier (t_{nom} not including organic coatings).

1.3.11
steel core thickness
A nominal thickness minus zinc and other metallic coating layers (t_{cor}).

1.3.12
design thickness
the steel core thickness used in design by calculation according to 3.2.4.

1.4 Symbols

(1) In addition to those given in EN 1993-1-1, the following main symbols are used:

Draft note:	Will be added later

(2) Additional symbols are defined where they occur.

1.5 Terminology and conventions for dimensions

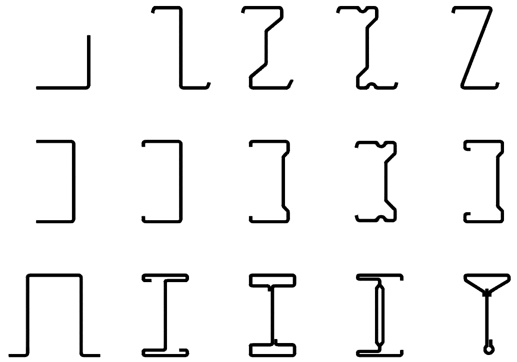
1.5.1 Form of sections

(1) Cold-formed members and profiled sheets have within the permitted tolerances a constant nominal thickness over their entire length and may have either a constant or a variable cross-section.

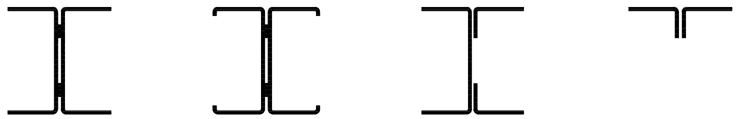
(2) The cross-sections of cold-formed members and profiled sheets essentially comprise a number of plane elements joined by curved elements.

(3) Typical forms of sections for cold-formed members are shown in figure 1.1.

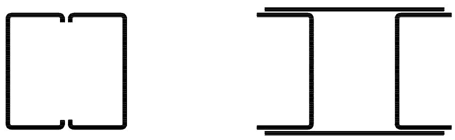
NOTE: The calculation methods of this Part 1-3 of EN 1993 does not cover all the cases shown in figures 1.1-1.2.



a) Single open sections



b) Open built-up sections

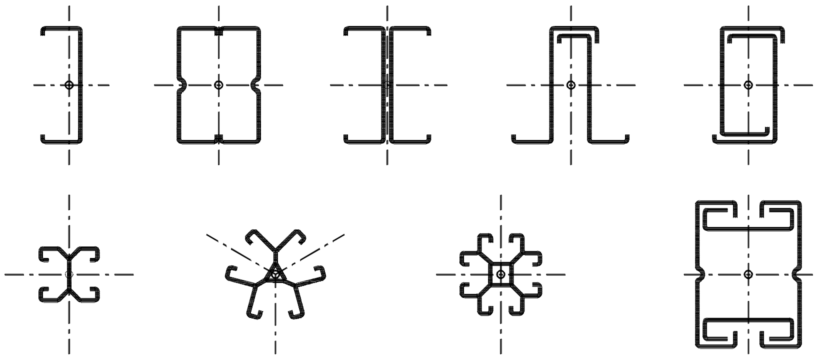


c) Closed built-up sections

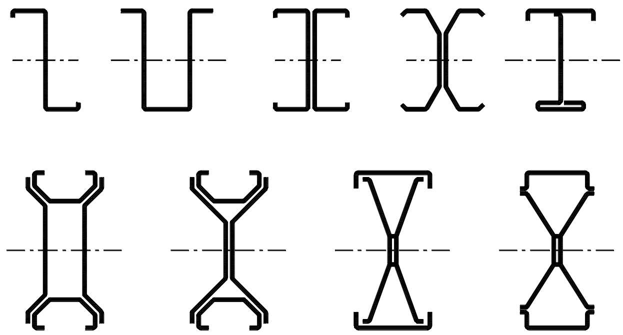
Figure 1.1: Typical forms of sections for cold-formed members

(4) Examples of cross-sections for cold-formed members and sheets are illustrated in figure 1.2.

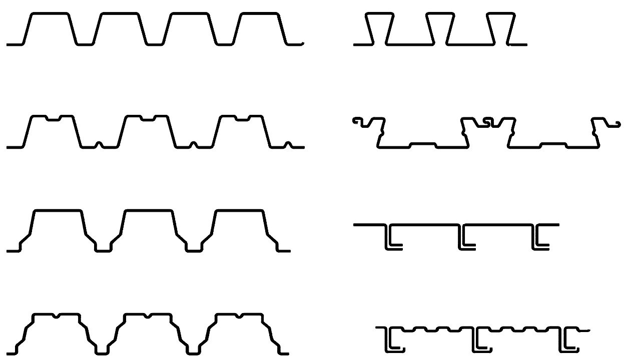
NOTE: All rules in this Part 1-3 of EN 1993 relate to the main axis properties, which are defined by the main axes y - y and z - z for symmetrical sections and u - u and v - v for unsymmetrical sections as e.g. angles and Zed-sections. In some cases the bending axis is imposed by connected structural elements whether the cross-section is symmetric or not.



a) Compression members and tension members



b) Beams and other members subject to bending



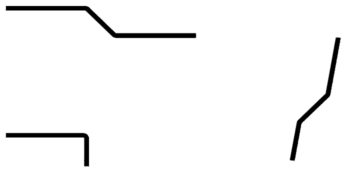
c) Profiled sheets and liner trays

Figure 1.2: Examples of cold-formed members and profiled sheets

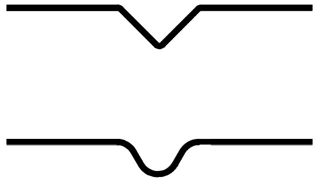
(5) Cross-sections of cold-formed members and sheets may either be unstiffened or incorporate longitudinal stiffeners in their webs or flanges, or in both.

1.5.2 Form of stiffeners

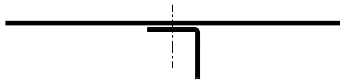
(1) Typical forms of stiffeners for cold-formed members and sheets are shown in figure 1.3.



a) Folds and bends



b) Folded groove and curved groove



c) Bolted angle stiffener

Figure 1.3: Typical forms of stiffeners for cold-formed members and sheeting

(2) Longitudinal flange stiffeners may be either edge stiffeners or intermediate stiffeners.

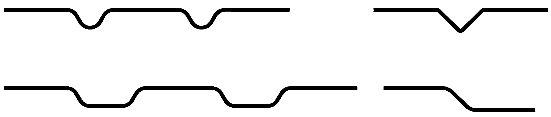
(3) Typical edge stiffeners are shown in figure 1.4.

a) Single edge fold stiffeners

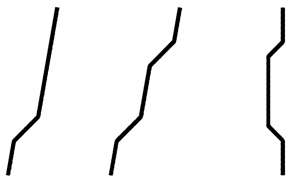
b) Double edge fold stiffeners

Figure 1.4: Typical edge stiffeners

(4) Typical intermediate longitudinal stiffeners are illustrated in figure 1.5.



a) Intermediate flange stiffeners



b) Intermediate web stiffeners

Figure 1.5: Typical intermediate longitudinal stiffeners

1.5.3 Cross-section dimensions

(1) Overall dimensions of cold-formed thin gauge members and sheeting, including overall width b, overall height h, internal bend radius r and other external dimensions denoted by symbols without subscripts, such as a, c or d, are measured to the face of the material, unless stated otherwise, as illustrated in figure 1.6.

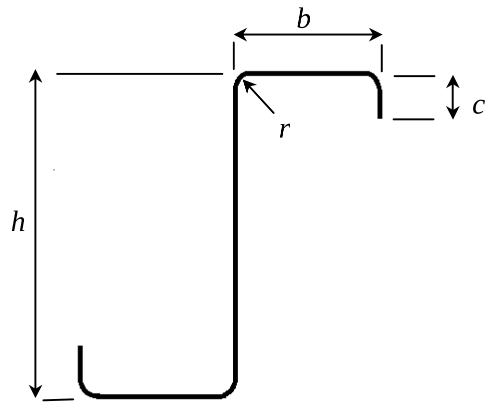


Figure 1.6: Dimensions of typical cross-section

(2) Unless stated otherwise, the other cross-sectional dimensions of cold-formed thin gauge members and sheeting, denoted by symbols with subscripts, such as b_d, h_w or s_w, are measured either to the midline of the material or the midpoint of the corner.

(3) In the case of sloping elements, such as webs of trapezoidal profiled sheets, the slant height s is measured parallel to the slope. The slope is straight line between intersection points of flanges and web.

(4) The developed height of a web is measured along its midline, including any web stiffeners.

(5) The developed width of a flange is measured along its midline, including any intermediate stiffeners.

(6) The thickness t is a steel design thickness (the steel core thickness extracted minus tolerance if needed as specified in clause 3.2.4), if not otherwise stated.

1.5.4 Convention for member axes

(1) In general the conventions for members is as used in Part 1-1 of EN 1993, see Figure 1.7.

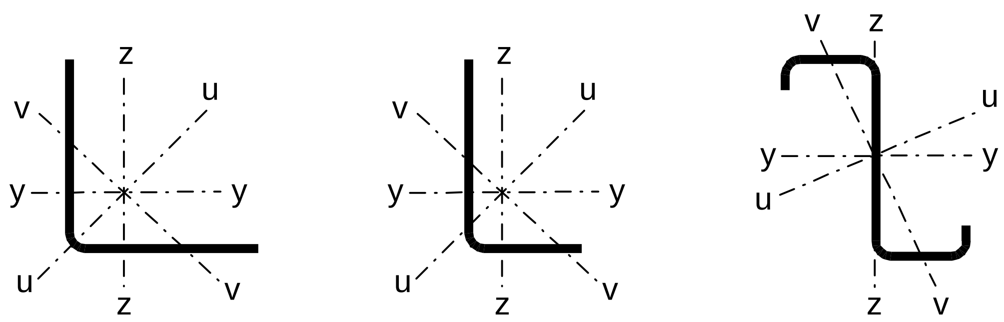


Figure 1.7: Axis convention

(2) For profiled sheets and liner trays the following axis convention is used:

- y - y axis parallel to the plane of sheeting;
- z - z axis perpendicular to the plane of sheeting.

2 Basis of design

(1) The design of cold formed thin gauge members and sheeting shall be in accordance with the general rules given in EN 1990 and EN 1993-1-1. For a general approach with FE-methods (or others) see EN 1993-1-5, Annex C.

(2) Appropriate partial factors shall be adopted for ultimate limit states and serviceability limit states.

(3) For verifications by calculation at ultimate limit states the partial factor γ_M shall be taken as follows:

- resistance of cross-sections to excessive yielding including local and distortional buckling: γ_{M0}
- resistance of members and sheeting where failure is caused by global buckling: γ_{M1}
- resistance of net sections at bolt holes: γ_{M2}

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

$\gamma_{M0} = 1,00$;

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

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

(4) For values of γ_M for resistance of connections, see Section 8 of this Part 1-3.

(5) For verifications at serviceability limit states the partial factor $\gamma_{M,ser}$ shall be used.

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

$\gamma_{M,ser} = 1,00$.

(6) For the design of structures made of cold formed thin gauge members and sheeting a distinction should be made between "structural classes" associated with failure consequences according to EN 1990 – Annex B defined as follows:

Structural Class I: Construction where cold-formed thin gauge members and sheeting are designed to contribute to the overall strength and stability of a structure;

Structural Class II: Construction where cold-formed thin gauge members and sheeting are designed to contribute to the strength and stability of individual structural elements;

Structural Class III: Construction where cold-formed sheeting is used as an element that only transfers loads to the structure.

NOTE 1: During different construction stages different construction classes may be considered.
NOTE 2: For requirements for execution of sheeting in structural classes I, II and III see EN 1090.

3 Materials

3.1 General

(1) All steels used for cold-formed members and profiled sheets shall be suitable for cold-forming and welding, if needed. Steels used for members and sheets to be galvanized shall also be suitable for galvanizing.

(2) The nominal values of material properties given in this Section should be adopted as characteristic values in design calculations.

(3) This part of EN 1993 covers the design of cold formed members and profiles sheets fabricated from steel material conforming to the steel grades listed in table 3.1.

(4) Other materials and products not specified in European Product Standards may only be used if their use is evaluated in accordance with the relevant rules in this standard and the applicable National Annex.

NOTE: For other steel materials and products see National Annex. Examples for steel grades that may conform to the requirements of this standard are given in Table 3.1b. It is assued that for materials for which the nominal ultimate tensile strength is higher than 550 N/mm^2 the resistance and ductility is verified by testing.

Table 3.1a: Nominal values of basic yield strength f_{yb} and ultimate tensile strength f_u

Type of steel	Standard	Grade	f_{yb} N/mm^2	f_u N/mm^2
Hot rolled products of non-alloy structural steels. Part 2: Technical delivery conditions for non alloy structural steels	EN 10025: Part 2	S 235	235	360
		S 275	275	430
		S 355	355	510
Hot-rolled products of structural steels. Part 3: Technical delivery conditions for normalized/normalized rolled weldable fine grain structural steels	EN 10025: Part 3	S 275 N	275	370
		S 355 N	355	470
		S 420 N	420	520
		S 460 N	460	550
		S 275 NL	275	370
		S 355 NL	355	470
		S 420 NL	420	520
		S 460 NL	460	550
Hot-rolled products of structural steels. Part 4: Technical delivery conditions for thermomechanical rolled weldable fine grain structural steels	EN 10025: Part 5	S 275 M	275	360
		S 355 M	355	450
		S 420 M	420	500
		S 460 M	460	530
		S 275 ML	275	360
		S 355 ML	355	450
		S 420 ML	420	500
		S 460 ML	460	530

1) Minimum values of the yield strength and ultimate tensile strength are not given in the standard. For all steel grades a minimum value of 140 N/mm^2 for yield strength and 270 N/mm^2 for ultimate tensile strength may be assumed.

2) The yield strength values given in the names of the materials correspond to transversal tension. The values for longitudinal tension are given in the table.

Table 3.1b: Nominal values of basic yield strength f_{yb} and ultimate tensile strength f_u

Cold reduced steel sheet of structural quality	ISO 4997	CR 220 CR 250 CR 320	220 250 320	300 330 400
Continuous hot dip zinc coated carbon steel sheet of structural quality	EN 10147	S220GD+Z S250GD+Z S280GD+Z S320GD+Z S350GD+Z	220 250 280 320 350	300 330 360 390 420
Hot-rolled flat products made of high yield strength steels for cold forming. Part 2: Delivery conditions for thermomechanically rolled steels	EN 10149: Part 2	S 315 MC S 355 MC S 420 MC S 460 MC S 500 MC S 550 MC S 600 MC S 650 MC S 700 MC	315 355 420 460 500 550 600 650 700	390 430 480 520 550 600 650 700 750
	EN 10149: Part 3	S 260 NC S 315 NC S 355 NC S 420 NC	260 315 355 420	370 430 470 530
Cold-rolled flat products made of high yield strength micro-alloyed steels for cold forming	EN 10268	H240LA H280LA H320LA H360LA H400LA	240 280 320 360 400	340 370 400 430 460
Continuously hot-dip coated strip and sheet of steels with higher yield strength for cold forming	EN 10292	H260LAD H300LAD H340LAD H380LAD H420LAD	240 2) 280 2) 320 2) 360 2) 400 2)	340 2) 370 2) 400 2) 430 2) 460 2)
Continuously hot-dipped zinc-aluminium (ZA) coated steel strip and sheet	EN 10214	S220GD+ZA S250GD+ZA S280GD+ZA S320GD+ZA S350GD+ZA	220 250 280 320 350	300 330 360 390 420
Continuously hot-dipped aluminium-zinc (AZ) coated steel strip and sheet	EN 10215	S220GD+ZA S250GD+ZA S280GD+ZA S320GD+ZA S350GD+ZA	220 250 280 320 350	300 330 360 390 420
Continuously hot-dipped zinc coated strip and sheet of mild steel for cold forming	EN 10142	DX51D+Z DX52D+Z DX53D+Z	140 1) 140 1) 140 1)	270 1) 270 1) 270 1)

1) Minimum values of the yield strength and ultimate tensile strength are not given in the standard. For all steel grades a minimum value of 140 N/mm^2 for yield strength and 270 N/mm^2 for ultimate tensile strength may be assumed.

2) The yield strength values given in the names of the materials correspond to transversal tension. The values for longitudinal tension are given in the table.

Drafting note: References to other technical specifications not included to Table 3.1 to be given.

3.2 Structural steel

3.2.1 Material properties of base material

(1) The nominal values of yield strength f_{yb} or tensile strength f_u shall be obtained

a) either by adopting the values $f_y = R_{eH}$ or $R_{p0,2}$ and $f_u = R_m$ direct from product standards, or

b) by using the values given in Table 3.1

c) by appropriate tests.

(2) Where the characteristic values are determined from tests, such tests shall be carried out in accordance with EN 10002-1. The number of test coupons should be at least 5 and should be taken from a lot in following way:

1. Coils: a. For a lot from one production (one pot of melted steel) at least one coupon per coil of 30% of the number of coils;

   b. For a lot from different productions at least one coupon per coil;

2. Strips: At least one coupon per 2000 kg from one production.

The coupons should be taken at random from the concerned lot of steel and the orientation should be in the length of the structural element. The characteristic values shall be determined on basis of a statistical evaluation in accordance with EN 1990 Annex D.

(3) It may be assumed that the properties of steel in compression are the same as those in tension.

(4) The ductility requirements should comply with 3.2.2 of EN 1993-1-1.

(5) The design values for material coefficients shall be taken as given in 3.2.6 of EN 1993-1-1

(6) The material properties for elevated temperatures are given in EN 1993-1-2.

3.2.2 Material properties of cold formed sections and sheeting

(1) Where the yield strength is specified using the symbol f_y the average yield strength f_{ya} may be used, unless in (4) to (8) apply. In that case the basic yield strength f_{yb} shall be used. Where the yield strength is specified using the symbol f_{yb} the basic yield strength f_{yb} shall be used.

(2) The average yield strength f_{ya} of a cross-section due to cold working may be determined from the results of full size tests.

(3) Alternatively the increased average yield strength f_{ya} may be determined by calculation using:

$$f_{ya} = f_{yb} + \left(f_u - f_{yb}\right)\frac{knt^2}{A_g} \qquad \text{but} \qquad f_{ya} \leq \frac{\left(f_u + f_{yb}\right)}{2} \qquad \ldots (3.1)$$

where:

A_g is the gross cross-sectional area;

k is a numerical coefficient that depends on the type of forming as follows:

- $k = 7$ for roll forming;
- $k = 5$ for other methods of forming;

n is the number of $90°$ bends in the cross-section with an internal radius $r \leq 5t$ (fractions of $90°$ bends should be counted as fractions of n);

t is the design core thickness of the steel material before cold-forming, exclusive of metal and organic coatings, see 3.2.4.

(4) The increased yield strength due to cold forming may be taken into account as follows:

- in axially loaded members in which the effective cross-sectional area A_{eff} equals the gross area A_g;
- in determining A_{eff} the yield strength f_y should be taken as f_{yb}.

(5) The average yield strength f_{ya} may be utilised in determining:

- the cross-section resistance of an axially loaded tension member;
- the cross-section resistance and the buckling resistance of an axially loaded compression member with a fully effective cross-section;
- the moment resistance of a cross-section with fully effective flanges.

(6) To determine the moment resistance of a cross-section with fully effective flanges, the cross-section may be subdivided into m nominal plane elements, such as flanges. Expression (3.1) may then be used to obtain values of increased yield strength $f_{y,i}$ separately for each nominal plane element i, provided that:

$$\frac{\sum_{i=1}^{m} A_{g,i}\, f_{y,i}}{\sum_{i=1}^{m} A_{g,i}} \leq f_{ya} \quad \dots (3.2)$$

where:

$A_{g,i}$ is the gross cross-sectional area of nominal plane element i,

and when calculating the increased yield strength $f_{y,i}$ using the expression (3.1) the bends on the edge of the nominal plane elements should be counted with the half their angle for each area $A_{g,i}$.

(7) The increase in yield strength due to cold forming shall not be utilised for members that are subjected to heat treatment after forming at more than 580°C for more than one hour.

NOTE: For further information see EN 1090, Part 2.

(8) Special attention should be paid to the fact that some heat treatments (especially annealing) might induce a reduced yield strength lower than the basic yield strength f_{yb}.

NOTE: For welding in cold formed areas see also EN 1993-1-8.

3.2.3 Fracture toughness

(1) See EN 1993-1-1 and EN 1993-1-10.

3.2.4 Thickness and thickness tolerances

(1) The provisions for design by calculation given in this Part 1-3 of EN 1993 may be used for steel within given ranges of core thickness t_{cor}:

NOTE: The ranges of core thickness t_{cor} for sheeting and members may be given in the National Annex. The following values are recommended:

- for sheeting and members: 0,45 mm $\leq t_{cor} \leq$ 15 mm except where otherwise specified, e.g. for joints in section 8 , where tcor ≤ 4 mm.

(2) Thicker or thinner material may also be used, provided that the load bearing resistance is determined by design assisted by testing.

(3) The steel core thickness t_{cor} should be used as design thickness, where

$$t_{cor} = (t_{nom} - t_{metallic\ coatings}) \qquad \text{if } tol \leq 5\% \quad \dots (3.3a)$$

$$t_{cor} = (t_{nom} - t_{metallic\ coatings})\frac{100 - tol}{95} \qquad \text{if } tol > 5\% \quad \dots (3.3b)$$

where *tol* is the minus tolerance in %.

(4) For continuously hot-dip metal coated members and sheeting supplied with negative tolerances less or equal to the “special tolerances (S)” given in EN 10143, the design thickness according to (3.3a) may be used. If the negative tolerance is beyond "special tolerance (S)" given in EN 10143 then the design thickness according to (3.3b) may be used.

(5) t_{nom} is the nominal sheet thickness after cold forming. It may be taken as the value to t_{nom} of the original sheet, if the calculative cross-sectional areas before and after cold forming do not differ more than 2%; otherwise the notional dimensions should be changed.

NOTE: For the usual Z 275 zinc coating, t_{zinc} = 0,04 mm.

3.3 Connecting devices

3.3.1 Bolt assemblies

(1) Bolts, nuts and washers shall conform to the requirements given in EN 1993-1-8.

3.3.2 Other types of mechanical fastener

(1) Other types of mechanical fasteners as:

- self-tapping screws as thread forming self-tapping screws, thread cutting self-tapping screws or self-drilling self-tapping screws,
- cartridge-fired pins,
- blind rivets

may be used where they comply with the relevant EN Product Standards or ETAG or ETA.

(2) The characteristic shear resistance $F_{v,Rk}$ and the characteristic minimum tension resistance $F_{t,Rk}$ of the mechanical fasteners may be taken from the EN Product Standard or ETAG ir ETA.

3.3.3 Welding consumables

(1) Welding consumables shall conform to the requirements given in EN 1993-1-8.

4 Durability

(1) For basic requirements see section 4 of EN 1993-1-1.

NOTE: EN 1090 lists the factors affecting execution that need to be specified during design.

(2) Special attention should be given to cases in which different materials are intended to act compositely, if these materials are such that electrochemical phenomena might produce conditions leading to corrosion.

NOTE 1 For corrosion resistance of fasteners for the environmental class following EN-ISO 12944-2 see Annex B

NOTE 2: For roofing products see EN 508-1.

NOTE 3: For other products see Part 1-1 of EN 1993.

5 Structural analysis

5.1 Influence of rounded corners

(1) In cross-sections with rounded corners, the notional flat widths b_p of the plane elements shall be measured from the midpoints of the adjacent corner elements as indicated in figure 5.3.

(2) In cross-sections with rounded corners, the calculation of section properties should be based upon the actual geometry of the cross-section.

(3) Unless more appropriate methods are used to determine the section properties the followign approximate procedure may be used. The influence of rounded corners on cross-section resistance may be neglected if the internal radius $r \leq 5\,t$ and $r \leq 0{,}10\,b_p$ and the cross-section may be assumed to consist of plane elements with sharp corners (according to figure 5.4, note b_p for all flat plane elements, inclusive plane elements in tension). For cross-section stiffness properties the influence of rounded corners should always be taken into account.

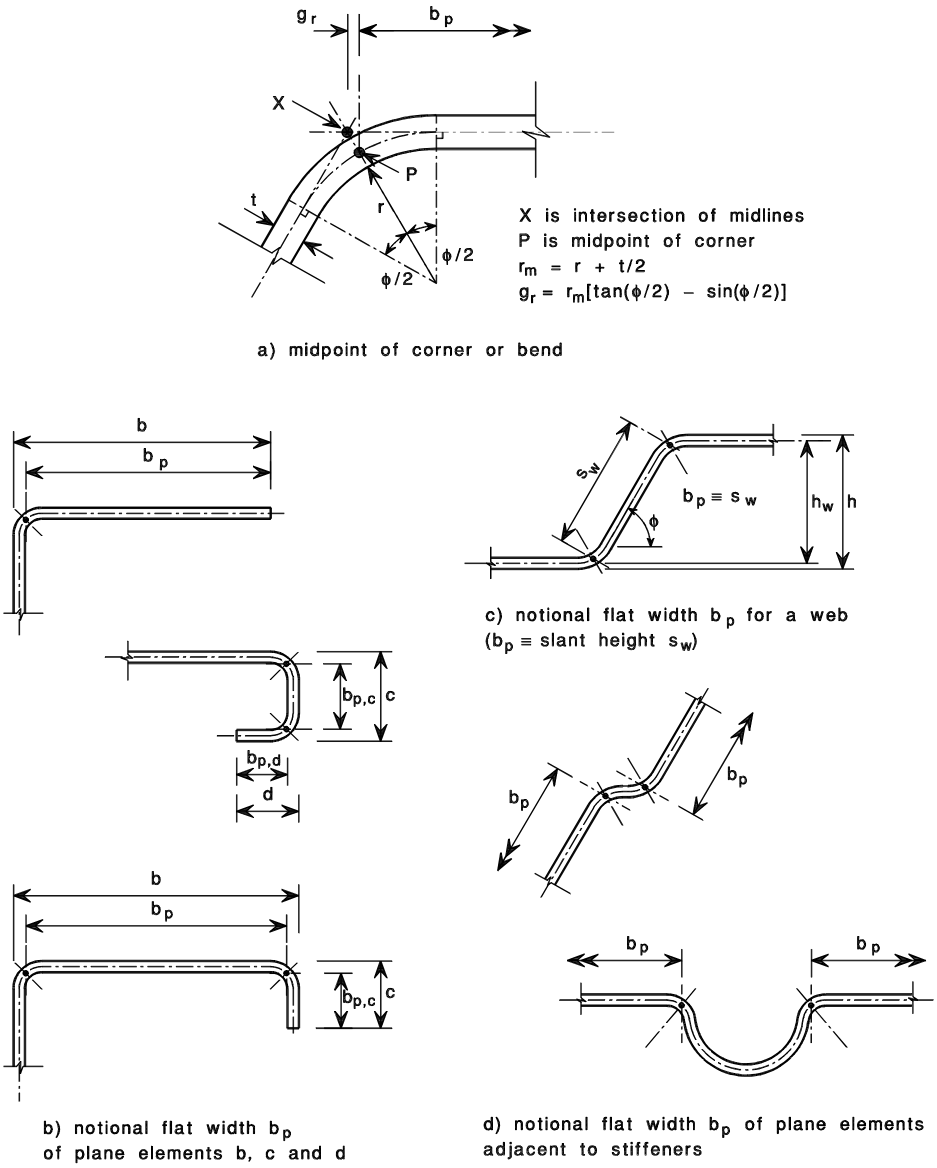


Figure 5.3: Notional widths of plane elements b allowing for corner radii

(4) The influence of rounded corners on section properties may be taken into account by reducing the properties calculated for an otherwise similar cross-section with sharp corners, see figure 5.4, using the following approximations:

$$A_g \approx A_{g,sh}(1 - \delta) \qquad \dots (5.1a)$$

$$I_g \approx I_{g,sh}(1 - 2\delta) \qquad \dots (5.1b)$$

$$I_w \approx I_{w,sh}(1 - 4\delta) \qquad \dots (5.1c)$$

with:

$$\delta = 0{,}43 \frac{\sum_{j=1}^{n} r_j \frac{\phi_j}{90°}}{\sum_{i=1}^{m} b_{p,i}} \quad \text{... (5.1d)}$$

where:

A_g	is	the area of the gross cross-section;
$A_{g,sh}$	is	the value of A_g for a cross-section with sharp corners;
$b_{p,i}$	is	the notional flat width of plane element i for a cross-section with sharp corners;
I_g	is	the second moment of area of the gross cross-section;
$I_{g,sh}$	is	the value of I_g for a cross-section with sharp corners;
I_w	is	the warping constant of the gross cross-section;
$I_{w,sh}$	is	the value of I_w for a cross-section with sharp corners;
ϕ	*is*	*the angle between two plane elements;*
m	is	the number of plane elements;
n	is	the number of curved elements;
r_j	is	the internal radius of curved element j.

(5) The reductions given by expression (5.1) may also be applied in calculating the effective section properties A_{eff}, $I_{y,eff}$, $I_{z,eff}$ and $I_{w,eff}$, provided that the notional flat widths of the plane elements are measured to the points of intersection of their midlines.

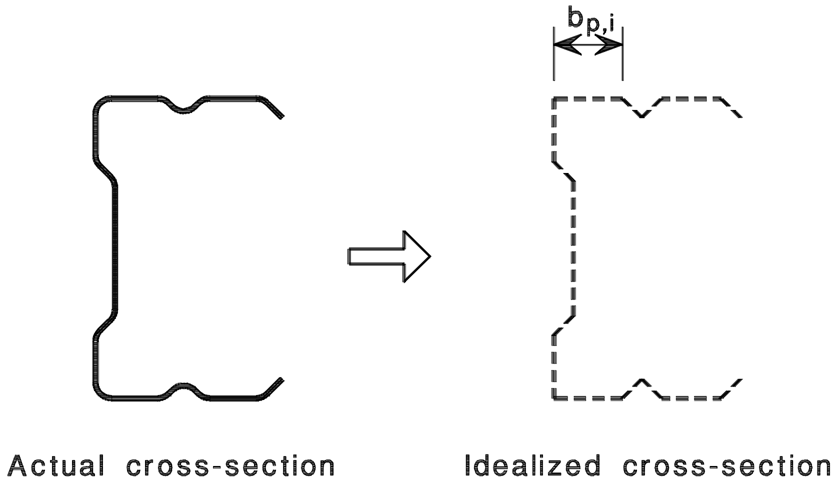


Figure 5.4: Approximate allowance for rounded corners

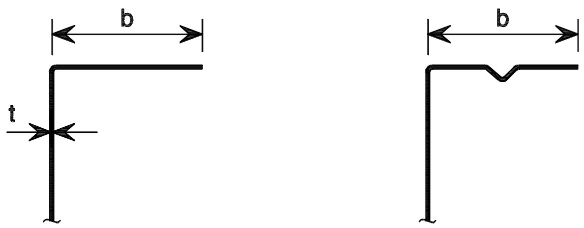
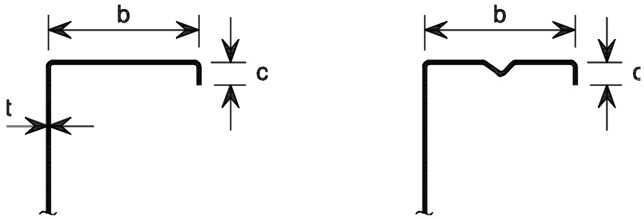
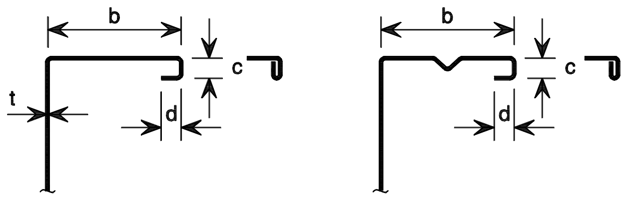
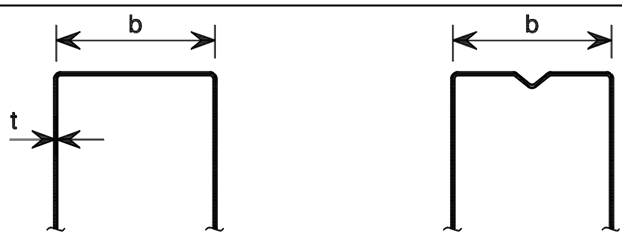
(6) Where the internal radius $r > 0{,}04\, t\, E / f_y$ then the resistance of the cross-section should be determined by tests.

5.2 Geometrical proportions

(1) The provisions for design by calculation given in this Part 1-3 of EN 1993 shall not be applied to cross-sections outside the range of width-to-thickness ratios b/t and h/t given in Table 5.1.

NOTE These limits b/t and h/t given in table 5.1 may be assumed to represent the field for which sufficient experience and verification by testing is already available. Cross-sections with larger width-to-thickness ratios may also be used, provided that their resistance at ultimate limit states and their behaviour at serviceability limit states are verified by testing and/or by calculations, where the results are confirmed by an appropriate number of tests.

Table 5.1: Maximum width-to-thickness ratios

Element of cross-section	Maximum value
	$b/t \leq 50$
	$b/t \leq 60$ $c/t \leq 50$
	$b/t \leq 90$ $c/t \leq 60$ $d/t \leq 50$
	$b/t \leq 500$
	$45° \leq \phi \leq 90°$ $h/t \leq 500 \sin\phi$

(2) In order to provide sufficient stiffness and to avoid primary buckling of the stiffener itself, the sizes of stiffeners should be within the following ranges:

$$0{,}2 \leq c/b \leq 0{,}6 \qquad \text{… (5.2a)}$$

$$0{,}1 \leq d/b \leq 0{,}3 \qquad \text{… (5.2b)}$$

in which the dimensions b, c and d are as indicated in table 5.1. If $c/b \leq$ 0,2 or $d/b \leq$ 0,1 the lip should be ignored (c = 0 or d = 0).

NOTE 1 Where effective cross-section properties are determined by testing and by calculations, these limits do not apply.

NOTE 2: The lip measure c is perpendicular to the flange if the lip is not perpendicular to the flange.

NOTE 3 For FE-methods see Annex C of EN 1993-1-5.

5.3 Structural modelling for analysis

(1) Unless more appropriate models are used according to EN 1993-1-5 the elements of a cross-section may be modelled for analysis as indicated in table 5.2.

(2) The mutual influence of multiple stiffeners should be taken into account.

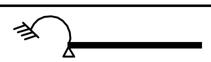
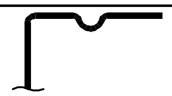
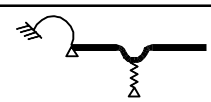
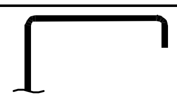
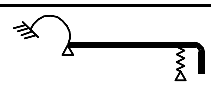
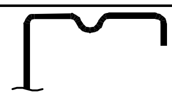
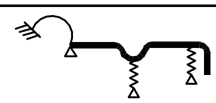
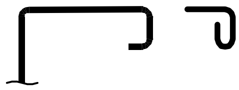
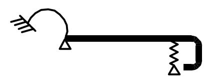
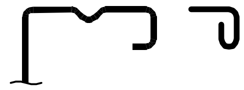
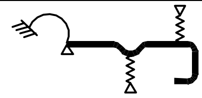
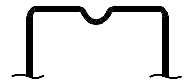
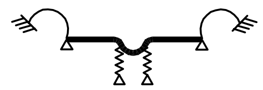
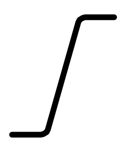
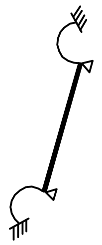
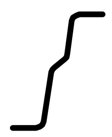
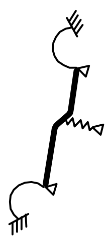
(3) Imperfections related to flexural buckling and torsional flexural buckling should be taken from table 5.1 of EN 1993-1-1

NOTE See also clause 5.3.4 of EN 1993-1-1.

(4) For imperfections related to lateral torsional buckling an initial bow imperfections e_0 of the weak axis of the profile may without assumed taking account at the same time an initial twist

NOTE The magnitude of the imperfection may be taken from the National Annex. The value $L/e_0 = 600$ is recommended for sections assigned to LTB buckling curve b;

Table 5.2: Modelling of elements of a cross-section

Type of element	Model	Type of element	Model
			
			
			
			
			

5.4 Flange curling

(1) The effect on the loadbearing resistance of curling (i.e. inward curvature towards the neutral plane) of a very wide flange in a profile subjected to flexure, or of a flange in an arched profile subjected to flexure in which the concave side is in compression, should be taken into account unless such curling is less than 5% of the depth of the profile cross-section. If curling is larger, then the reduction in loadbearing resistance, for instance due to a decrease in the length of the lever arm for parts of the wide flanges, and to the possible effect of the bending of the webs should be taken into account.

NOTE: For liner trays this effect has been taken into account in 10.2.2.2.

(2) Calculation of the curling may be carried out as follows. The formulae apply to both compression and tensile flanges, both with and without stiffeners, but without closely spaced transversal stiffeners at flanges. For a profile which is straight prior to application of loading (see figure 5.5),

$$u = 2\frac{\sigma_a^{\ 2}}{E^2}\frac{b_s^{\ 4}}{t^2 z} \quad \text{... (5.3a)}$$

For an arched beam:

$$u = 2\frac{\sigma_a}{E}\frac{b_s^{\ 4}}{t^2 r} \quad \text{... (5.3b)}$$

where:

- u is bending of the flange towards the neutral axis (curling), see figure 5.5;
- b_s is one half the distance between webs in box and hat sections, or the width of the portion of flange projecting from the web, see figure 5.5;
- t is flange thickness;
- z is distance of flange under consideration from neutral axis;
- r is radius of curvature of arched beam;
- σ_a is mean stress in the flanges calculated with gross area. If the stress has been calculated over the effective cross-section, the mean stress is obtained by multiplying the stress for the effective cross-section by the ratio of the effective flange area to the gross flange area.

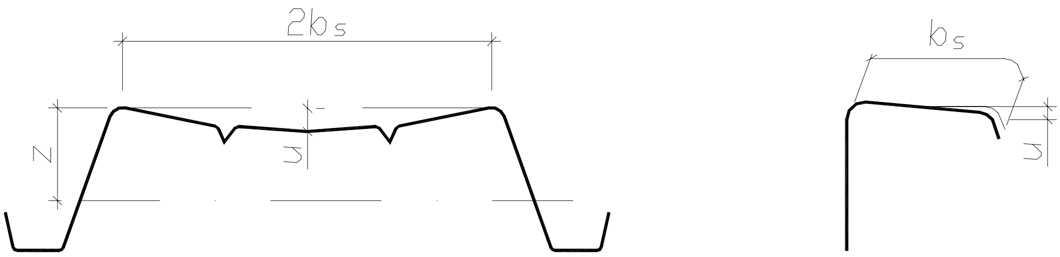


Figure 5.5: Flange curling

5.5 Local and distortional buckling

5.5.1 General

(1) The effects of local and distortional buckling shall be taken into account in determining the resistance and stiffness of cold-formed members and sheeting.

(2) Local buckling effects may be accounted for by using effective cross-sectional properties, calculated on the basis of the effective widths, see EN 1993-1-5.

(3) In determining resistance to local buckling, the yield strength f_y should be taken as f_{yb}.

(4) For serviceability verifications, the effective width of a compression element should be based on the compressive stress $\sigma_{com,Ed,ser}$ in the element under the serviceability limit state loading.

(5) The distortional buckling for elements with edge or intermediate stiffeners as indicated in figure 5.6(d) are considered in Section 5.5.3.

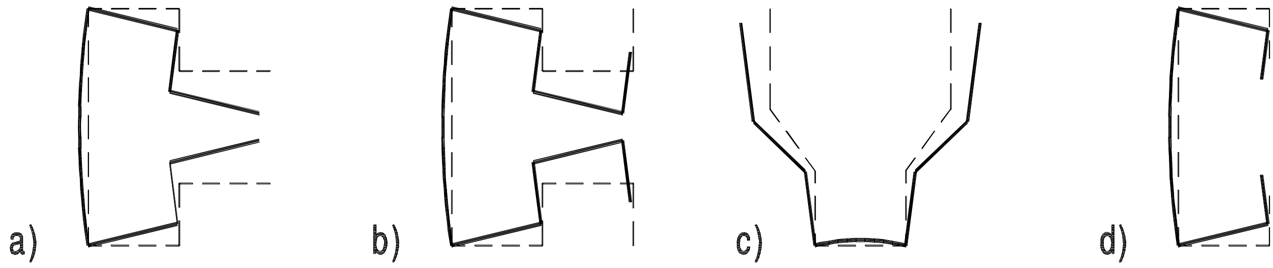


Figure 5.6: Examples of distortional buckling modes

(6) The effects of distortional buckling should be allowed for in cases such as those indicated in figures 5.6(a), (b) and (c). In these cases the effects of distortional buckling should be determined performing linear (see 5.5.1(8)) or non-linear buckling analysis (see EN 1993-1-5) using numerical methods or column stub tests.

(7) Unless the simplified procedure in 5.5.3 is used and where the elastic buckling stress is obtained from linear buckling analysis the following procedure may be applied:

1) For the wavelength up to the actual member length, calculate the elastic buckling stress and identify the corresponding buckling modes, see figure 5.7a.
2) Calculate the effective width(s) according to 5.5.2 for locally buckled cross-section parts based on the minimum local buckling stress, see figure 5.7b.
3) Calculate the reduced thickness (see 5.5.3.1(7)) of edge and intermediate stiffeners or other cross-section parts undergoing distortional buckling based on the minimum distortional buckling stress, see figure 5.7b.
4) Calculate overall buckling resistance according to 6.2 (flexural, torsional or lateral-torsional buckling depending on buckling mode) for actual member length and based on the effective cross-section from 2) and 3).

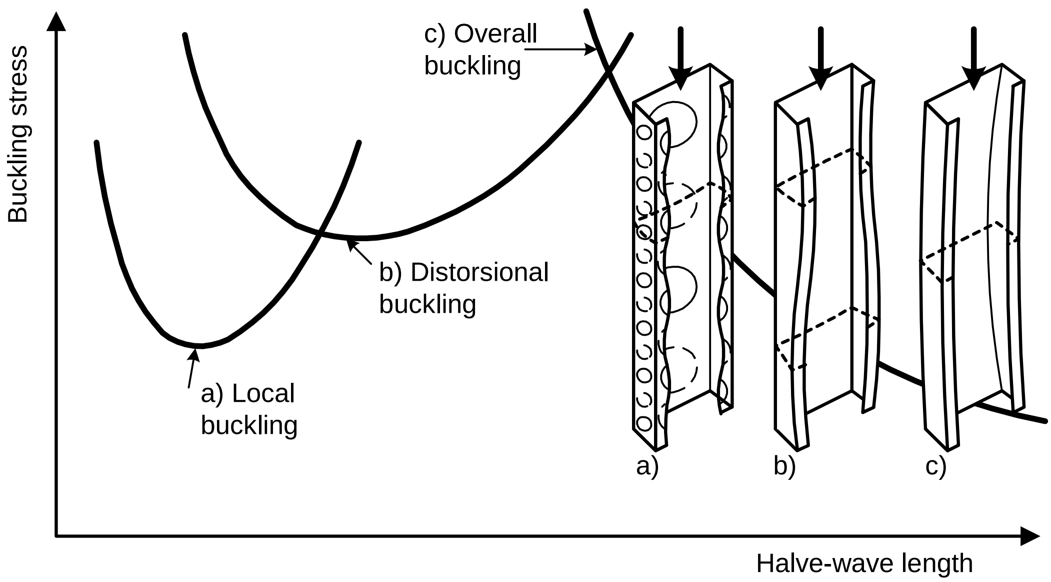


Figure 5.7a: Examples of elastic critical stress for various buckling modes as function of *halve-wave length* and examples of buckling modes.

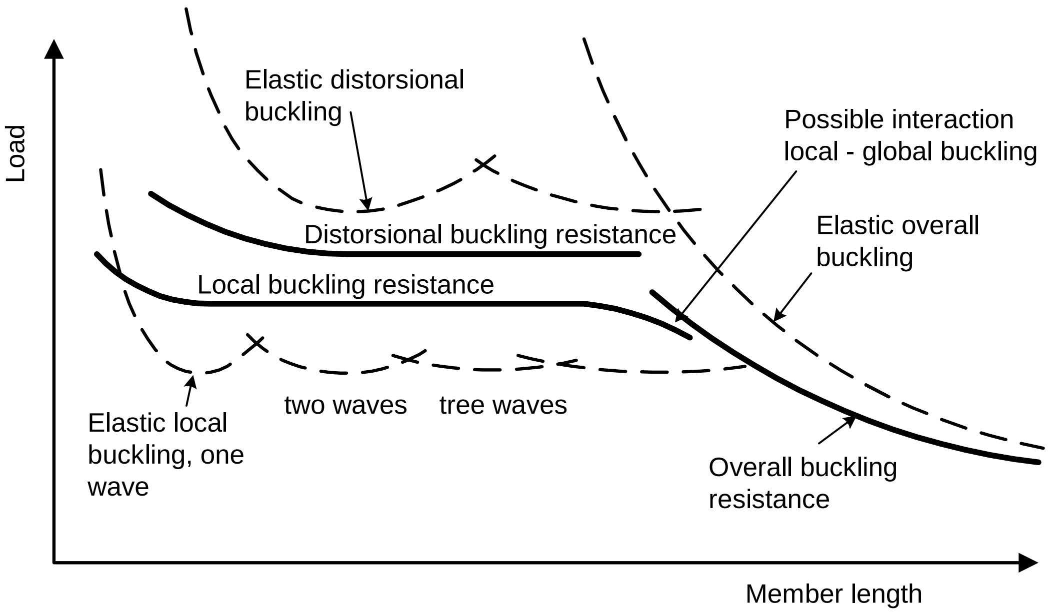


Figure 5.7b: Examples of elastic buckling load and buckling resistance as a function of *member length*

5.5.2 Plane elements without stiffeners

(1) The effective widths of unstiffened elements should be obtained from EN 1993-1-5 using the notional flat width b_p for $\overline{b}$.

(2) The notional flat width b_p of a plane element should be determined as specified in figure 5.3 of section 5.1.4. In the case of plane elements in a sloping webs, the appropriate slant height should be used.

NOTE For outstands a more refined method for calculating effective widths is given in Annex D.

(3) In applying the method in EN 1993-1-5 the following procedure may be used:

- The stress ratio ψ, from tables 5.3 and 5.4 used to determine the effective width of flanges of a section subject to stress gradient, may be based on gross section properties.
- The stress ratio ψ, from table 5.3 and 5.4 used to determine the effective width of web, may be obtained using the effective area of compression flange and the gross area of the web.
- The effective section properties may be refined by repeating (7) an (8) iteratively, but using in (7) the stress ratio ψ based the effective cross-section already found in place of the gross cross-section. The minimum steps in the iteration dealing with the stress gradient are two.
- The simplified method given in 5.5.3.4 may be used in the case of webs of trapetzoidal sheeting under stress gradient.

5.5.3 Plane elements with edge or intermediate stiffeners

5.5.3.1 General

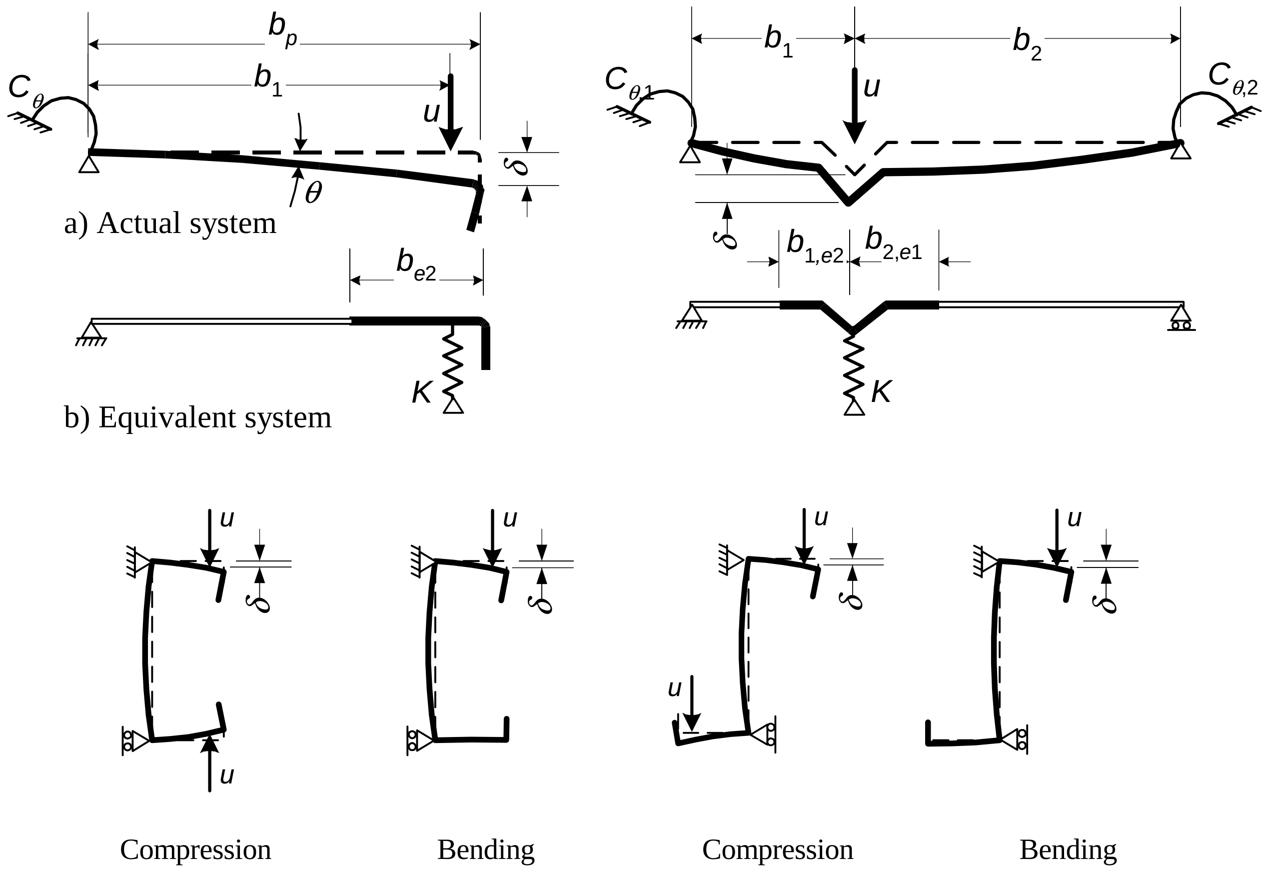
(1) The design of compression elements with edge or intermediate stiffeners should be based on the assumption that the stiffener behaves as a compression member with continuous partial restraint, with a spring stiffness that depends on the boundary conditions and the flexural stiffness of the adjacent plane elements.

(2) The spring stiffness of a stiffener should be determined by applying an unit load per unit length u as illustrated in figure 5.8. The spring stiffness K per unit length may be determined from:

$$K = u/\delta \qquad \text{... (5.9)}$$

where:

δ is the deflection of the stiffener due to the unit load u acting in the centroid (b_1) of the effective part of the cross-section.



c) Calculation of δ for C and Z sections

Figure 5.8: Determination of spring stiffness

(3) In determining the values of the rotational spring stiffnesses C_θ, $C_{\theta,1}$ and $C_{\theta,2}$ from the geometry of the cross-section, account should be taken of the possible effects of other stiffeners that exist on the same element, or on any other element of the cross-section that is subject to compression.

(4) For an edge stiffener, the deflection δ should be obtained from:

$$\delta = \theta\, b_{\mathrm{p}} + \frac{u b_{\mathrm{p}}^{3}}{3} \cdot \frac{12\left(1-\nu^{2}\right)}{E t^{3}} \qquad \text{... (5.10)}$$

with:

$$\theta = u\, b_{\mathrm{p}} / C_\theta$$

(5) In the case of the edge stiffeners of lipped C-sections and lipped Z-sections, C_θ should be determined with the unit loads u applied as shown in figure 5.8(c). This results in the following expression for the spring stiffness K_1 for the flange 1:

$$K_1 = \frac{E\, t^3}{4\,(1-\nu^2)} \cdot \frac{1}{b_1^2\, h_{\mathrm{w}} + b_1^3 + 0{,}5\, b_1\, b_2\, h_{\mathrm{w}}\, k_{\mathrm{f}}} \qquad \text{… (5.10b)}$$

where:

b_1 is the distance from the web-to-flange junction to the center of the effective area of the edge stiffener (including effective part b_{e2} of the flange) of flange 1, see figure 5.8(a);

b_2 is the distance from the web-to-flange junction to the center of the effective area of the edge stiffener (including effective part of the flange) of flange 2;

h_w is the web depth;

$k_f = 0$ if flange 2 is in tension (e.g. for beam in bending about the y-y axis);

$k_f = \frac{A_{eff\,2}}{A_{eff\,1}}$ if flange 2 is also in compression (e.g. for a beam in axial compression);

$k_f = 1$ for a symmetric section in compression.

A_{eff1} and A_{eff2} is the effective area of the edge stiffener (including effective part b_{e2} of the flange, see figure 5.8(b)) of flange 1 and flange 2 respectively.

(6) For an intermediate stiffener, as a conservative alternative the values of the rotational spring stiffnesses $C_{\theta,1}$ and $C_{\theta,2}$ may be taken as equal to zero, and the deflection δ may be obtained from:

$$\delta = \frac{u b_1^{\ 2} b_2^{\ 2}}{3(b_1 + b_2)} \cdot \frac{12(1-v^2)}{Et^3} \quad \text{... (5.11)}$$

(7) The reduction factor χ_d for the distortional buckling resistance (flexural buckling of a stiffener) should be obtained from the relative slenderness $\bar{\lambda}_d$ from:

$$\chi_d = 1{,}0 \qquad \text{if } \bar{\lambda}_d \le 0{,}65 \quad \text{... (5.12a)}$$

$$\chi_d = 1{,}47 - 0{,}723\bar{\lambda}_d \qquad \text{if } 0{,}65 < \bar{\lambda}_d < 1{,}38 \quad \text{... (5.12b)}$$

$$\chi_d = \frac{0{,}66}{\bar{\lambda}_d} \qquad \text{if } \bar{\lambda}_d \ge 1{,}38 \quad \text{... (5.12c)}$$

where:

$$\bar{\lambda}_d = \sqrt{f_{yb} / \sigma_{cr,s}} \quad \text{… (5.12d)}$$

where:

$\sigma_{cr,s}$ is the elastic critical stress for the stiffener(s) from 5.5.3.2, 5.5.3.3 or 5.5.3.4.

(8) Alternatively, the elastic critical buckling stress $\sigma_{cr,s}$ may be obtained from elastic first order buckling analysis using numerical methods (see 5.5.1(11)).

(9) In the case of a plane element with an edge and intermediate stiffener(s) in the absence of a more accurate method the effect of the intermediate stiffener(s) may be neglected.

5.5.3.2 Plane elements with edge stiffeners

(1) The following orocedure is applicable to an edge stiffener if the requirements in 5.2 are met and the angle between the stiffener and the plane element is between 45° and 135°.

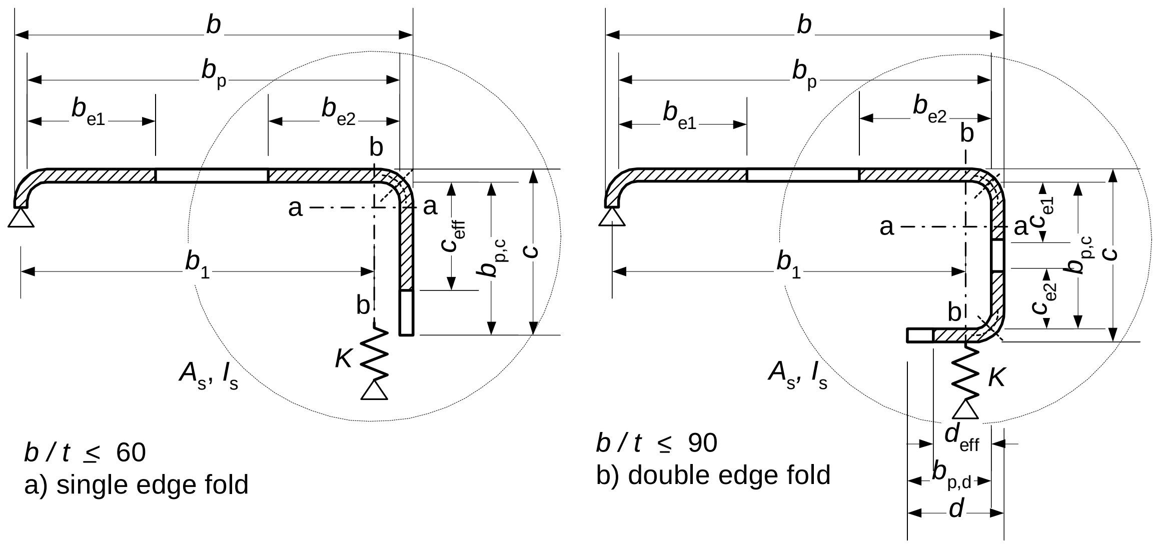


Figure 5.9: Edge stiffeners

(2) The cross-section of an edge stiffener should be taken as comprising the effective portions of the stiffener, element c or elements c and d as shown in figure 5.9, plus the adjacent effective portion of the plane element $b_{p.}$

(3) The procedure, which is illustrated in figure 5.10, should be carried out in steps as follows:

- **Step 1**: Obtain an initial effective cross-section for the stiffener using effective widths determined by assuming that the stiffener gives full restraint and that $\sigma_{com,Ed} = f_{yb} / \gamma_{M0}$, see (3) to (5);
- **Step 2**: Use the initial effective cross-section of the stiffener to determine the reduction factor for distortional buckling (flexural buckling of a stiffener), allowing for the effects of the continuous spring restraint, see (6) and (7);
- **Step 3**: Optionally iterate to refine the value of the reduction factor for buckling of the stiffener, see (8) and (9).

(4) Initial values of the effective widths b_{e1} and b_{e2} shown in figure 5.9 should be determined from clause 5.5.2 by assuming that the plane element b_p is doubly supported, see table 5.3.

(5) Initial values of the effective widths c_{eff} and d_{eff} shown in figure 5.9 should be obtained as follows:

a) for a single edge fold stiffener:

$$c_{eff} = \rho\, b_{p,c} \qquad \text{... (5.13a)}$$

with ρ obtained from 5.5.2, except using a value of the buckling factor k_σ given by the following:

- if $b_{p,c} / b_p \leq 0{,}35$:

$$k_\sigma = 0{,}5 \qquad \text{... (5.13b)}$$

- if $0{,}35 < b_{p,c} / b_p \leq 0{,}6$:

$$k_\sigma = 0{,}5 + 0{,}83\,\sqrt[3]{\left(b_{p,c} / b_p - 0{,}35\right)^2} \qquad \text{...(5.13c)}$$

b) for a double edge fold stiffener:

$$c_{eff} = \rho\, b_{p,c} \qquad \text{... (5.13d)}$$

with ρ obtained from 5.5.2 with a buckling factor k_σ for a doubly supported element from table 5.3;

$$d_{eff} = \rho\, b_{p,d} \qquad \text{... (5.13e)}$$

with ρ obtained from 5.5.2 with a buckling factor k_σ for an outstand element from table 5.4.

(6) The effective cross-sectional area of the edge stiffener A_s should be obtained from:

$$A_s = t\,(b_{e2} + c_{eff}) \qquad \text{or} \qquad \text{... (5.14a)}$$

$$A_s = t\,(b_{e2} + c_{e1} + c_{e2} + d_{eff}) \qquad \text{...(5.14b)}$$

respectively.

NOTE: The rounded corners should be taken into account if needed, see 5.1.

(7) The elastic critical buckling stress $\sigma_{cr,s}$ for an edge stiffener should be obtained from:

$$\sigma_{cr,s} = \frac{2\sqrt{K\,E\,I_s}}{A_s} \qquad \text{... (5.15)}$$

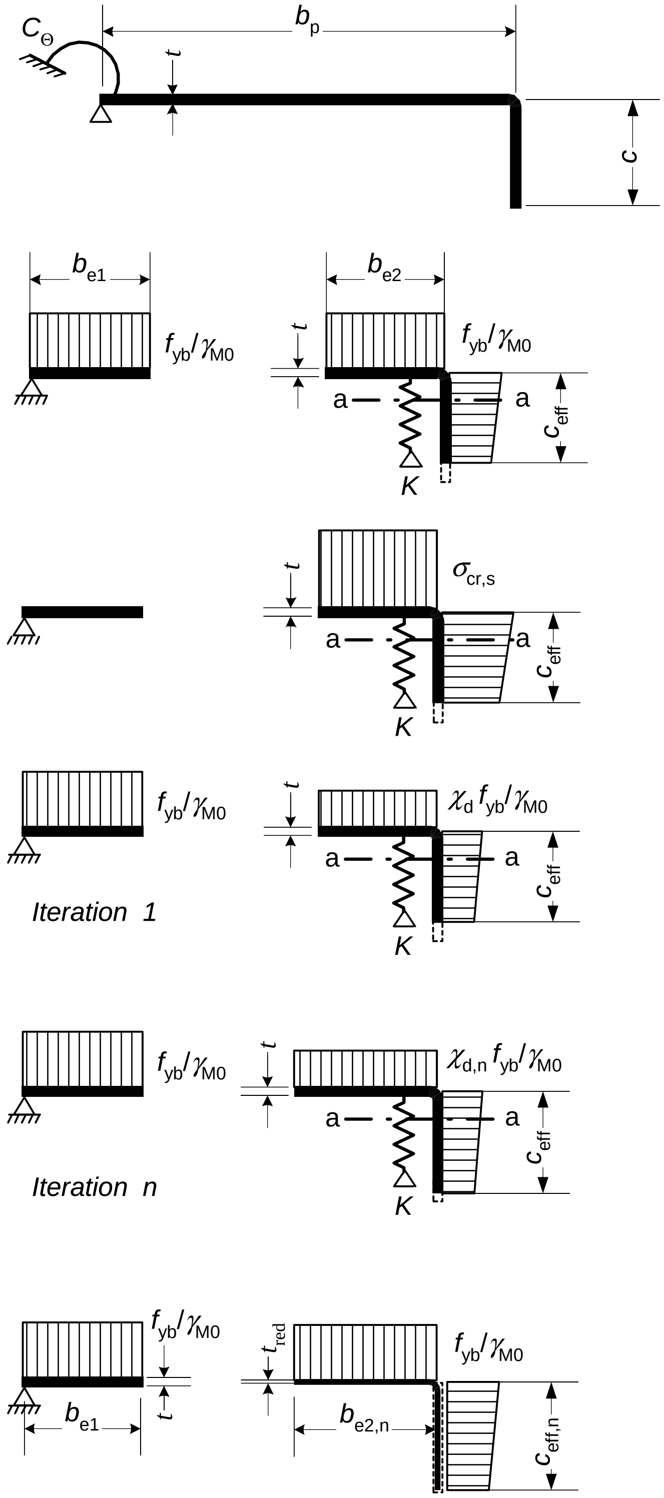
where:

K is the spring stiffness per unit length, see 5.5.3.1(2).

I_s is the effective second moment of area of the stiffener, taken as that of its effective area A_s about the centroidal axis a - a of its effective cross-section, see figure 5.9.

(8) Alternatively, the elastic critical buckling stress $\sigma_{cr,s}$ may be obtained from elastic first order buckling analyses using numerical methods, see 5.5.1(8).

(9) The reduction factor χ_d for the distortional buckling (flexural buckling of a stiffener) resistance of an edge stiffener should be obtained from the value of $\sigma_{cr,s}$ using the method given in 5.5.3.1(7).



a) Gross cross-section and boundary conditions

b) **Step 1**: Effective cross-section for $K = \infty$ based on $\sigma_{com,Ed} = f_{yb}/\gamma_{M0}$

c) **Step 2**: Elastic critical stress $\sigma_{cr,s}$ for effective area of stiffener A_s from step 1

d) Reduced strength $\chi_d\, f_{yb}/\gamma_{M0}$ for effective area of stiffener A_s, with reduction factor χ_d based on $\sigma_{cr,s}$

e) **Step 3**: Optionally repeat step 1 by calculating the effective width with a reduced compressive stress $\sigma_{com,Ed,i} = \chi_d\, f_{yb} / \gamma_{M0}$ with χ_d from previous iteration, continuing until $\chi_{d,n} \approx \chi_{d,(n-1)}$ but $\chi_{d,n} \leq \chi_{d,(n-1)}$.

f) Adopt an effective cross-section with b_{e2}, c_{eff} and reduced thickness t_{red} corresponding to $\chi_{d,n}$

Figure 5.10: Compression resistance of a flange with an edge stiffener

(10)If $\chi_d < 1$ it may be refined iteratively, starting the iteration with modified values of ρ obtained using 5.5.2(5) with $\sigma_{com,Ed,i}$ equal to $\chi_d\, f_{yb}/\gamma_{M0}$, so that:

$$\bar{\lambda}_{p,red} = \bar{\lambda}_p \sqrt{\chi_d} \qquad \text{... (5.16)}$$

(11)The reduced effective area of the stiffener $A_{s,red}$ allowing for flexural buckling should be taken as:

$$A_{s,red} = \chi_d A_s \frac{f_{yb}/\gamma_{M0}}{\sigma_{com,Ed}} \qquad \text{but } A_{s,red} \leq A_s \qquad \text{... (5.17)}$$

where

$\sigma_{com,Ed}$ is compressive stress at the centreline of the stiffener calculated on the basis of the effective cross-section.

(12)In determining effective section properties, the reduced effective area $A_{s,red}$ should be represented by using a reduced thickness $t_{red} = t\, A_{s,red} / A_s$ for all the elements included in A_s.

5.5.3.3 Plane elements with intermediate stiffeners

(1) The following procedure is applicable to one or two equal imtermediate stiffeners formed by grooves or bends provided that all plane elements are calculated accorrding to 5.5.2.

(2) The cross-section of an intermediate stiffener should be taken as comprising the stiffener itself plus the adjacent effective portions of the adjacent plane elements $b_{p,1}$ and $b_{p,2}$ shown in figure 5.11.

(3) The procedure, which is illustrated in figure 5.12, should be carried out in steps as follows:

- **Step 1**: Obtain an initial effective cross-section for the stiffener using effective widths determined by assuming that the stiffener gives full restraint and that $\sigma_{com,Ed} = f_{yb}/\gamma_{M0}$, see (3) and (4);

- **Step 2**: Use the initial effective cross-section of the stiffener to determine the reduction factor for distortional buckling (flexural buckling of an intermediate stiffener), allowing for the effects of the continuous spring restraint, see (5) and (6);

- **Step 3**: Optionally iterate to refine the value of the reduction factor for buckling of the stiffener, see (7) and (8).

(4) Initial values of the effective widths $b_{1,e2}$ and $b_{2,e1}$ shown in figure 5.11 should be determined from 5.5.2 by assuming that the plane elements $b_{p,1}$ and $b_{p,2}$ are doubly supported, see table 5.3.

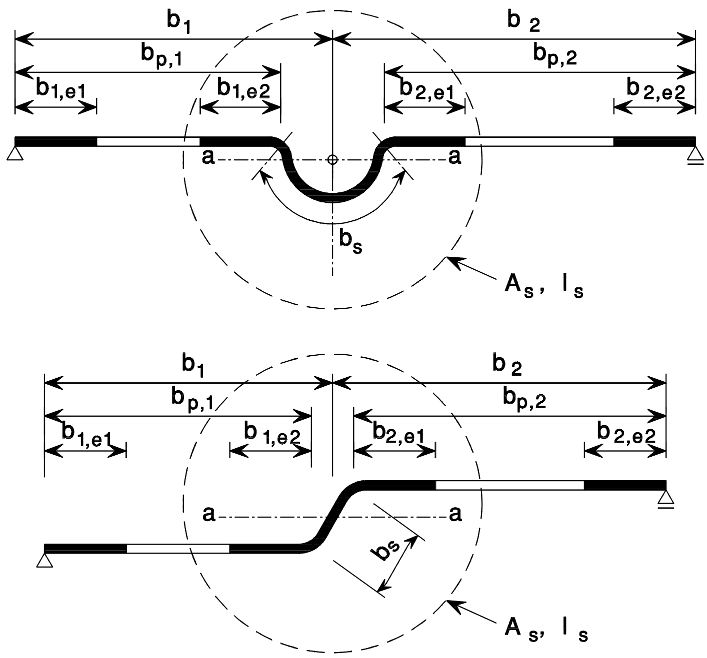


Figure 5.11: Intermediate stiffeners

(5) The effective cross-sectional area of an intermediate stiffener A_s should be obtained from:

$$A_s = t\,(b_{1,e2} + b_{2,e1} + b_s) \qquad \text{... (5.18)}$$

in which the stiffener width b_s is as shown in figure 5.11.

NOTE: The rounded corners should be taken into account if needed, see 5.1..

(6) The critical buckling stress $\sigma_{cr,s}$ for an intermediate stiffener should be obtained from:

$$\sigma_{cr,s} = \frac{2\sqrt{KEI_s}}{A_s} \qquad \text{... (5.19)}$$

where:

K is the spring stiffness per unit length, see 5.5.3.1(2).

I_s is the effective second moment of area of the stiffener, taken as that of its effective area A_s about the centroidal axis a - a of its effective cross-section, see figure 5.11.

(7) Alternatively, the elastic critical buckling stress $\sigma_{cr,s}$ may be obtained from elastic first order buckling analyses using numerical methods, see 5.5.1(11).

(8) The reduction factor χ_d for the distortional buckling resistance (flexural buckling of an intermediate stiffener) should be obtained from the value of $\sigma_{cr,s}$ using the method given in 5.5.3.1(7).

(9) If $\chi_d < 1$ it may optionally be refined iteratively, starting the iteration with modified values of ρ obtained using 5.5.2(5) with $\sigma_{com,Ed,i}$ equal to $\chi_d f_{yb} / \gamma_{M0}$, so that:

$$\bar{\lambda}_{p,red} = \bar{\lambda}_p \sqrt{\chi_d} \qquad \text{... (5.20)}$$

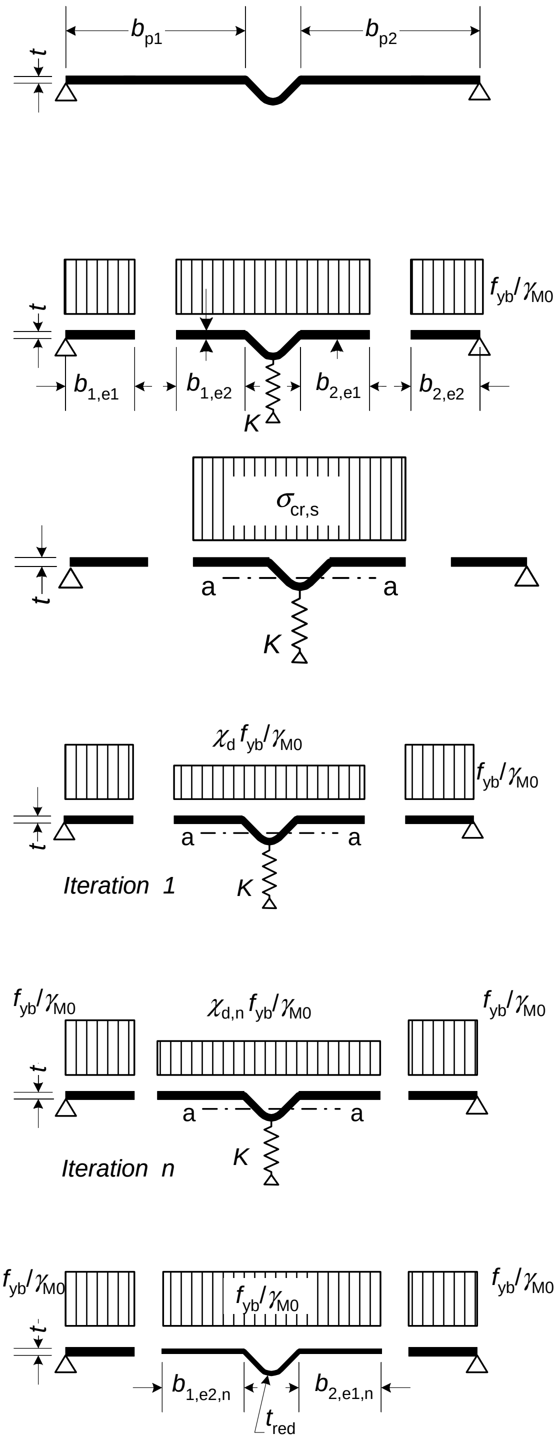
(10)The reduced effective area of the stiffener $A_{s,red}$ allowing for distortional buckling (flexural buckling of a stiffener) should be taken as:

$$A_{s,red} = \chi_d A_s \frac{f_{yb} / \gamma_{M0}}{\sigma_{com,Ed}} \quad \text{but } A_{s,red} \leq A_s \qquad \text{... (5.21)}$$

where

$\sigma_{com,Ed}$ is compressive stress at the centreline of the stiffener calculated on the basis of the effective cross-section.

(11)In determining effective section properties, the reduced effective area $A_{s,red}$ should be represented by using a reduced thickness $t_{red} = t A_{s,red} / A_s$ for all the elements included in A_s.



a) Gross cross-section and boundary conditions

b) **Step 1**: Effective cross-section for $K = \infty$ based on $\sigma_{com,Ed} = f_{yb} / \gamma_{M0}$

c) **Step 2**: Elastic critical stress $\sigma_{cr,s}$ for effective area of stiffener A_s from step 1

d) Reduced strength $\chi_d f_{yb} / \gamma_{M0}$ for effective area of stiffener A_s, with reduction factor χ_d based on $\sigma_{cr,s}$

e) **Step 3**: Optionally repeat step 1 by calculating the effective width with a reduced compressive stress $\sigma_{com,Ed,i} = \chi_d f_{yb} / \gamma_{M0}$ with χ_d from previous iteration, continuing until $\chi_{d,n} \approx \chi_{d,(n-1)}$ but $\chi_{d,n} \leq \chi_{d,(n-1)}$.

f) Adopt an effective cross-section with $b_{1,e2}$, $b_{2,e1}$ and reduced thickness t_{red} corresponding to $\chi_{d,n}$

Figure 5.12: Compression resistance of a flange with an intermediate stiffener

5.5.3.4 Trapezoidal sheeting profiles with intermediate stiffeners

5.5.3.4.1 General

(1) This sub-clause 5.5.3.4 should be used for trapezoidal profiled sheets, in association with 5.5.3.3 for flanges with intermediate flange stiffeners and 5.5.3.3 for webs with intermediate stiffeners.

(2) Interaction between the buckling of intermediate flange stiffeners and intermediate web stiffeners should also be taken into account using the method given in 5.5.3.4.4.

5.5.3.4.2 Flanges with intermediate stiffeners

(1) If it is subject to uniform compression, the effective cross-section of a flange with intermediate stiffeners should be assumed to consist of the reduced effective areas $A_{s,red}$ and two strips of width $0{,}5b_{eff}$ (or $15\,t$, see figure 5.13) adjacent to the stiffener.

(2) For one central flange stiffener, the elastic critical buckling stress $\sigma_{cr,s}$ should be obtained from:

$$\sigma_{cr,s} = \frac{4{,}2\,k_w E}{A_s}\sqrt{\frac{I_s\,t^3}{4\,b_p^2\,(2\,b_p + 3\,b_s)}} \qquad \text{... (5.22)}$$

where:

b_p is the notional flat width of plane element shown in figure 5.13;

b_s is the stiffener width, measured around the perimeter of the stiffener, see figure 5.13;

A_s, I_s are the cross-section area and the second moment of area of the stiffener cross-section according to figure 5.13;

k_w is a coefficient that allows for partial rotational restraint of the stiffened flange by the webs or other adjacent elements, see (5) and (6). For the calculation of the effective cross-section in axial compression the value $k_w = 1{,}0$.

The equation 5.22 may be used for wide grooves provided that flat part of the stiffener is reduced due to local buckling and b_p in the equation 5.22 is replaced by the larger of b_p and $0{,}25(3b_p+b_r)$, see figure 5.13. Similar method is valid for flange with two or more wide grooves.

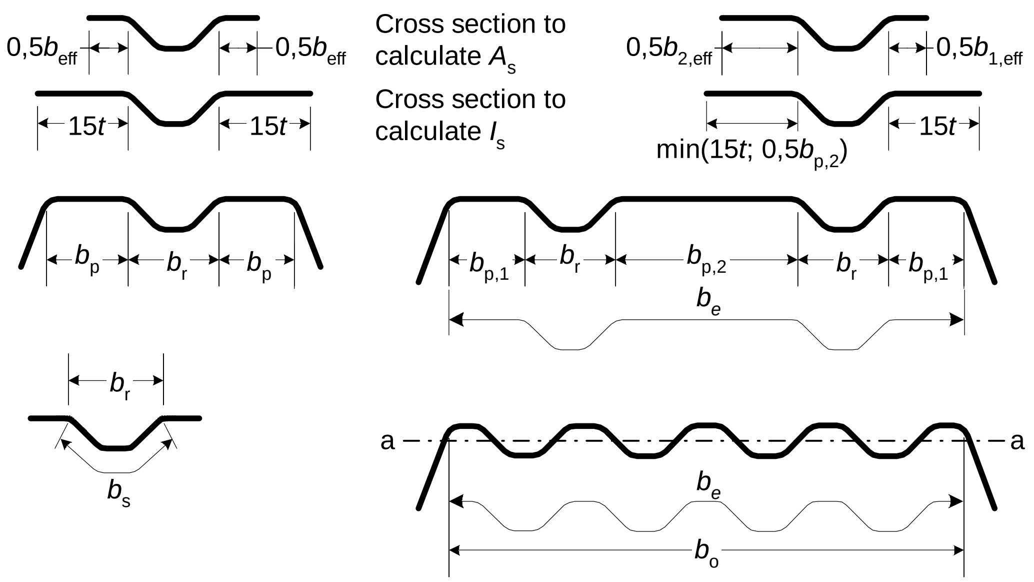


Figure 5.13: Compression flange with one, two or multiple stiffeners

(3) For two symmetrically placed flange stiffeners, the elastic critical buckling stress $\sigma_{cr,s}$ should be obtained from:

$$\sigma_{cr,s} = \frac{4{,}2\ k_w\ E}{A_s}\sqrt{\frac{I_s\ t^3}{8\ b_1^2\ (3\ b_e - 4\ b_1)}} \qquad \text{... (5.23a)}$$

with:

$b_e = 2b_{p,1} + b_{p,2} + 2b_s$

$b_1 = b_{p,1} + 0{,}5\,b_r$

where:

$b_{p,1}$ is the notional flat width of an outer plane element, as shown in figure 5.13;

$b_{p,2}$ is the notional flat width of the central plane element, as shown in figure 5.13;

b_r is the overall width of a stiffener, see figure 5.13;

A_s, I_s are the cross-section area and the second moment of area of the stiffener cross-section according to figure 5.13.

(4) For a multiple stiffened flange (three or more equal stiffeners) the effective area of the *entire flange* is

$$A_{eff} = \rho b_e t \qquad \text{…(5.23b)}$$

where ρ is the reduction factor according to expression (5.4b) for the slenderness $\overline{\lambda}_p$ based on the elastic buckling stress

$$\sigma_{cr,s} = 1{,}8E\sqrt{\frac{I_s\,t}{b_o^3 b_e^2}} + 3{,}6\frac{Et^2}{b_o^2} \qquad \text{…(5.23c)}$$

where:

I_s is the sum of the second moment of area of the stiffeners about the centroidal axis a-a, neglecting the thickness terms $bt^3/12$;

b_o is the width of the flange as shown in 5.13;

b_e is the developed width of the flange as shown in figure 5.13.

(5) The value of k_w may be calculated from the compression flange buckling wavelength l_b as follows:

- if $l_b / s_w \geq 2$:

$$k_w = k_{wo} \qquad \text{...(5.24a)}$$

- if $l_b / s_w < 2$:

$$k_w = k_{wo} - (k_{wo} - 1)\left[\frac{2l_b}{s_w} - \left(\frac{l_b}{s_w}\right)^2\right] \qquad \text{...(5.24b)}$$

where:

s_w is the slant height of the web, see figure 5.3(c).

(6) Alternatively, the rotational restraint coefficient k_w may conservatively be taken as equal to 1,0 corresponding to a pin-jointed condition.

(7) The values of l_b and k_{wo} may be determined from the following:

- for a compression flange with one intermediate stiffener:

$$l_b = 3{,}07 \sqrt[4]{\frac{I_s\, b_p^2 \,(2\, b_p + 3\, b_s)}{t^3}} \qquad \text{... (5.25)}$$

$$k_{wo} = \sqrt{\frac{s_w + 2\, b_d}{s_w + 0{,}5\, b_d}} \qquad \text{... (5.26)}$$

with:

$b_d = 2b_p + b_s$

- for a compression flange with two or three intermediate stiffeners:

$$l_b = 3{,}65 \sqrt[4]{I_s\, b_1^2 \,(3\, b_e - 4\, b_1)/t^3} \qquad \text{... (5.27)}$$

$$k_{wo} = \sqrt{\frac{(2\, b_e + s_w)(3\, b_e - 4\, b_1)}{b_1\,(4\, b_e - 6\, b_1) + s_w\,(3\, b_e - 4\, b_1)}} \qquad \text{... (5.28)}$$

(8) The reduced effective area of the stiffener $A_{s,red}$ allowing for distortional buckling (flexural buckling of an intermediate stiffener) should be taken as:

$$A_{s.red} = \chi_d\, A_s \frac{f_{yb}/\gamma_{M0}}{\sigma_{com,ser}} \quad \text{but } A_{s,red} \leq A_s \qquad \text{... (5.29)}$$

(9) If the webs are unstiffened, the reduction factor χ_d should be obtained directly from $\sigma_{cr,s}$ using the method given in 5.5.3.1(7).

(10)If the webs are also stiffened, the reduction factor χ_d should be obtained using the method given in 5.5.3.1(7), but with the modified elastic critical stress $\sigma_{cr,mod}$ given in 5.5.3.4.4.

(11)In determining effective section properties, the reduced effective area $A_{s,red}$ should be represented by using a reduced thickness $t_{red} = t\, A_{s,red} / A_s$ for all the elements included in A_s.

5.5.3.4.3 Webs with up to two intermediate stiffeners

(1) The effective cross-section of the compression zone of a web (or other element of a cross-section that is subject to stress gradient) should be assumed to consist of the reduced effective areas $A_{s,red}$ of up to two intermediate stiffeners, a strip adjacent to the compression flange and a strip adjacent to the centroidal axis of the effective cross-section, see figure 5.14.

(2) The effective cross-section of a web as shown in figure 5.14 should be taken to include:

a) a strip of width $s_{eff,1}$ adjacent to the compression flange;

b) the reduced effective area $A_{s,red}$ of each web stiffener, up to a maximum of two;

c) a strip of width $s_{eff,n}$ adjacent to the effective centroidal axis;

d) the part of the web in tension.

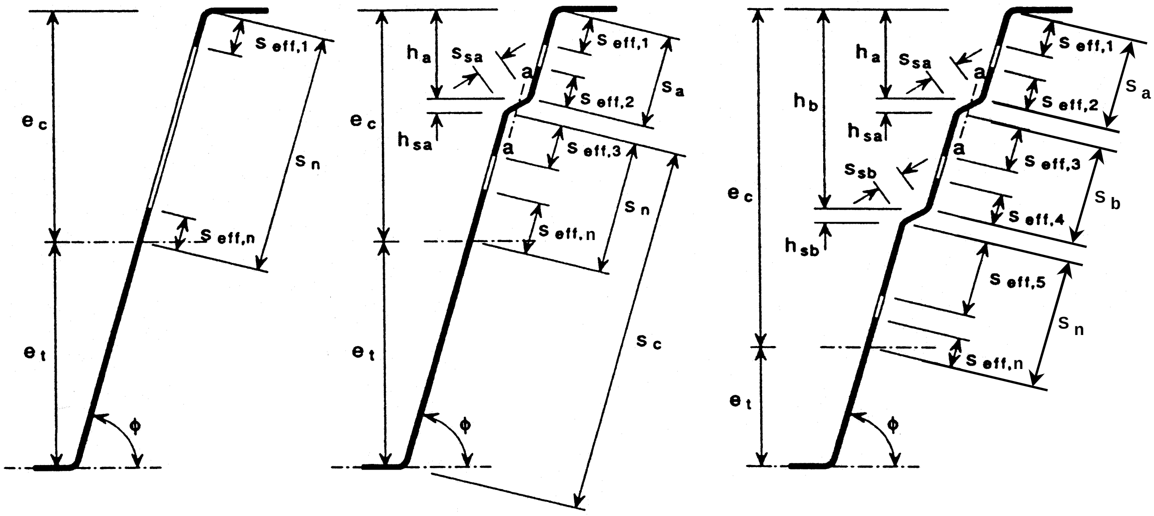


Figure 5.14: Effective cross-sections of webs of trapezoidal profiled sheets

(3) The effective areas of the stiffeners should be obtained from the following:

- for a single stiffener, or for the stiffener closer to the compression flange:

$$A_{sa} = t\,(s_{eff,2} + s_{eff,3} + s_{sa}) \qquad \text{... (5.30)}$$

- for a second stiffener:

$$A_{sb} = t\,(s_{eff,4} + s_{eff,5} + s_{sb}) \qquad \text{... (5.31)}$$

in which the dimensions $s_{eff,1}$ to $s_{eff,n}$ and s_{sa} and s_{sb} are as shown in figure 5.14.

(4) Initially the location of the effective centroidal axis should be based on the effective cross-sections of the flanges but the gross cross-sections of the webs. In this case the basic effective width $s_{eff,0}$ should be obtained from:

$$s_{eff,0} = 0{,}76\,t\,\sqrt{E\,/\left(\gamma_{M0}\,\sigma_{com,Ed}\right)} \qquad \text{... (5.32)}$$

where:

$\sigma_{com,Ed}$ is the stress in the compression flange when the cross-section resistance is reached.

(5) If the web is not fully effective, the dimensions $s_{eff,1}$ to $s_{eff,n}$ should be determined as follows:

$$s_{eff,1} = s_{eff,0} \qquad \text{... (5.33a)}$$

$$s_{eff,2} = (1 + 0{,}5 h_a / e_c)\, s_{eff,0} \qquad \text{... (5.33b)}$$

$$s_{eff,3} = [1 + 0{,}5(h_a + h_{sa})/ e_c]\, s_{eff,0} \qquad \text{... (5.33c)}$$

$$s_{eff,4} = (1 + 0{,}5 h_b / e_c)\, s_{eff,0} \qquad \text{... (5.33d)}$$

$$s_{eff,5} = [1 + 0{,}5(h_b + h_{sb})/ e_c]\, s_{eff,0} \qquad \text{... (5.33e)}$$

$$s_{eff,n} = 1{,}5 s_{eff,0} \qquad \text{... (5.33f)}$$

where:

e_c is the distance from the effective centroidal axis to the system line of the compression flange, see figure 5.14;

and the dimensions h_a, h_b, h_{sa} and h_{sb} are as shown in figure 5.14.

(6) The dimensions $s_{\mathrm{eff,1}}$ to $s_{\mathrm{eff,n}}$ should initially be determined from (5) and then revised if the relevant plane element is fully effective, using the following:

- in an unstiffened web, if $s_{\mathrm{eff,1}} + s_{\mathrm{eff,n}} \geq s_{\mathrm{n}}$ the entire web is effective, so revise as follows:

$$s_{\mathrm{eff,1}} = 0{,}4 s_{\mathrm{n}} \qquad \text{... (5.34a)}$$

$$s_{\mathrm{eff,n}} = 0{,}6 s_{\mathrm{n}} \qquad \text{... (5.34b)}$$

- in stiffened web, if $s_{\mathrm{eff,1}} + s_{\mathrm{eff,2}} \geq s_{\mathrm{a}}$ the whole of s_{a} is effective, so revise as follows:

$$s_{\mathrm{eff,1}} = \frac{s_{\mathrm{a}}}{2 + 0{,}5 h_{\mathrm{a}} / e_{\mathrm{c}}} \qquad \text{... (5.35a)}$$

$$s_{\mathrm{eff,2}} = s_{\mathrm{a}} \frac{(1 + 0{,}5 h_{\mathrm{a}} / e_{\mathrm{c}})}{2 + 0{,}5 h_{\mathrm{a}} / e_{\mathrm{c}}} \qquad \text{... (5.35b)}$$

- in a web with one stiffener, if $s_{\mathrm{eff,3}} + s_{\mathrm{eff,n}} \geq s_{\mathrm{n}}$ the whole of s_{n} is effective, so revise as follows:

$$s_{\mathrm{eff,3}} = s_{\mathrm{n}} \frac{[1 + 0{,}5 (h_{\mathrm{a}} + h_{\mathrm{sa}}) / e_{\mathrm{c}}]}{2{,}5 + 0{,}5 (h_{\mathrm{a}} + h_{\mathrm{sa}}) / e_{\mathrm{c}}} \qquad \text{... (5.36a)}$$

$$s_{\mathrm{eff,n}} = \frac{1{,}5 s_{\mathrm{n}}}{2{,}5 + 0{,}5 (h_{\mathrm{a}} + h_{\mathrm{sa}}) / e_{\mathrm{c}}} \qquad \text{... (5.36b)}$$

- in a web with two stiffeners:

  - if $s_{\mathrm{eff,3}} + s_{\mathrm{eff,4}} \geq s_{\mathrm{b}}$ the whole of s_{b} is effective, so revise as follows:

$$s_{\mathrm{eff,3}} = s_{\mathrm{b}} \frac{1 + 0{,}5 (h_{\mathrm{a}} + h_{\mathrm{sa}}) / e_{\mathrm{c}}}{2 + 0{,}5 (h_{\mathrm{a}} + h_{\mathrm{sa}} + h_{\mathrm{b}}) / e_{\mathrm{c}}} \qquad \text{... (5.37a)}$$

$$s_{\mathrm{eff,4}} = s_{\mathrm{b}} \frac{1 + 0{,}5 h_{\mathrm{b}} / e_{\mathrm{c}}}{2 + 0{,}5 (h_{\mathrm{a}} + h_{\mathrm{sa}} + h_{\mathrm{b}}) / e_{\mathrm{c}}} \qquad \text{... (5.37b)}$$

  - if $s_{\mathrm{eff,5}} + s_{\mathrm{eff,n}} \geq s_{\mathrm{n}}$ the whole of s_{n} is effective, so revise as follows:

$$s_{\mathrm{eff,5}} = s_{\mathrm{n}} \frac{1 + 0{,}5 (h_{\mathrm{b}} + h_{\mathrm{sb}}) / e_{\mathrm{c}}}{2{,}5 + 0{,}5 (h_{\mathrm{b}} + h_{\mathrm{sb}}) / e_{\mathrm{c}}} \qquad \text{... (5.38a)}$$

$$s_{\mathrm{eff,n}} = \frac{1{,}5 s_{\mathrm{n}}}{2{,}5 + 0{,}5 (h_{\mathrm{b}} + h_{\mathrm{sb}}) / e_{\mathrm{c}}} \qquad \text{... (5.38b)}$$

(7) For a single stiffener, or for the stiffener closer to the compression flange in webs with two stiffeners, the elastic critical buckling stress $\sigma_{cr,sa}$ should be determined using:

$$\sigma_{cr,sa} = \frac{1{,}05\ k_f\ E\sqrt{I_s\ t^3\ s_1}}{A_{sa}\ s_2\ (s_1 - s_2)} \qquad \text{... (5.39a)}$$

in which s_1 is given by the following:

- for a single stiffener:

$$s_1 = 0{,}9\,(s_a + s_{sa} + s_c) \qquad \text{... (5.39b)}$$

- for the stiffener closer to the compression flange, in webs with two stiffeners:

$$s_1 = s_a + s_{sa} + s_b + 0{,}5(s_{sb} + s_c) \qquad \text{... (5.39c)}$$

with:

$$s_2 = s_1 - s_a - 0{,}5 s_{sa} \qquad \text{... (5.39d)}$$

where:

k_f is a coefficient that allows for partial rotational restraint of the stiffened web by the flanges;

I_s is the second moment of area of a stiffener cross-section comprising the fold width s_{sa} and two adjacent strips, each of width $s_{eff,1}$, about its own centroidal axis parallel to the plane web elements, see figure 5.15. In calculating I_s the possible difference in slope between the plane web elements on either side of the stiffener may be neglected.

(8) In the absence of a more detailed investigation, the rotational restraint coefficient k_f may conservatively be taken as equal to 1,0 corresponding to a pin-jointed condition.

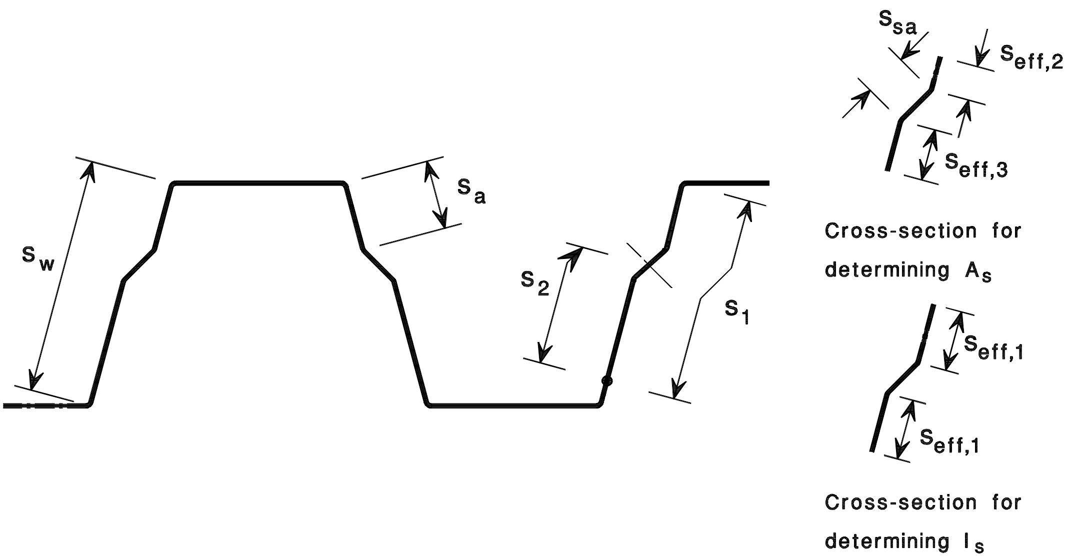


Figure 5.15: Web stiffeners for trapezoidal profiled sheeting

(9) For a single stiffener in compression, or for the stiffener closer to the compression flange in webs with two stiffeners, the reduced effective area $A_{sa,red}$ should be determined from:

$$A_{sa,red} = \frac{\chi_d\ A_{sa}}{1-(h_a + 0{,}5 h_{sa})/e_c} \quad \text{but } A_{sa,red} \le A_{sa} \qquad \text{... (5.40)}$$

(10)If the flanges are unstiffened, the reduction factor χ_d should be obtained directly from $\sigma_{cr,sa}$ using the method given in 5.5.3.1(7).

(11)If the flanges are also stiffened, the reduction factor χ_d should be obtained using the method given in 5.5.3.1(7), but with the modified elastic critical stress $\sigma_{cr,mod}$ given in 5.5.3.4.4.

(12)For a single stiffener in tension, the reduced effective area $A_{sa,red}$ should be taken as equal to A_{sa}.

(13)For webs with two stiffeners, the reduced effective area $A_{sb,red}$ for the second stiffener, should be taken as equal to A_{sb}.

(14)In determining effective section properties, the reduced effective area $A_{sa,red}$ should be represented by using a reduced thickness $t_{red} = \chi_d t$ for all the elements included in A_{sa}.

(15)The effective section properties *of the stiffeners* at serviceability limit states should be based on the design thickness t.

(16)Optionally, the effective section properties may be refined iteratively by basing the location of the effective centroidal axis on the effective cross-sections of the webs determined by the previous iteration and the effective cross-sections of the flanges determined using the reduced thickness t_{red} for all the elements included in the flange stiffener areas A_s. This iteration should be based on an increased basic effective width $s_{eff,0}$ obtained from:

$$s_{eff,0} = 0{,}95\, t \sqrt{\frac{E}{\gamma_{M0}\, \sigma_{com,Ed}}} \quad \text{... (5.41)}$$

5.5.3.4.4 Sheeting with flange stiffeners and web stiffeners

(1) In the case of sheeting with intermediate stiffeners in the flanges and in the webs, see figure 5.16, interaction between the distorsional buckling (flexural buckling of the flange stiffeners and the web stiffeners) should be allowed for by using a modified elastic critical stress $\sigma_{cr,mod}$ for both types of stiffeners, obtained from:

$$\sigma_{cr,mod} = \frac{\sigma_{cr,s}}{\sqrt[4]{1 + \left[\beta_s \dfrac{\sigma_{cr,s}}{\sigma_{cr,sa}}\right]^4}} \quad \text{... (5.42)}$$

where:

$\sigma_{cr,s}$ is the elastic critical stress for an intermediate flange stiffener, see 5.5.3.4.2(2) for a flange with a single stiffener or 5.5.3.4.2(3) for a flange with two stiffeners;

$\sigma_{cr,sa}$ is the elastic critical stress for a single web stiffener, or the stiffener closer to the compression flange in webs with two stiffeners, see 5.5.3.4.3(7);

A_s is the effective cross-section area of an intermediate flange stiffener;

A_{sa} is the effective cross-section area of an intermediate web stiffener;

$\beta_s = 1 - (h_a + 0{,}5\, h_{ha}) / e_c$ for a profile in bending;

$\beta_s = 1$ for a profile in axial compression.

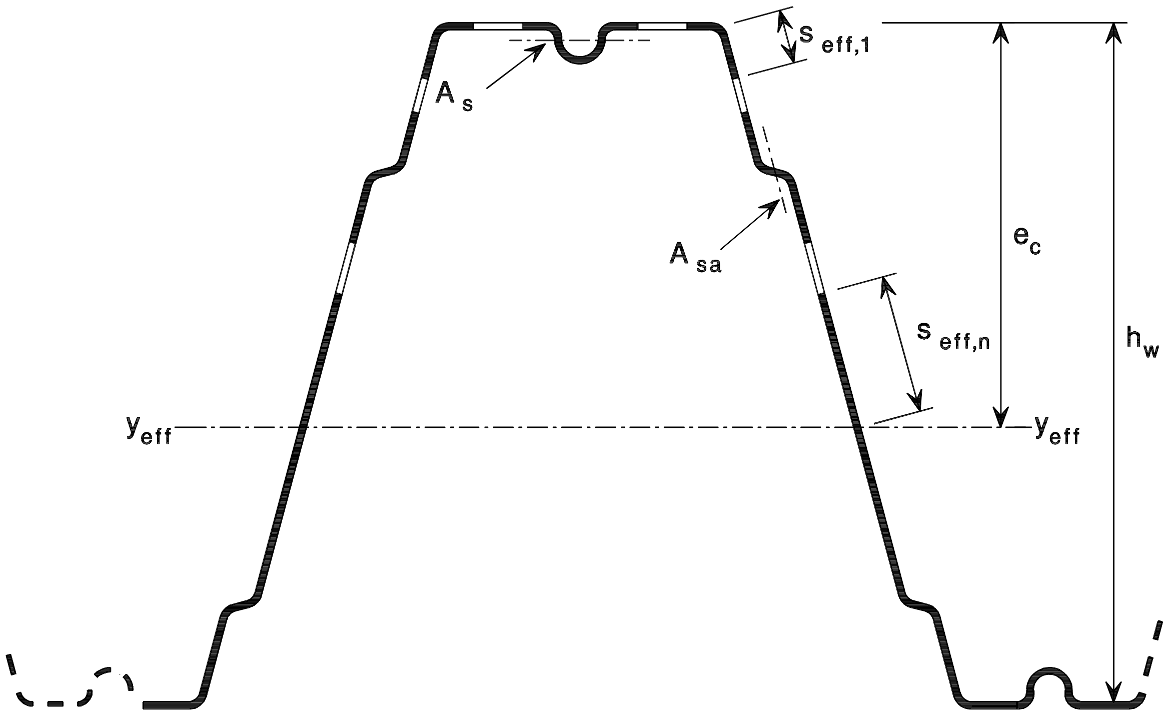


Figure 5.16: Trapezoidal profiled sheeting with flange stiffeners and web stiffeners

5.6 Buckling between fasteners

(1) Buckling between fasteners should be checked for composed elements of plates and mechanical fasteners, see Table 3.3 of EN 1993-1-8.

6 Ultimate limit states

6.1 Resistance of cross-sections

6.1.1 General

(1) Design assisted by testing may be used instead of design by calculation for any of these resistances.

NOTE: Design assisted by testing is particularly likely to be beneficial for cross-sections with relatively high b_p / t ratios, e.g. in relation to inelastic behaviour, web crippling or shear lag.

(2) For design by calculation, the effects of local buckling shall be taken into account by using effective section properties determined as specified in Section 5.5.

(3) The buckling resistance of members shall be verified as specified in Section 6.2.

(4) In members with cross-sections that are susceptible to cross-sectional distortion, account shall be taken of possible lateral buckling of compression flanges and lateral bending of flanges generally, see 5.5, and 10.1.

6.1.2 Axial tension

(1) The design resistance of a cross-section for uniform tension $N_{t,Rd}$ should be determined from:

$$N_{t,Rd} = \frac{f_{ya} A_g}{\gamma_{M0}} \quad \text{but} \quad N_{t,Rd} \leq F_{n,Rd} \qquad \text{... (6.1)}$$

where:

A_g is the gross area of the cross-section;

$F_{n,Rd}$ is the net-section resistance from 8.4 for the appropriate type of mechanical fastener;

f_{ya} is the average yield strength, see 3.2.3.

(2) The design resistance of an angle for uniform tension connected through one leg, or other types of section connected through outstands, should be determined as specified in EN 1993-1-1.

6.1.3 Axial compression

(1) The design resistance of a cross-section for compression $N_{c,Rd}$ should be determined from:

- if the effective area A_{eff} is less than the gross area A_g (section with reduction due to local and/or distortional buckling)

$$N_{c,Rd} = A_{eff} f_{yb} / \gamma_{M0} \qquad \ldots(6.2)$$

- if the effective area A_{eff} is equal to the gross area A_g (section with no reduction due to local or distortional buckling)

$$N_{c,Rd} = A_g \left(f_{yb} + (f_{ya} - f_{yb}) 4(1 - \lambda / \lambda_{el}) \right) / \gamma_{M0} \text{ but not more than } A_g f_{ya} / \gamma_{M0} \qquad \ldots(6.3)$$

where

A_{eff} is the effective area of the cross-section, obtained from Section 5.5 by assuming a uniform compressive stress equal to f_{yb} / γ_{M0};

f_{ya} is the average yield strength, see 3.2.2;

f_{yb} is the basic yield strength.;

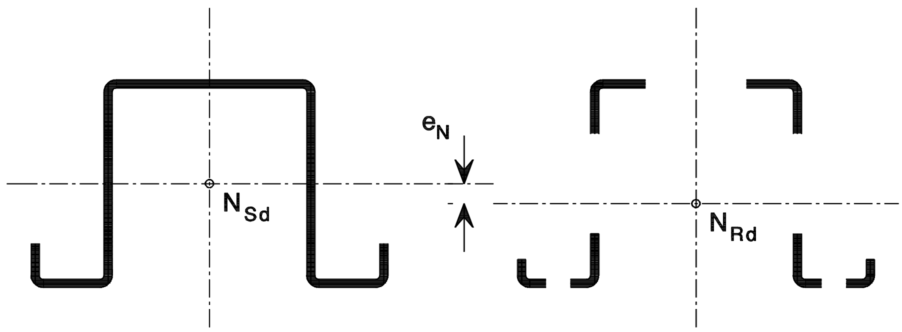
λ is the slenderness of the element which correspond to the largest value of λ / λ_{el};

For plane elements $\lambda = \overline{\lambda}_p$ and $\lambda_{el} = 0{,}673$, see 5.5.2;

For stiffened elements $\lambda = \overline{\lambda}_d$ and $\lambda_{el} = 0{,}65$, see 5.5.3.

(2) The internal normal force in a member should be taken as acting at the centroid of its gross cross-section. This is a conservative assumption, but may be used without further analysis. Further analysis may give a more realistic situation of the internal forces for instance in case of uniformly building-up of normal force in the compression element.

(3) The design compression resistance of a cross-section refers to the axial load acting in the centroid of its effective cross-section. If this does not coincide with the centroid of its gross cross-section, the shift e_N of the centroidal axes (see figure 6.1) should be taken into account, using the method given in 6.1.9. When the shift of the neutral axis gives a favourable result in the stress/unity check, then that shift should be neglected only if the shift has been calculated at yield strength and not with the actual compressive stresses.



6.1.4 Bending moment

6.1.4.1 Elastic and elastic-plastic resistance with yielding at the compressed flange

(1) The design moment resistance of a cross-section for bending about one principal axis $M_{c,Rd}$ is determined as follows (see figure 6.2):

- if the effective section modulus W_{eff} is less than the gross elastic section modulus W_{el}

$$M_{c,Rd} = W_{eff} f_{yb} / \gamma_{M0} \quad \text{...(6.4)}$$

- if the effective section modulus W_{eff} is equal to the gross elastic section modulus W_{el}

$$M_{c,Rd} = f_{yb}\left(W_{el} + (W_{pl} - W_{el})4(1 - \lambda / \lambda_{el})\right)/ \gamma_{M0} \text{ but not more than } W_{pl} f_{yb} / \gamma_{M0} \quad \text{...(6.5)}$$

where

λ is the slenderness of the element which correspond to the largest value of λ / λ_{el};

For double supported plane elements $\lambda = \bar{\lambda}_p$ and $\lambda_{el} = 0{,}5 + \sqrt{0{,}25 - 0{,}055(3+\psi)}$ where ψ is the stress ratio, see 5.5.2;

For outstand elements $\lambda = \bar{\lambda}_p$ and $\lambda_{el} = 0{,}673$, see 5.5.2;

For stiffened elements $\lambda = \bar{\lambda}_d$ and $\lambda_{el} = 0{,}65$, see 5.5.3.

The resulting bending moment resistance as a function of a decisive element is illustrated in the figure 6.2.

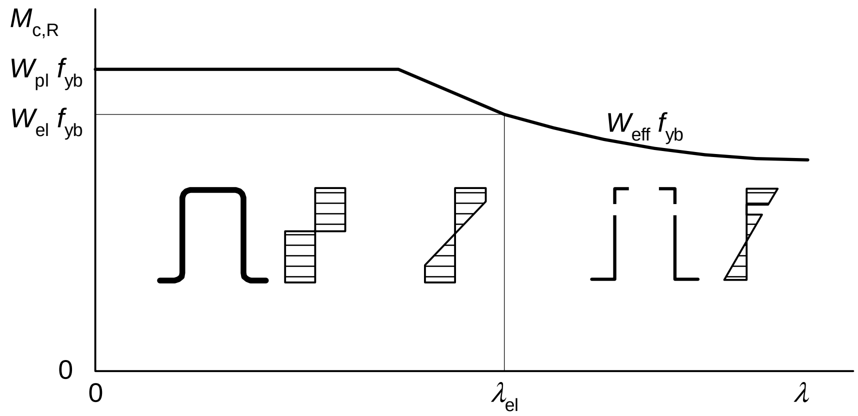


Figure 6.2: Bending moment resistance as a function of slenderness

(2) Expression (6.5) is applicable provided that the following conditions are satisfied:

a) Bending moment is applied only about one principal axes of the cross-section;

b) The member is not subject to torsion or to torsional, torsional flexural or lateral-torsional or distortional buckling;

c) The angle ϕ between the web (see figure 6.5) and the flange is larger than 60°.

(3) If (2) is not fulfilled the following expression may be used:

$$M_{c,Rd} = W_{el} f_{ya} / \gamma_{M0} \quad \text{…(6.6)}$$

(4) The effective section modulus W_{eff} should be based on an effective cross-section that is subject only to bending moment about the relevant principal axis, with a maximum stress $\sigma_{max,Ed}$ equal to f_{yb} / γ_{M0}, allowing for the effects of local and distortional buckling as specified in Section 5.5. Where shear lag is relevant, allowance should also be made for its effects.

(5) The stress ratio $\psi = \sigma_2 / \sigma_1$ used to determine the effective portions of the web may be obtained by using the effective area of the compression flange but the gross area of the web, see figure 6.3.

(6) If yielding occurs first at the compression edge of the cross-section, unless the conditions given in 6.1.4.2 are met the value of W_{eff} should be based on a linear distribution of stress across the cross-section.

(7) For biaxial bending the following criterion may be used:

$$\frac{M_{y,Ed}}{M_{cy,Rd}} + \frac{M_{z,Ed}}{M_{cz,Rd}} \leq 1 \qquad \text{... (6.7)}$$

where:

$M_{y,Ed}$ is the applied bending moment about the major main axis;

$M_{z,Ed}$ is the applied bending moment about the minor main axis;

$M_{cy,Rd}$ is the resistance of the cross-section if subject only to moment about the main y – y axis;

$M_{cz,Rd}$ is the resistance of the cross-section if subject only to moment about the main z – z axis.

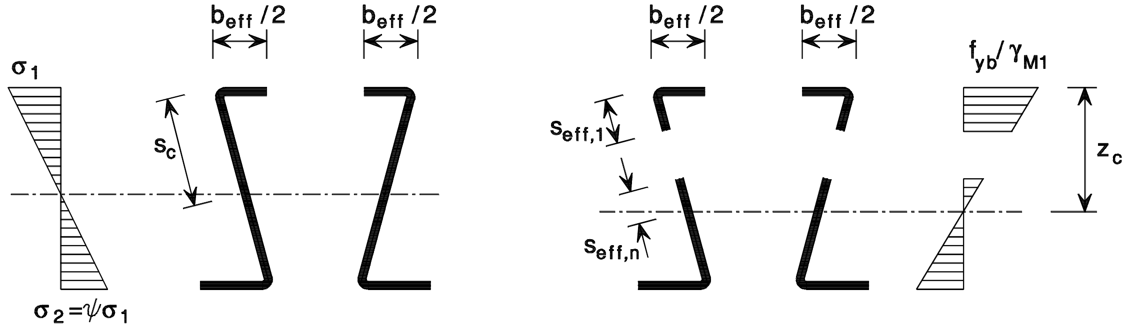


Figure 6.3: Effective cross-section for resistance to bending moments

(8) If redistribution of bending moments is assumed in the global analysis, it should be demonstrated from the results of tests in accordance with Section 9 that the provisions given in 7.2 are satisfied.

6.1.4.2 Elastic and elastic-plastic resistance with yielding at the tension flange only

(1) Provided that bending moment is applied only about one principal axis of the cross-section, and provided that yielding occurs first at the tension edge, plastic reserves in the tension zone may be utilised without any strain limit until the maximum compressive stress $\sigma_{com,Ed}$ reaches f_{yb} / γ_{M0}. In this clause only the bending case is considered. For axial load and bending the clause 6.1.8 or 6.1.9 should be applied.

(2) In this case, the effective partially plastic section modulus $W_{pp,eff}$ should be based on a stress distribution that is bilinear in the tension zone but linear in the compression zone.

(3) In the absence of a more detailed analysis, the effective width b_{eff} of an element subject to stress gradient may be obtained using 5.5.2 by basing b_c on the bilinear stress distribution (see figure 6.4), by assuming $\psi = -1$.

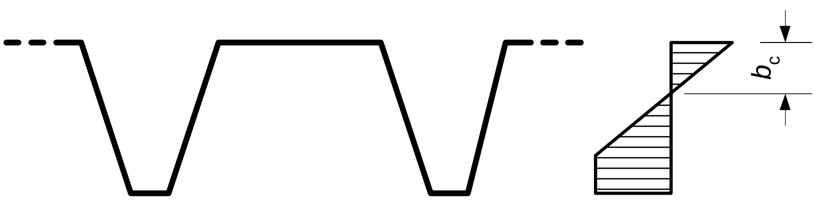


Figure 6.4: Measure b_c for determination of effective width

(4) If redistribution of bending moments is assumed in the global analysis, it should be demonstrated from the results of tests in accordance with Section 9 that the provisions given in 7.2 are satisfied.

6.1.4.3 Effects of shear lag

(1) The effects of shear lag shall be taken into account according to EN 1993-1-5.

6.1.5 Shear force

(1) The shear resistance $V_{\mathrm{b,Rd}}$ should be determined from:

$$V_{\mathrm{b,Rd}} = \frac{\dfrac{h_{\mathrm{w}}}{\sin \phi} t f_{\mathrm{bv}}}{\gamma_{\mathrm{M0}}} \qquad \text{... (6.8)}$$

where:

f_{bv} is the shear strength considering buckliong according to Table 6.1;

h_{w} is the web height between the midlines of the flanges, see figure 5.3(c);

ϕ is the slope of the web relative to the flanges, see figure 6.5.

Table 6.1: Shear buckling strength f_{bv}

Relative web slenderness	Web without stiffening at the support	Web with stiffening at the support [1)]
$\bar{\lambda}_{\mathrm{w}} \geq 0{,}83$	$0{,}58\ f_{\mathrm{yb}}$	$0{,}58\ f_{\mathrm{yb}}$
$0{,}83 < \bar{\lambda}_{\mathrm{w}} < 1{,}40$	$0{,}48 f_{\mathrm{yb}} / \bar{\lambda}_{\mathrm{w}}$	$0{,}48 f_{\mathrm{yb}} / \bar{\lambda}_{\mathrm{w}}$
$\bar{\lambda}_{\mathrm{w}} \geq 1{,}40$	$0{,}67 f_{\mathrm{yb}} / \bar{\lambda}_{\mathrm{w}}^2$	$0{,}48 f_{\mathrm{yb}} / \bar{\lambda}_{\mathrm{w}}$

[1)] Stiffening at the support, such as cleats, arranged to prevent distortion of the web and designed to resist the support reaction.

(2) The relative web slenderness $\bar{\lambda}_{\mathrm{w}}$ should be obtained from the following:

- for webs without longitudinal stiffeners:

$$\bar{\lambda}_{\mathrm{w}} = 0{,}346 \frac{s_{\mathrm{w}}}{t} \sqrt{\frac{f_{\mathrm{yb}}}{E}} \qquad \text{... (6.10a)}$$

- for webs with longitudinal stiffeners, see figure 6.5:

$$\bar{\lambda}_{\mathrm{w}} = 0{,}346 \frac{s_{\mathrm{d}}}{t} \sqrt{\frac{5{,}34}{k_{\tau}} \frac{f_{\mathrm{yb}}}{E}} \quad \text{but} \quad \bar{\lambda}_{w} \geq 0{,}346 \frac{s_{\mathrm{p}}}{t} \sqrt{\frac{f_{\mathrm{yb}}}{E}} \qquad \text{... (6.10b)}$$

with:

$$k_{\tau} = 5{,}34 + \frac{2{,}10}{t} \left(\frac{\Sigma I_{\mathrm{s}}}{s_{\mathrm{d}}} \right)^{1/3}$$

where:

I_{s} is the second moment of area of the individual longitudinal stiffener as defined in 5.5.3.4.3(7), about the axis a – a as indicated in figure 6.5;

s_{d} is the total developed slant height of the web, as indicated in figure 6.5;

s_{p} is the slant height of the largest plane element in the web, see figure 6.5;

s_{w} is the slant height of the web, as shown in figure 6.5, between the midpoints of the corners, these points are the median points of the corners, see figure 5.3(c).

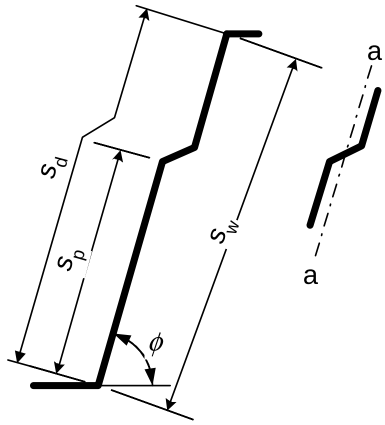


Figure 6.5: Longitudinally stiffened web

6.1.6 Torsional moment

(1) Where loads are applied eccentric to the shear centre of the cross-section, the effects of torsion shall be taken into account.

(2) The centroidal axis and shear centre and imposed rotation centre to be used in determining the effects of the torsional moment, should be taken as those of the gross cross-section.

(3) The direct stresses due to the axial force N_{Ed} and the bending moments $M_{y,Ed}$ and $M_{z,Ed}$ should be based on the respective effective cross-sections used in 6.1.2 to 6.1.4. The shear stresses due to transverse shear forces, the shear stress due to uniform (St. Venant) torsion and the direct stresses and shear stresses due to warping, should all be based on the properties of the gross cross-section.

(4) In cross-sections subject to torsion, the following conditions should be satisfied (average yield strength is allowed here, see 3.2.1(5)):

$$\sigma_{tot,Ed} \leq f_{ya} / \gamma_{M0} \quad \text{... (6.11a)}$$

$$\tau_{tot,Ed} \leq \frac{f_{ya} / \sqrt{3}}{\gamma_{M0}} \quad \text{... (6.11b)}$$

$$\sqrt{\sigma_{tot,Ed}^{2} + 3\tau_{tot,Ed}^{2}} \leq 1{,}1\frac{f_{ya}}{\gamma_{M0}} \quad \text{... (6.11c)}$$

where:

$\sigma_{tot,Ed}$ is the total direct stress, calculated on the relevant effective cross-section;

$\tau_{tot,Ed}$ is the total shear stress, calculated on the gross cross-section.

(5) The total direct stress $\sigma_{tot,Ed}$ and the total shear stress $\tau_{tot,Ed}$ should by obtained from:

$$\sigma_{tot,Ed} = \sigma_{N,Ed} + \sigma_{My,Ed} + \sigma_{Mz,Ed} + \sigma_{w,Ed} \quad \text{... (6.12a)}$$

$$\tau_{tot,Ed} = \tau_{Vy,Ed} + \tau_{Vz,Ed} + \tau_{t,Ed} + \tau_{w,Ed} \quad \text{... (6.12b)}$$

where:

$\sigma_{My,Ed}$ is the direct stress due to the bending moment $M_{y,Ed}$ (using effective cross-section);

$\sigma_{Mz,Ed}$ is the direct stress due to the bending moment $M_{z,Ed}$ (using effective cross-section);

$\sigma_{N,Ed}$ is the direct stress due to the axial force N_{Ed} (using effective cross-section);

$\sigma_{w,Ed}$ is the direct stress due to warping (using gross cross-section);

$\tau_{Vy,Ed}$ is the shear stress due to the transverse shear force $V_{y,Ed}$ (using gross cross-section);

$\tau_{Vz,Ed}$ is the shear stress due to the transverse shear force $V_{z,Ed}$ (using gross cross-section);

$\tau_{t,Ed}$ is the shear stress due to uniform (St. Venant) torsion (using gross cross-section);

$\tau_{w,Ed}$ is the shear stress due to warping (using gross cross-section).

6.1.7 Local transverse forces

6.1.7.1 General

(1) To avoid crushing, crippling or buckling in a web subject to a support reaction or other local transverse force applied through the flange, the transverse force F_{Ed} should satisfy:

$$F_{Ed} \leq R_{w,Rd} \quad \text{... (6.13)}$$

where:

$R_{w,Rd}$ is the local transverse resistance of the web.

(2) The local transverse resistance of a web $R_{w,Rd}$ should be obtained as follows:

a) for an unstiffened web:

- for a cross-section with a single web: from 6.1.7.2;
- for any other case, including sheeting: from 6.1.7.3;

b) for a stiffened web: from 6.1.7.4.

(3) Where the local load or support reaction is applied through a cleat that is arranged to prevent distortion of the web and is designed to resist the local transverse force, the local resistance of the web to the transverse force need not be considered.

(4) In beams with I-shaped cross-sections built up from two channels, or with similar cross-sections in which two components are interconnected through their webs, the connections between the webs should be located as close as practicable to the flanges of the beam.

6.1.7.2 Cross-sections with a single unstiffened web

(1) For a cross-section with a single unstiffened web, see figure 6.6, the local transverse resistance of the web may be determined as specified in (2), provided that the cross-section satisfies the following criteria:

$$h_w / t \leq 200 \quad \text{... (6.14a)}$$

$$r / t \leq 6 \quad \text{... (6.14b)}$$

$$45° \leq \phi \leq 90° \quad \text{... (6.14c)}$$

where:

h_w is the web height between the midlines of the flanges;

r is the internal radius of the corners;

ϕ is the slope of the web relative to the flanges [degrees].

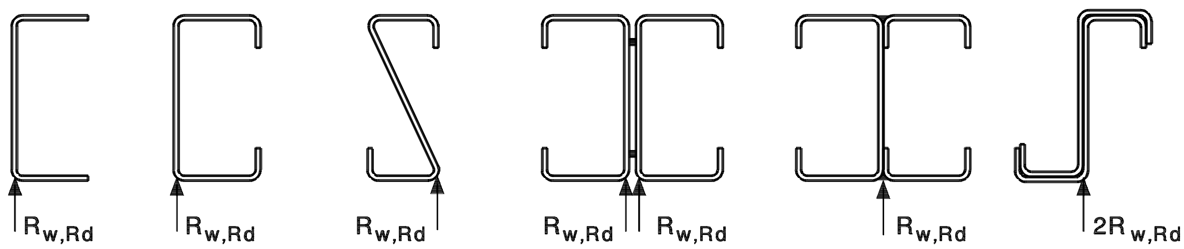


Figure 6.6: Examples of cross-sections with a single web

(2) For cross-sections that satisfy the criteria specified in (1), the local transverse resistance of a web $R_{w,Rd}$ may be determined as shown if figure 6.7.

(3) The values of the constants k_1 to k_5 should be determined as follows:

$k_1 = 1{,}33 - 0{,}33\,k$

$k_2 = 1{,}15 - 0{,}15\,r/t$ but $k_2 \geq 0{,}50$ and $k_2 \leq 1{,}0$

$k_3 = 0{,}7 + 0{,}3\,(\phi / 90)^2$

$k_4 = 1{,}22 - 0{,}22\,k$

$k_5 = 1{,}06 - 0{,}06\,r/t$ but $k_5 \leq 1{,}0$

where:

$k = f_{yb} / 228$ [with f_{yb} in N/mm^2];

s_s is the actual length of stiff bearing.

In the case of two equal and opposite local transverse forces distributed over unequal bearing lengths, the smaller value of s_s should be used.

(4) If the web rotation is prevented either by suitable restraint or because of the section geometry (e.g. I-beams, see fourth and fifth from the left in the figure 6.6) then the local transverse resistance of a web $R_{w,Rd}$ may be determined as follows:

a) for a single load or support reaction

i) $c < 1.5\,h_w$ (near or at free end)

for a cross-section of stiffened and unstiffened flanges

$$R_{w,Rd} = \frac{k_7 \left[8{,}8 + 1{,}1\sqrt{\dfrac{s_s}{t}} \right] t^2 f_{yb}}{\gamma_{M1}} \quad \text{… (6.16a)}$$

ii) $c > 1.5\,h_w$ (far from free end)

for a cross-section of stiffened and unstiffened flanges

$$R_{w,Rd} = \frac{k_5^* k_6 \left[13{,}2 + 2{,}87\sqrt{\dfrac{s_s}{t}} \right] t^2 f_{yb}}{\gamma_{M1}} \quad \text{… (6.16b)}$$

b) for opposite loads or reactions

i) $c < 1.5\,h_w$ (near or at free end)

for a cross-section of stiffened and unstiffened flanges

$$R_{w,Rd} = \frac{k_{10} k_{11} \left[8{,}8 + 1{,}1\sqrt{\dfrac{s_s}{t}} \right] t^2 f_{yb}}{\gamma_{M1}} \quad \text{… (6.16c)}$$

ii) $c > 1.5\,h_w$ (loads or reactions far from free end)

for a cross-section of stiffened and unstiffened flanges

$$R_{w,Rd} = \frac{k_8 k_9 \left[13{,}2 + 2{,}87\sqrt{\dfrac{s_s}{t}} \right] t^2 f_{yb}}{\gamma_{M1}} \quad \text{… (6.16d)}$$

Where the values of constants k_5^* to k_{11} should be determined as follows:

$k_5^* = 1{,}49 - 0{,}53\,k$ but $k_5^* \geq 0{,}6$

$k_6 = 0{,}88 - 0{,}12\,t / 1{,}9$

$k_7 = 1 + s_s / t / 750$ when $s_s / t < 150$; $k_7 = 1{,}20$ when $s_s / t > 150$

$k_8 = 1 / k$ when $s_s / t < 66{,}5$; $k_8 = (1{,}10 - s_s / t / 665) / k$ when $s_s / t > 66{,}5$

$k_9 = 0,82 + 0,15\, t\,/1,9$

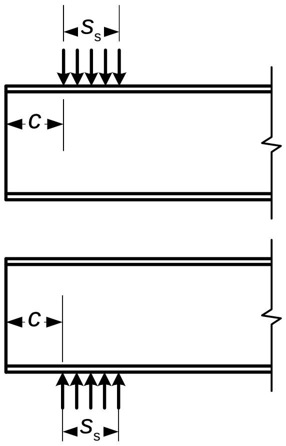
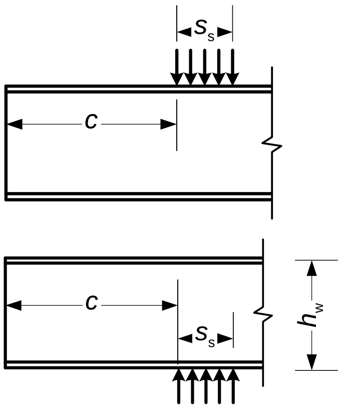
$k_{10} = (0,98 - s_s\, /\, t\, /\, 865)\, /\, k$

$k_{11} = 0,64 + 0,31\, t\,/1,9$

where:

$k = f_{yb}\,/\,228$ [with f_{yb} in N/mm^2];

s_s is the actual length of stiff bearing.

<table>
<tr>
<td>
</td>
<td>a) For a single local load or support reaction
i) $c \leq 1,5\, h_w$ clear from a free end:
- for a cross-section with stiffened flanges:
$$R_{w,Rd} = \frac{k_1 k_2 k_3 \left[9,04 - \frac{h_w/t}{60}\right]\left[1 + 0,01\frac{s_s}{t}\right] t^2 f_{yb}}{\gamma_{M1}} \qquad (6.15a)$$
- for a cross-section with unstiffened flanges:
- if $s_s/t \leq 60$:
$$R_{w,Rd} = \frac{k_1 k_2 k_3 \left[5,92 - \frac{h_w/t}{132}\right]\left[1 + 0,01\frac{s_s}{t}\right] t^2 f_{yb}}{\gamma_{M1}} \qquad (6.15b)$$
- if $s_s/t > 60$:
$$R_{w,Rd} = \frac{k_1 k_2 k_3 \left[5,92 - \frac{h_w/t}{132}\right]\left[0,71 + 0,015\frac{s_s}{t}\right] t^2 f_{yb}}{\gamma_{M1}} \qquad (6.15c)$$</td>
</tr>
<tr>
<td>
</td>
<td>ii) $c > 1,5\, h_w$ clear from a free end:
- if $s_s/t \leq 60$:
$$R_{w,Rd} = \frac{k_3 k_4 k_5 \left[14,7 - \frac{h_w/t}{49,5}\right]\left[1 + 0,007\frac{s_s}{t}\right] t^2 f_{yb}}{\gamma_{M1}} \qquad (6.15d)$$
- if $s_s/t > 60$:
$$R_{w,Rd} = \frac{k_3 k_4 k_5 \left[14,7 - \frac{h_w/t}{49,5}\right]\left[0,75 + 0,011\frac{s_s}{t}\right] t^2 f_{yb}}{\gamma_{M1}} \qquad (6.15e)$$</td>
</tr>
</table>

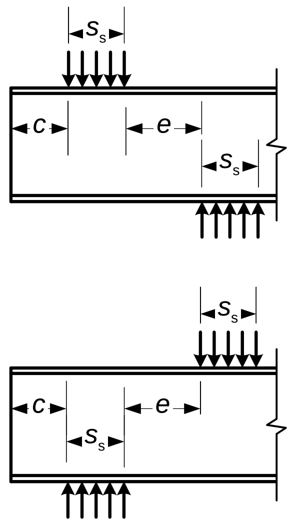
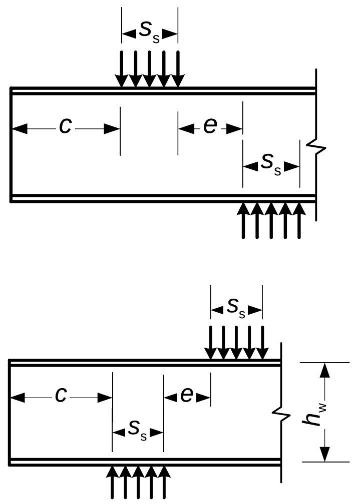
	b) For two opposing local transverse forces closer together than $1{,}5\ h_w$: i) $c \le 1{,}5\ h_w$ clear from a free end: $$R_{w,Rd} = \frac{k_1 k_2 k_3 \left[6{,}66 - \frac{h_w/t}{64}\right]\left[1 + 0{,}01\frac{s_s}{t}\right] t^2 f_{yb}}{\gamma_{M1}} \qquad (6.15f)$$
	ii) $c > 1{,}5\ h_w$ clear from a free end: $$R_{w,Rd} = \frac{k_3 k_4 k_5 \left[21{,}0 - \frac{h_w/t}{16{,}3}\right]\left[1 + 0{,}0013\frac{s_s}{t}\right] t^2 f_{yb}}{\gamma_{M1}} \qquad (6.15g)$$

Figure 6.7: Local loads and supports — cross-sections with a single web

6.1.7.3 Cross-sections with two or more unstiffened webs

(1) In cross-sections with two or more webs, including sheeting, see figure 6.8, the local transverse resistance of an unstiffened web should be determined as specified in (2), provided that both of the following conditions are satisfied:

- the clear distance c from the actual bearing length for the support reaction or local load to a free end, see figure 6.9, is at least 40 mm;
- the cross-section satisfies the following criteria:

$r/t \le 10$ ... (6.17a)

$h_w/t \le 200 \sin\phi$ ... (6.17b)

$45° \le \phi \le 90°$ ... (6.17c)

where:

h_w is the web height between the midlines of the flanges;

r is the internal radius of the corners;

ϕ is the slope of the web relative to the flanges [degrees].

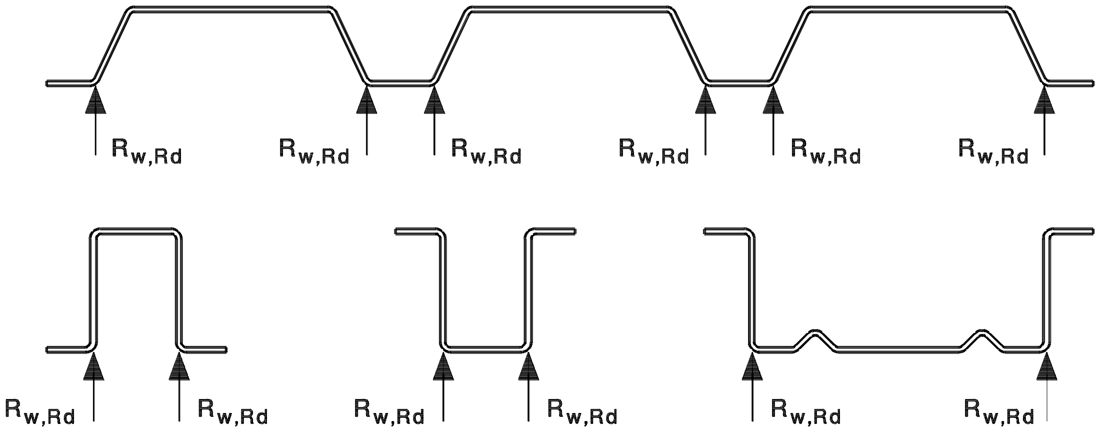


Figure 6.8: Examples of cross-sections with two or more webs

(2) Where both of the conditions specified in (1) are satisfied, the local transverse resistance $R_{w,Rd}$ per web of the cross-section should be determined from

$$R_{w,Rd} = \alpha t^2 \sqrt{f_{yb} E}\left(1 - 0{,}1\sqrt{r/t}\right)\left[0{,}5 + \sqrt{0{,}02\, l_a / t}\right]\left(2{,}4 + (\phi/90)^2\right)/\gamma_{M1} \quad \text{... (6.18)}$$

where:

l_a is the effective bearing length for the relevant category, see (3);

α is the coefficient for the relevant category, see (3).

(3) The values of l_a and α should be obtained from (4) and (5) respectively. The maximum design value for l_a = 200 mm. When the support is a cold-formed section with one web or round tube, for s_s should be taken a value of 10 mm. The relevant category (1 or 2) should be based on the clear distance e between the local load and the nearest support, or the clear distance c from the support reaction or local load to a free end, see figure 6.9.

(4) The value of the effective bearing length l_a should be obtained from the following:

a) for Category 1: $l_a = 10$ mm ... (6.19a)

b) for Category 2:

- $\beta_V \leq 0{,}2$: $l_a = s_s$ ... (6.19b)
- $\beta_V \geq 0{,}3$: $l_a = 10$ mm ... (6.19c)
- $0{,}2 < \beta_V < 0{,}3$: Interpolate linearly between the values of l_a for 0,2 and 0,3

with:

$$\beta_v = \frac{|V_{Ed,1}| - |V_{Ed,2}|}{|V_{Ed,1}| + |V_{Ed,2}|}$$

in which $|V_{Ed,1}|$ and $|V_{Ed,2}|$ are the absolute values of the transverse shear forces on each side of the local load or support reaction, and $|V_{Ed,1}| \geq |V_{Ed,2}|$ and s_s is the actual length of stiff bearing.

(5) The value of the coefficent α should be obtained from the following:

a) for Category 1:

- for sheeting profiles: $\alpha = 0{,}075$ ... (6.20a)
- for liner trays and hat sections: $\alpha = 0{,}057$ ... (6.20b)

b) for Category 2:

- for sheeting profiles: $\alpha = 0{,}15$ ... (6.20c)
- for liner trays and hat sections: $\alpha = 0{,}115$ ... (6.20d)

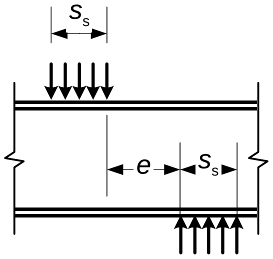
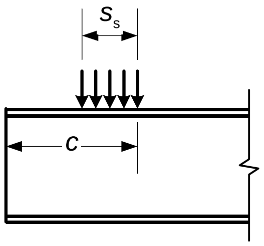
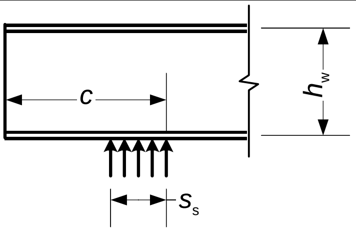
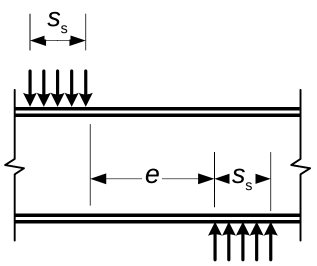
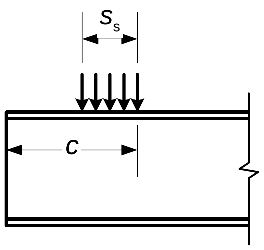
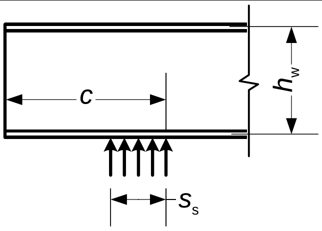
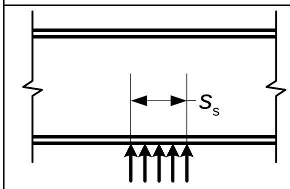
	Category 1 - local load applied with $e \le 1{,}5\ h_w$ clear from the nearest support;
	Category 1 - local load applied with $c \le 1{,}5\ h_w$ clear from a free end;
	Category 1 - reaction at end support with $c \le 1{,}5\ h_w$ clear from a free end.
	Category 2 - local load applied with $e > 1{,}5\ h_w$ clear from the nearest support;
	Category 2 - local load applied with $c > 1{,}5\ h_w$ clear from a free end;
	Category 2 - reaction at end support with $c > 1{,}5\ h_w$ clear from a free end;
	Category 2 - reaction at internal support.

Figure 6.9: Local loads and supports —categories of cross-sections with two or more webs

6.1.7.4 Stiffened webs

(1) The local transverse resistance of a stiffened web may be determined as specified in (2) for cross-sections with longitudinal web stiffeners folded in such a way that the two folds in the web are on opposite sides of the system line of the web joining the points of intersection of the midline of the web with the midlines of the flanges, see figure 6.10, that satisfy the condition:

$$2 < \frac{e_{\max}}{t} < 12 \qquad \text{... (6.21)}$$

where:

$e_{\max}$ is the larger eccentricity of the folds relative to the system line of the web.

(2) For cross-sections with stiffened webs satisfying the conditions specified in (1), the local transverse resistance of a stiffened web may be determined by multiplying the corresponding value for a similar unstiffened web, obtained from 6.1.7.2 or 6.1.7.3 as appropriate, by the factor $\kappa_{a,s}$ given by:

$$\kappa_{a,s} = 1,45 - 0,05\, e_{\max} / t \quad \text{but} \quad \kappa_{a,s} \leq 0,95 + 35\,000\, t^2\, e_{\min} / (\, b_d^{\,2}\, s_p\,) \qquad \text{... (6.22)}$$

where:

b_d is the developed width of the loaded flange, see figure 6.10;

$e_{\min}$ is the smaller eccentricity of the folds relative to the system line of the web;

s_p is the slant height of the plane web element nearest to the loaded flange, see figure 6.10.

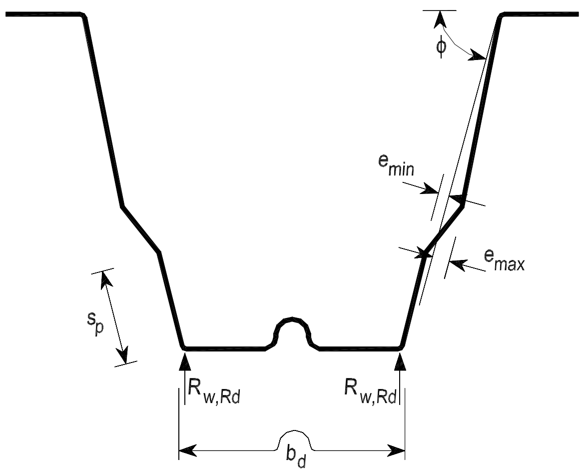


Figure 6.10: Stiffened webs

6.1.8 Combined tension and bending

(1) Cross-sections subject to combined axial tension N_{Ed} and bending moments $M_{y,Ed}$ and $M_{z,Ed}$ should satisfy the criterion:

$$\frac{N_{Ed}}{N_{t,Rd}} + \frac{M_{y,Ed}}{M_{cy,Rd,ten}} + \frac{M_{z,Ed}}{M_{cz,Rd,ten}} \leq 1 \qquad \text{... (6.23)}$$

where:

$N_{t,Rd}$ is the design resistance of a cross-section for uniform tension (6.1.2);

$M_{cy,Rd,ten}$ is the design moment resistance of a cross-section for maximum tensile stress if subject only to moment about the y - y axis (6.1.4);

$M_{cz,Rd,ten}$ is the design moment resistance of a cross-section for maximum tensile stress if subject only to moment about the z - z axis (6.1.4).

(2) If $M_{cy,Rd,com} \leq M_{cy,Rd,ten}$ or $M_{cz,Rd,com} \leq M_{cz,Rd,ten}$ (where $M_{cy,Rd,com}$ and $M_{cz,Rd,com}$ are the moment resistances for the maximum compressive stress in a cross-section that is subject only to moment about the relevant axis), the following criterion should also be satisfied:

$$\frac{M_{y,Ed}}{M_{cy,Rd.com}} + \frac{M_{z,Ed}}{M_{cz,Rd,com}} - \frac{N_{Ed}}{N_{t,Rd}} \leq 1 \qquad \text{... (6.24)}$$

6.1.9 Combined compression and bending

(1) Cross-sections subject to combined axial compression N_{Ed} and bending moments $M_{y,Ed}$ and $M_{z,Ed}$ should satisfy the criterion:

$$\frac{N_{Ed}}{N_{c,Rd}} + \frac{M_{y,Ed} + \Delta M_{y,Ed}}{M_{cy,Rd,com}} + \frac{M_{z,Ed} + \Delta M_{z,Ed}}{M_{cz,Rd,com}} \leq 1 \qquad \text{... (6.25)}$$

in which $N_{c,Rd}$ is as defined in 6.1.3, $M_{cy,Rd,com}$ and $M_{cz,Rd,com}$ are as defined in 6.1.8.

(2) The additional moments $\Delta M_{y,Ed}$ and $\Delta M_{z,Ed}$ due to shifts of the centroidal axes should be taken as:

$$\Delta M_{y,Ed} = N_{Ed}\, e_{Ny}$$

$$\Delta M_{z,Ed} = N_{Ed}\, e_{Nz}$$

in which e_{Ny} and e_{Nz} are the shifts of y-y and z-z centroidal axis due to axial forces, see 6.1.3(3).

(3) If $M_{cy,Rd,ten} \leq M_{cy,Rd,com}$ or $M_{cz,Rd,ten} \leq M_{cz,Rd,com}$ the following criterion should also be satisfied:

$$\frac{M_{y,Ed} + \Delta M_{y,Ed}}{M_{cy,Rd,ten}} + \frac{M_{z,Ed} + \Delta M_{z,Ed}}{M_{cz,Rd,ten}} - \frac{N_{Ed}}{N_{c,Rd}} \leq 1 \qquad \text{... (6.26)}$$

in which $M_{cy,Rd,ten}$, $M_{cz,Rd,ten}$ are as defined in 6.1.8.

6.1.10 Combined shear force, axial force and bending moment

(1) Cross-sections subject to the combined action of an axial force N_{Ed}, a bending moment M_{Ed} and a shear force V_{Ed} no reduction due to shear force need not be done provided that $V_{Ed} \leq 0{,}5\, V_{w,Rd}$. If the shear force is larger than half of the shear force resistance then following equations should be satisfied:

$$\frac{N_{Ed}}{N_{Rd}} + \frac{M_{y,Ed}}{M_{y,Rd}} + \left(1 - \frac{M_{f,Rd}}{M_{pl,Rd}}\right)\left(\frac{2V_{Ed}}{V_{w,Rd}} - 1\right)^2 \leq 1{,}0 \qquad \text{…(6.27)}$$

where:

- N_{Rd} is the design resistance of a cross-section for uniform tension or compression given in 6.1.2 or 6.1.3;
- $M_{y,Rd}$ is the design moment resistance of the cross-section given in 6.1.4;
- $V_{w,Rd}$ is the design shear resistance of the web given in 6.1.5(1);
- $M_{f,Rd}$ is the design plastic moment resistance of a cross-section consisting only flanges, see EN 1993-1-5;
- $M_{pl,Rd}$ is the plastic moment resistance of the cross-section, see EN 1993-1-5.

For members and sheeting with more than one web $V_{w,Rd}$ is the sum of the resistances of the webs. See also EN 1993-1-5.

6.1.11 Combined bending moment and local load or support reaction

(1) Cross-sections subject to the combined action of a bending moment M_{Ed} and a transverse force due to a local load or support reaction F_{Ed} should satisfy the following:

$$M_{Ed} / M_{c,Rd} \leq 1 \qquad \text{... (6.28a)}$$

$$F_{Ed} / R_{w,Rd} \leq 1 \qquad \text{... (6.28b)}$$

$$\frac{M_{Ed}}{M_{c,Rd}} + \frac{F_{Ed}}{R_{w,Rd}} \leq 1{,}25 \qquad \text{... (6.28c)}$$

where:

$M_{c,Rd}$ is the moment resistance of the cross-section given in 6.1.4.1(1);

$R_{w,Rd}$ is the appropriate value of the local transverse resistance of the web from 6.1.7.

In equation (6.2.8c) the bending moment M_{Ed} may be calculated at the edge of the support.

6.2 Buckling resistance

6.2.1 General

(1) In members with cross-sections that are susceptible to cross-sectional distortion, account shall be taken of possible lateral buckling of compression flanges and lateral bending of flanges generally, see 6.2.4.

(2) The effects of local and distortional buckling shall be taken into account as specified in Section 5.5.

6.2.2 Flexural buckling

(1) The design buckling resistance $N_{b,Rd}$ for flexural buckling should be obtained from EN 1993-1-1 using the appropriate buckling curve from table 6.3 according to the type of cross-section and axis of buckling.

(2) The buckling curve for a cross-section not included in table 6.3 may be obtained by analogy.

(3) The buckling resistance of a closed built-up cross-section should be determined using either:

- buckling curve b in association with the basic yield strength f_{yb} of the flat sheet material out of which the member is made by cold forming;
- buckling curve c in association with the average yield strength f_{ya} of the member after cold forming, determined as specified in 3.2.3, provided that $A_{eff} = A_g$.

6.2.3 Torsional buckling and torsional-flexural buckling

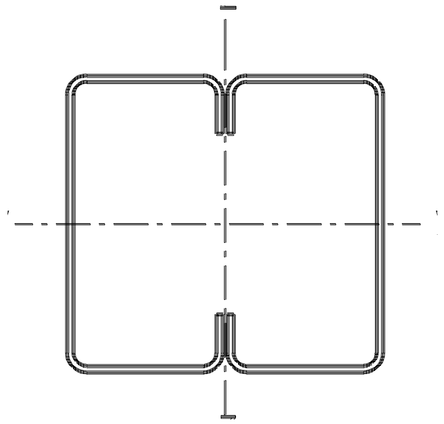
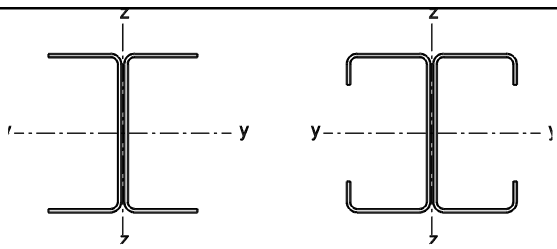
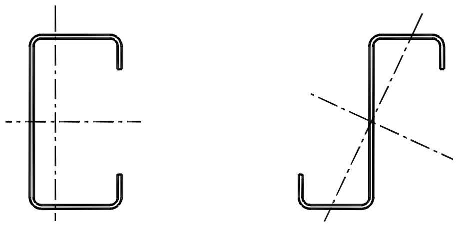
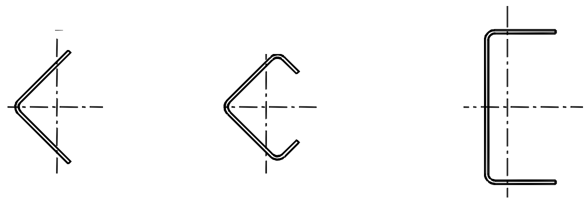
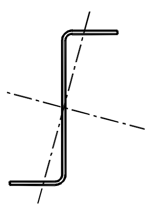
(1) For members with point-symmetric open cross-sections (e.g Z-purlin with equal flanges), account shall be taken of the possibility that the resistance of the member to torsional buckling might be less than its resistance to flexural buckling.

(2) For members with mono-symmetric open cross-sections, see figure 6.12, account shall be taken of the possibility that the resistance of the member to torsional-flexural buckling might be less than its resistance to flexural buckling.

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

(4) The design buckling resistance $N_{b,Rd}$ for torsional or torsional-flexural buckling should be obtained from 6.2.1 using the relevant buckling curve for buckling about the z-z axis obtained from table 6.3 .

Table 6.3: Appropriate buckling curve for various types of cross-section

Type of cross-section		Buckling about axis	Buckling curve
	if f_{yb} is used	Any	b
	if f_{ya} is used [*)]	Any	c
		y - y z - z	a b
		Any	b
  or other cross-section		Any	c

*) The average yield strength f_{ya} should not be used unless $A_{eff} = A_g$

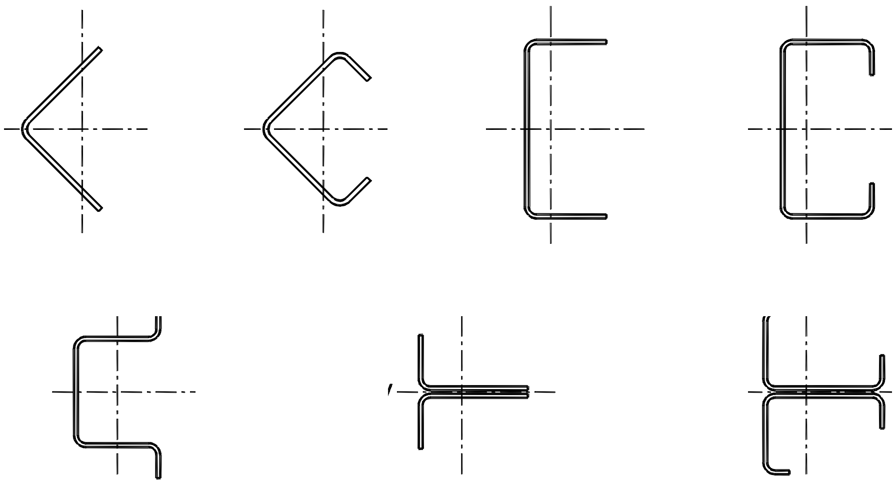


Figure 6.12: Cross-sections susceptible to torsional-flexural buckling

(5) The elastic critical force $N_{cr,T}$ for torsional buckling of simply supported beam should be determined from:

$$N_{cr,T} = \frac{1}{i_o^{\ 2}}\left(G\,I_t + \frac{\pi^2 E\,I_w}{l_T^2} \right) \qquad \text{... (6.33a)}$$

with:

$$i_o^{\ 2} = i_y^{\ 2} + i_z^{\ 2} + y_o^{\ 2} + z_o^{\ 2} \qquad \text{... (6.33b)}$$

where:

G is the shear modulus;

I_t is the torsion constant of the gross cross-section;

I_w is the warping constant of the gross cross-section;

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

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

l_T is the buckling length of the member for torsional buckling;

y_o, z_o are the shear centre co-ordinates with respect to the centroid of the gross cross-section.

(6) For doubly symmetric cross-sections (e.g. $y_o = z_o = 0$)

$$N_{cr,TF} = N_{cr,T} \qquad \text{...(6.34)}$$

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

(7) For cross-sections that are symmetrical about the y - y axis (e.g. $z_o = 0$), the elastic critical force $N_{cr,TF}$ for torsional-flexural buckling should be determined from:

$$N_{cr,TF} = \frac{N_{cr,y}}{2\beta}\left[1 + \frac{N_{cr,T}}{N_{cr,y}} - \sqrt{\left(1 - \frac{N_{cr,T}}{N_{cr,y}}\right)^2 + 4\left(\frac{y_o}{i_o}\right)^2 \frac{N_{cr,T}}{N_{cr,y}}} \right] \qquad \text{... (6.35)}$$

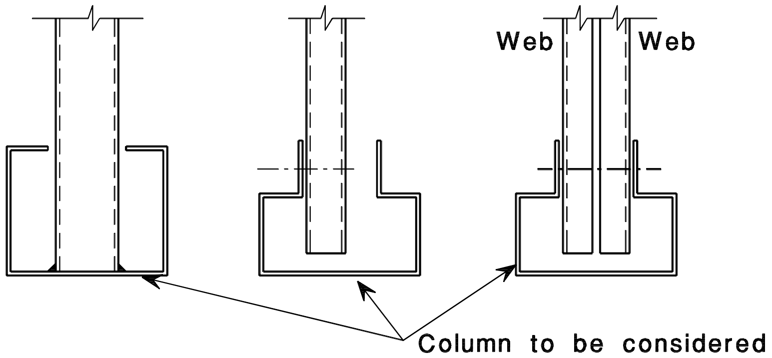
with:

$$\beta = 1 - \left(\frac{y_o}{i_o} \right)^2 .$$

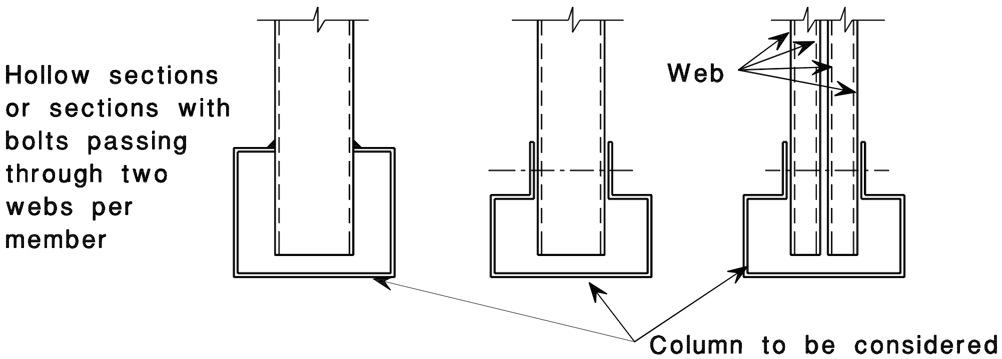
(8) The buckling length l_T for torsional or torsional-flexural buckling should be determined taking into account the degree of torsional and warping restraint at each end of the system length L_T.

(9) For practical connections at each end, the value of l_T / L_T may be taken as follows:

- 1,0 for connections that provide partial restraint against torsion and warping, see figure 6.13(a);
- 0,7 for connections that provide significant restraint against torsion and warping, see figure 6.13(b).



a) connections capable of giving partial torsional and warping restraint



b) connections capable of giving significant torsional and warping restraint

Figure 6.13: Torsional and warping restraint from practical connections

6.2.4 Lateral-torsional buckling of members subject to bending

(1) The design buckling resistance moment of a member that is susceptible to lateral-torsional buckling should be determined according to EN 1993-1-1, section 6.3.4 using lateral buckling curve a with $\alpha_{LT} = 0{,}21$.

(2) This method should not be used for the sections that have a significant angle between the principal axes of the effective cross-section, compared to those of the gross cross-section.

6.2.5 Bending and axial compression

(1) The interaction between axial force and bending moment may be obtained from a second-order analysis of the member as specified in EN 1993-1-1, based on the properties of the effective cross-section obtained from Section 5.5. See also 5.3.

(2) As an alternative the interaction formula (6.38) may be used

$$\left(\frac{N_{Ed}}{N_{b,Rd}}\right)^{0,8} + \left(\frac{M_{Ed}}{M_{b,Rd}}\right)^{0,8} \le 1{,}0 \qquad \ldots(6.38)$$

where $N_{b,Rd}$ is the design buckling resistance of a compression member according to 6.2.2 (flexural, torsional or torsional-flexural buckling) and $M_{b,Rd}$ is the design bending moment resistance according to 6.2.3.

6.3 Bending and axial tension

(1) The interaction equations for compressive force in 6.2 are applicable.

7 Serviceability limit states

7.1 General

(1) The rules for serviceability limit states given in Section 7 of EN 1993-1-1 shall also be applied to cold-formed thin gauge members and sheeting.

(2) The properties of the effective cross-section for serviceability limit states obtained from Section 5.1 should be used in all serviceability limit state calculations for cold-formed thin gauge members and sheeting.

(3) The second moment of area may be calculated alternatively by interpolation of gross cross-section and effective cross-section using the expression

$$I_{\mathrm{fic}} = I_{\mathrm{gr}} - \frac{\sigma_{\mathrm{gr}}}{\sigma}\left(I_{\mathrm{gr}} - I(\sigma)_{\mathrm{eff}}\right) \qquad \ldots (7.1)$$

where

I_{gr} is second moment of area of the gross cross-section;

σ_{gr} is maximum compressive bending stress in the serviceability limit state, based on the gross cross-section (positive in formula);

$I(\sigma)_{eff}$ is the second moment of area of the effective cross-section with allowance for local buckling calculated for a maximum stress $\sigma \geq \sigma_{gr}$, in which the maximum stress is the largest absolute value of stresses within the calculation length considered.

(4) The effective second moment of area I_{eff} (or I_{fic}) may be taken as variable along the span according to most severe locations. Alternatively a uniform value may be used, based on the maximum absolute span moment due to serviceability loading.

7.2 Plastic deformation

(1) In case of plastic global analysis the combination of support moment and support reaction at an internal support should not exceed 0,9 times the combined design resistance, determined using $\gamma_{M,ser}$.

(2) The combined design resistance may be determined from 6.1.11, but using the effective cross-section for serviceability limit states and $\gamma_{M,ser}$.

7.3 Deflections

(1) The deflections may be calculated assuming elastic behaviour.

(2) The influence of slip in the connections (for example in the case of continuous beam systems with sleeves and overlaps) should be considered in the calculation of deflections, forces and moments.

8 Design of joints

8.1 General

(1) For design assumptions and requirements of joints see EN 1993-1-8.

8.2 Splices and end connections of members subject to compression

(1) Splices and end connections in members that are subject to compression, shall either have at least the same resistance as the cross-section of the member, or be designed to resist an additional bending moment due to the second-order effects within the member, in addition to the internal compressive force N_{Ed} and the internal moments $M_{y,Ed}$ and $M_{z,Ed}$ obtained from the global analysis.

(2) In the absence of a second-order analysis of the member, this additional moment ΔM_{Ed} should be taken as acting about the cross-sectional axis that gives the smallest value of the reduction factor χ for flexural buckling, see 6.2.2.1(2), with a value determined from:

$$\Delta M_{Ed} = N_{Ed} \left(\frac{1}{\chi} - 1 \right) \frac{W_{eff}}{A_{eff}} \sin \frac{\pi a}{l} \qquad \text{…(8.1a)}$$

where:

A_{eff} is the effective area of the cross-section;

a is the distance from the splice or end connection to the nearer point of contraflexure;

l is the buckling length of the member between points of contraflexure, for buckling about the relevant axis;

W_{eff} is the section modulus of the effective cross-section for bending about the relevant axis.

Splices and end connections should be designed to resist an additional internal shear force

$$\Delta V_{Ed} = \frac{\pi N_{Ed}}{l} \left(\frac{1}{\chi} - 1 \right) \frac{W_{eff}}{A_{eff}} \qquad \text{…(8.1b)}$$

(3) Splices and end connections should be designed in such a way that load may be transmitted to the effective portions of the cross-section.

(4) If the constructional details at the ends of a member are such that the line of action of the internal axial force cannot be clearly identified, a suitable eccentricity should be assumed and the resulting moments should be taken into account in the design of the member, the end connections and the splice, if there is one.

8.3 Connections with mechanical fasteners

(1) Connections with mechanical fasteners shall be compact in shape. The positions of the fasteners shall be arranged to provide sufficient room for satisfactory assembly and maintenance.

NOTE More information see Part 1-8 of EN 1993.

(2) The shear forces on individual mechanical fasteners in a connection may be assumed to be equal, provided that:

- the fasteners have sufficient ductility;
- shear is not the critical failure mode.

(3) For design the resistances of mechanical fasteners subject to predominantly static loads should be determined from:

- table 8.1 for blind rivets;
- table 8.2 for self-tapping screws;
- table 8.3 for cartridge fired pins;
- table 8.4 for bolts.

(4) In tables 8.1 to 8.4 the meanings of the symbols shall be taken as follows:

A is the gross cross-sectional area of a bolt;

A_s is the tensile stress area of a bolt;

A_{net} is the net cross-sectional area of the connected part;

β_{Lf} is the reduction factor for long joints according to EN 1993-1-8;

d is the nominal diameter of the fastener;

d_o is the nominal diameter of the hole;

d_w is the diameter of the washer or the head of the fastener;

e_1 is the end distance from the centre of the fastener to the adjacent end of the connected part, in the direction of load transfer, see figure 8.1;

e_2 is the edge distance from the centre of the fastener to the adjacent edge of the connected part, in the direction perpendicular to the direction of load transfer, see figure 8.1;

f_{ub} is the ultimate tensile strength of the bolt material;

$f_{u,sup}$ is the ultimate tensile strength of the supporting member into which a screw is fixed;

n is the number of sheets that are fixed to the supporting member by the same screw or pin;

n_f is the number of mechanical fasteners in one connection;

p_1 is the spacing centre-to-centre of fasteners in the direction of load transfer, see figure 8.1;

p_2 is the spacing centre-to-centre of fasteners in the direction perpendicular to the direction of load transfer, see figure 8.1;

t is the thickness of the thinner connected part or sheet;

t_1 is the thickness of the thicker connected part or sheet;

t_{sup} is the thickness of the supporting member into which a screw or a pin is fixed.

(5) The partial factor γ_M for calculating the design resistances of mechanical fasteners shall be taken as γ_{M2}:

NOTE The value γ_{M2} may be given in the National Annex. The value $\gamma_{M2} = 1,25$ is recommended.

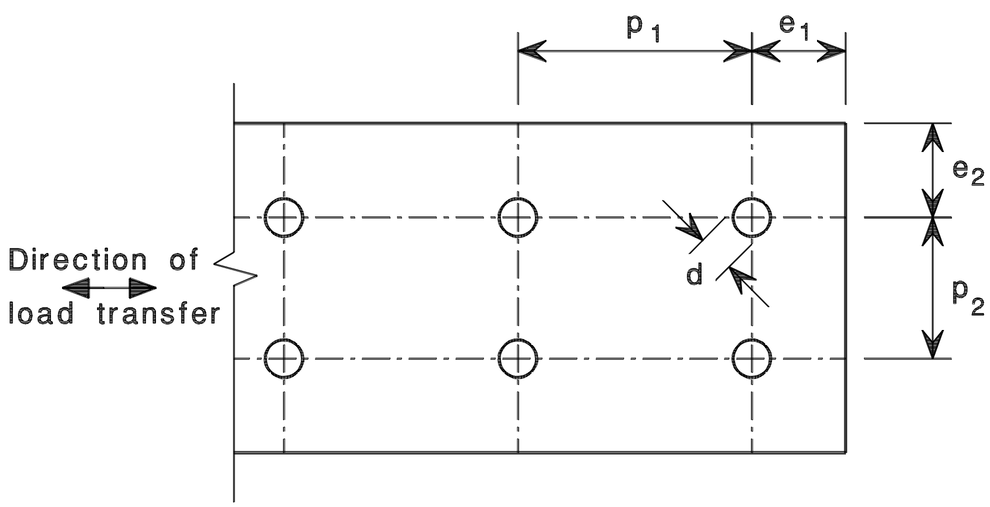


Figure 8.1: End distance, edge distance and spacings for fasteners and spot welds

(6) If the pull-out resistance $F_{o,Rd}$ of a fastener is smaller than its pull-through resistance $F_{p,Rd}$ the deformation capacity should be determined from tests.

(7) The pull-through resistances given in tables 8.2 and 8.3 for self-tapping screws and cartridge fired pins should be reduced if the fasteners are not located centrally in the troughs of the sheeting. If attachment is at a quarter point, the design resistance should be reduced to $0,9F_{p,Rd}$ and if there are fasteners at both quarter points, the resistance should be taken as $0,7F_{p,Rd}$ per fastener, see figure 8.2.

(8) For a fastener loaded in combined shear and tension, provided that both $F_{t,Rd}$ and $F_{v,Rd}$ are determined by calculation on the basis of tables 8.1 to 8.4, the resistance of the fastener to combined shear and tension may be verified using:

$$\frac{F_{t,Ed}}{\min(F_{p,Rd}, F_{o,Rd})} + \frac{F_{v,Ed}}{\min(F_{b,Rd}, F_{n,Rd})} \leq 1 \qquad \text{... (8.2)}$$

(9) The gross section distortion may be neglected if the design resistance is obtained from tables 8.1 to 8.4, provided that the fastening is through a flange not more than 150 mm wide.

(10)The diameter of holes for screws should be in accordance with the manufacturer's guidelines. These guidelines should be based on following criteria:

- the applied torque should be just higher than the threading torque;
- the applied torque should be lower than the thread stripping torque or head-shearing torque;
- the threading torque should be smaller than 2/3 of the head-shearing torque.

(11)For long joints a reduction factor β_{Lf} should be taken into account according to EN 1993-1-8.

(12)The design rules for blind rivets are valid only if the diameter of the hole is not 0,1 mm larger than the diameter of the rivet.

(13)For the bolts M12 and M14 with the hole diameters 2 mm larger than the bolt diameter, reference is made to EN 1993-1-8.

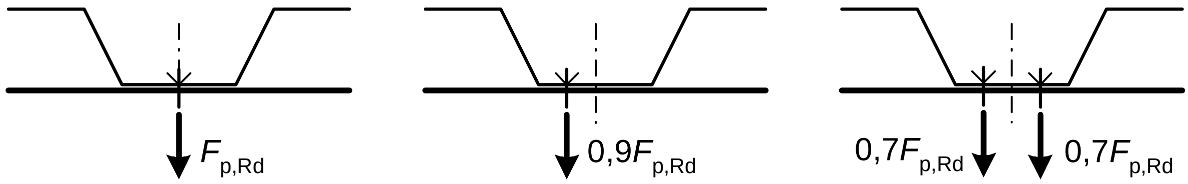


Figure 8.2: Reduction of tension resistance due to the position of fasteners

Table 8.1: Design resistances for blind rivets [1)]

Rivets loaded in shear:

Bearing resistance:

$F_{b,Rd} = \alpha f_u d\, t / \gamma_{M2}$ but $F_{b,Rd} \leq f_u e_1 t / (1{,}2\ \gamma_{M2})$

In which α is given by the following:

- if $t = t_1$: $\alpha = 3{,}6\sqrt{t/d}$ but $\alpha \leq 2{,}1$
- if $t_1 \geq 2{,}5t$: $\alpha = 2{,}1$
- if $t < t_1 < 2{,}5t$: obtain α by linear interpolation.

Net-section resistance:

$F_{n,Rd} = A_{net} f_u / \gamma_{M2}$

Shear resistance:

Shear resistance $F_{v,Rd}$ to be determined by testing *[1)] and $F_{v,Rd} = F_{v,Rk} / \gamma_{M2}$

Conditions: [4)] $F_{v,Rd} \geq 1{,}2 F_{b,Rd} / (n_f \beta_{Lf})$ or $F_{v,Rd} \geq 1{,}2 F_{n,Rd}$

Rivets loaded in tension: [2)]

Pull-through resistance: Pull-through resistance $F_{p,Rd}$ to be determined by testing *[1)].

Pull-out resistance: Not relevant for rivets.

Tension resistance: Tension resistance $F_{t,Rd}$ to be determined by testing *[1)]

Conditions:

$F_{t,Rd} \geq n F_{p,Rd}$

Range of validity: [3)]

$e_1 \geq 1{,}5d$ $p_1 \geq 3d$ $2{,}6\ \text{mm} \leq d \leq 6{,}4\ \text{mm}$

$e_2 \geq 1{,}5d$ $p_2 \geq 3d$

$f_u \leq 550\ \text{MPa}$

[1)] In this table it is assumed that the thinnest sheet is next to the preformed head of the blind rivet.

[2)] Blind rivets are not usually used in tension.

[3)] Blind rivets may be used beyond this range of validity if the resistance is determined from the results of tests.

[4)] The required conditions should be fulfilled when deformation capacity of the connection is needed. When these conditions are not fulfilled there should be proved that the needed deformation capacity will be provided by other parts of the structure.

NOTE*[1)] The National Annex may give further information on shear resistance of blind rivets loaded in shear and pull-through resistance and tension resistance of blind rivets loaded in tension.

Table 8.2: Design resistances for self-tapping screws [1)]

Screws loaded in shear:
Bearing resistance: $F_{b,Rd} = \alpha f_u d t / \gamma_{M2}$ In which α is given by the following: - if $t = t_1$: $\alpha = 3,2\sqrt{t/d}$ but $\alpha \leq 2,1$ - if $t_1 \geq 2,5t$ and $t < 1,0$ mm: $\alpha = 3,2\sqrt{t/d}$ but $\alpha \leq 2,1$ - if $t_1 \geq 2,5t$ and $t \geq 1,0$ mm: $\alpha = 2,1$ - if $t < t_1 < 2,5t$: obtain α by linear interpolation.   Net-section resistance: $F_{n,Rd} = A_{net} f_u / \gamma_{M2}$ Shear resistance: Shear resistance $F_{v,Rd}$ to be determined by testing *[2)] $F_{v,Rd} = F_{v,Rk} / \gamma_{M2}$
Conditions: [4)] $F_{v,Rd} \geq 1,2 F_{b,Rd} / (n_f \beta_{Lf})$ or $F_{v,Rd} \geq 1,2 F_{n,Rd}$
Screws loaded in tension:
Pull-through resistance: [2)]   - for static loads: $F_{p,Rd} = d_w t f_u / \gamma_{M2}$ - for screws subject to wind loads and combination of wind loads and static loads: $F_{p,Rd} = 0,5 d_w t f_u / \gamma_{M2}$ Pull-out resistance: If $t_{sup} / s < 1$: $F_{o,Rd} = 0,45 d t_{sup} f_{u,sup} / \gamma_{M2}$ (s is the thread pitch)   If $t_{sup} / s \geq 1$: $F_{o,Rd} = 0,65 d t_{sup} f_{u,sup} / \gamma_{M2}$ Tension resistance: Tension resistance $F_{t,Rd}$ to be determined by testing *[2)].
Conditions: [4)] $F_{t,Rd} \geq n F_{p,Rd}$ or $F_{t,Rd} \geq F_{o,Rd}$
Range of validity: [3)]
Generally: $e_1 \geq 3d$ $p_1 \geq 3d$ $3,0 \text{ mm} \leq d \leq 8,0 \text{ mm}$ $e_2 \geq 1,5d$ $p_2 \geq 3d$ For tension: $0,5 \text{ mm} \leq t \leq 1,5 \text{ mm}$ and $t_1 \geq 0,9$ mm $f_u \leq 550$ MPa
[1)] In this table it is assumed that the thinnest sheet is next to the head of the screw.   [2)] These values assume that the washer has sufficient rigidity to prevent it from being deformed appreciably or pulled over the head of the fastener.   [3)] Self-tapping screws may be used beyond this range of validity if the resistance is determined from the results of tests.   [4)] The required conditions should be fulfilled when deformation capacity of the connection is needed. When these conditions are not fulfilled there should be proved that the needed deformation capacity will be provided by other parts of the structure.

NOTE*[2)] The National Annex may give further information on shear resistance of self-tapping skrews loaded in shear and tension resistance of self-tapping skrews loaded in tension.

Table 8.3: Design resistances for cartridge fired pins

Pins loaded in shear:

Bearing resistance:

$F_{b,Rd} \;=\; 3{,}2\, f_u\, d\, t / \gamma_{M2}$

Net-section resistance: $F_{n,Rd} \;=\; A_{net} f_u / \gamma_{M2}$

Shear resistance: Shear resistance $F_{v,Rd}$ to be determined by testing *[3)]

$F_{v,Rd} \;=\; F_{v,Rk} / \gamma_{M2}$

Conditions: [3)] $F_{v,Rd} \geq 1{,}5\, F_{b,Rd} / (n_f\, \beta_{Lf})$ or $F_{v,Rd} \geq 1{,}5\, F_{n,Rd}$

Pins loaded in tension:

Pull-through resistance: [1)]

- for static loads: $F_{p,Rd} \;=\; d_w\, t\, f_u / \gamma_{M2}$
- for wind loads and combination of wind loads and static loads: $F_{p,Rd} \;=\; 0{,}5\, d_w\, t\, f_u / \gamma_{M2}$

Pull-out resistance:

Pull-out resistance $F_{o,Rd}$ to be determined by testing *[3)]

Tension resistance:

Tension resistance $F_{t,Rd}$ to be determined by testing *[3)]

Conditions: [3)] $F_{o,Rd} \geq n\, F_{p,Rd}$ or $F_{t,Rd} \geq F_{o,Rd}$

Range of validity: [2)]

Generally:	$e_1 \geq 4{,}5\, d$	$3{,}7 \text{ mm} \leq d \leq 6{,}0 \text{ mm}$
	$e_2 \geq 4{,}5\, d$	for $d = 3{,}7$ mm: $t_{sup} \geq 4{,}0$ mm
	$p_1 \geq 4{,}5\, d$	for $d = 4{,}5$ mm: $t_{sup} \geq 6{,}0$ mm
	$p_2 \geq 4{,}5\, d$	for $d = 5{,}2$ mm: $t_{sup} \geq 8{,}0$ mm
	$f_u \leq 550$ MPa	
For tension:	$0{,}5 \text{ mm} \leq t \leq 1{,}5 \text{ mm}$	$t_{sup} \geq 6{,}0$ mm

[1)] These values assume that the washer has sufficient rigidity to prevent it from being deformed appreciably or pulled over the head of the fastener.

[2)] Cartridge fired pins may be used beyond this range of validity if the resistance is determined from the results of tests.

[3)] The required conditions should be fulfilled when deformation capacity of the connection is needed. When these conditions are not fulfilled there should be proved that the needed deformation capacity will be provided by other parts of the structure.

NOTE*[3)] The National Annex may give further information on shear resistance of cartrige fired pins loaded in shear and pull-out resistance and tension resistance of cartridge fired pins loaded in tension.

Table 8.4: Design resistances for bolts

Bolts loaded in shear:

Bearing resistance: 2)

$F_{b,Rd} = 2{,}5\alpha_b\, k_t\, f_u\, d\, t\, /\, \gamma_{M2}$ with α_b is the smallest of 1,0 or $e_1\,/\,(3d)$ and

$k_t = (0{,}8\, t + 1{,}5)\,/\,2{,}5$ for 0,75 mm $\leq t \leq$ 1,25 mm; $k_t = 1{,}0$ for $t >$ 1,25 mm

Net-section resistance:

$F_{n,Rd} = (1 + 3\, r\, (d_o / u - 0{,}3))\, A_{net} f_u / \gamma_{M2}$ but $F_{n,Rd} \leq A_{net} f_u / \gamma_{M2}$

with:

r = [number of bolts at the cross-section] / [total number of bolts in the connection]

$u = 2\, e_2$ but $u \leq p_2$

Shear resistance:

- for strength grades 4.6, 5.6 and 8.8:

$F_{v,Rd} = 0{,}6\, f_{ub} A_s / \gamma_{M2}$

- for strength grades 4.8, 5.8, 6.8 and 10.9:

$F_{v,Rd} = 0{,}5\, f_{ub} A_s / \gamma_{M2}$

Conditions: 3) $F_{v,Rd} \geq 1{,}2\, F_{b,Rd}\, /(n_f\, \beta_{Lf})$ or $F_{v,Rd} \geq 1{,}2\, F_{n,Rd}$

Bolts loaded in tension:

Pull-through resistance: Pull-through resistance $F_{p,Rd}$ to be determined by testing *4).

Pull-out resistance: Not relevant for bolts.

Tension resistance: $F_{t,Rd} = 0{,}9\, f_{ub} A_s / \gamma_{M2}$

Conditions: 3) $F_{t,Rd} \geq n\, F_{p,Rd}$

Range of validity: 1)

$e_1 \geq 1{,}0\, d$	$p_1 \geq 3\, d$	3 mm $> t \geq$ 0,75 mm	Minimum bolt size: M 6
$e_2 \geq 1{,}5\, d$	$p_2 \geq 3\, d$		Strength grades: 4.6 - 10.9
$f_u \leq 550$ N/mm^2			

1) Bolts may be used beyond this range of validity if the resistance is determined from the results of tests.

2) For thickness larger than or equal to 3 mm the rules for bolts in EN 1993-1-8 should be used.

3) The required conditions should be fulfilled when deformation capacity of the connection is needed. When these conditions are not fulfilled there should be proved that the needed deformation capacity will be provided by other parts of the structure.

NOTE*4) The National Annex may give further information on pull-through resistance of bolts loaded in tension.

8.4 Spot welds

(1) Spot welds may be used with as-rolled or galvanized parent material up to 4,0 mm thick, provided that the thinner connected part is not more than 3,0 mm thick.

(2) Spot welds may be either resistance welded or fusion welded.

(3) The design resistance $F_{v,Rd}$ of a spot weld loaded in shear should be determined using table 8.5.

(4) In table 8.5 the meanings of the symbols should be taken as follows:

A_{net}	is	the net cross-sectional area of the connected part;
n_w	is	the number of spot welds in one connection;
t	is	the thickness of the thinner connected part or sheet [mm];
t_1	is	the thickness of the thicker connected part or sheet;

and the end and edge distances e_1 and e_2 and the spacings p_1 and p_2 are as defined in 8.4(5).

(5) The partial factor γ_M for calculating the design resistances of spot welds shall be taken as γ_{M2}.

NOTE The National Annex may chose the value of γ_{M2}. The value $\gamma_{M2} = 1,25$ is recommended.

Table 8.5: Design resistances for spot welds

Spot welds loaded in shear:
Tearing and bearing resistance: - if $t \leq t_1 \leq 2,5t$: $F_{tb,Rd} = 2,7\sqrt{t}\, d_s f_u / \gamma_{M2}$ [with t in mm] - if $t_1 > 2,5t$: $F_{tb,Rd} = 2,7\sqrt{t}\, d_s f_u / \gamma_{M2}$ but $F_{tb,Rd} \leq 0,7 d_s^2 f_u / \gamma_{M2}$ and $F_{tb,Rd} \leq 3,1 t\, d_s f_u / \gamma_{M2}$
End resistance: $F_{e,Rd} = 1,4\, t\, e_1 f_u / \gamma_{M2}$
Net section resistance: $F_{n,Rd} = A_{net} f_u / \gamma_{M2}$
Shear resistance: $F_{V,Rd} = \frac{\pi}{4} d_s^2 f_u / \gamma_{M2}$
Conditions: $F_{v,Rd} \geq 1,25 F_{tb,Rd}$ or $F_{v,Rd} \geq 1,25 F_{e,Rd}$ or $F_{v,Rd} \geq 1,25 F_{n,Rd} / n_w$
Range of validity:
$2d_s \leq e_1 \leq 6d_s$ $3d_s \leq p_1 \leq 8d_s$ $e_2 \leq 4d_s$ $3d_s \leq p_2 \leq 6d_s$

(6) The interface diameter d_s of a spot weld should be determined from the following:

- for fusion welding: $d_s = 0,5t + 5$ mm ... (8.3a)

- for resistance welding: $d_s = 5\sqrt{t}$ [with t in mm] ... (8.3b)

(7) The value of d_s actually produced by the welding procedure should be verified by shear tests in accordance with Section 9, using single-lap test specimens as shown in figure 8.3. The thickness t of the specimen should be the same as that used in practice.

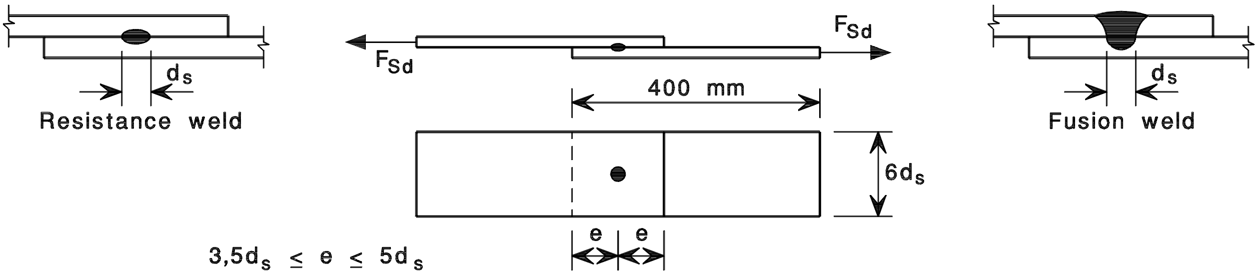


Figure 8.3: Test specimen for shear tests of spot welds

8.5 Lap welds

8.5.1 General

(1) This clause 8.6 shall be used for the design of arc-welded lap welds where the parent material is 4,0 mm thick or less. For thicker parent material, lap welds shall be designed using EN 1993-1-1.

(2) The weld size shall be chosen such that the resistance of the connection is governed by the thickness of the connected part or sheet, rather than the weld.

(3) The requirement in (2) may be assumed to be satisfied if the throat size of the weld is at least equal to the thickness of the connected part or sheet.

(4) The partial factor γ_M for calculating the design resistances of lap welds shall be taken as γ_{M2}.

NOTE The NAtional Annex may give a choice of γ_{M2}. The value $\gamma_{M2} = 1{,}25$ is recommended.

8.5.2 Fillet welds

(1) The design resistance $F_{w.Rd}$ of a fillet-welded connection should be determined from the following:

- for a side fillet that comprises one of a pair of side fillets:

$$F_{w,Rd} = t L_{w,s} (0{,}9 - 0{,}45 L_{w,s} / b) f_u / \gamma_{M2} \quad \text{if } L_{w,s} \leq b \quad \text{... (8.4a)}$$

$$F_{w,Rd} = 0{,}45 t\, b\, f_u / \gamma_{M2} \quad \text{if } L_{w,s} > b \quad \text{... (8.4b)}$$

- for an end fillet:

$$F_{w,Rd} = t L_{w,e} (1 - 0{,}3 L_{w,e} / b) f_u / \gamma_{M2} \quad [\text{for one weld and if } L_{w,s} \leq b] \quad \text{... (8.4c)}$$

where:

b is the width of the connected part or sheet, see figure 8.4;

$L_{w,e}$ is the effective length of the end fillet weld, see figure 8.4;

$L_{w,s}$ is the effective length of a side fillet weld, see figure 8.4.

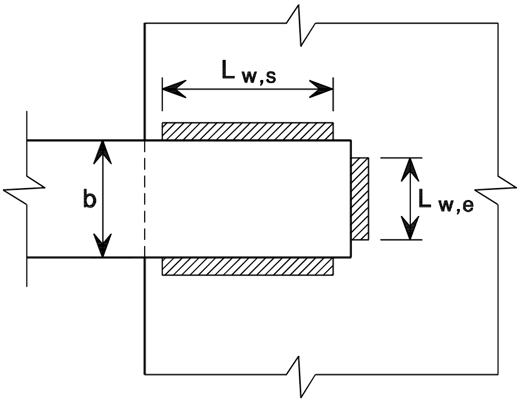


Figure 8.4: Fillet welded lap connection

(2) If a combination of end fillets and side fillets is used in the same connection, its total resistance should be taken as equal to the sum of the resistances of the end fillets and the side fillets. The position of the centroid and realistic assumption of the distribution of forces should be taken into account.

(3) The effective length L_w of a fillet weld should be taken as the overall length of the full-size fillet, including end returns. Provided that the weld is full size throughout this length, no reduction in effective length need be made for either the start or termination of the weld.

(4) Fillet welds with effective lengths less than 8 times the thickness of the thinner connected part should not be designed to transmit any forces.

8.5.3 Arc spot welds

(1) Arc spot welds shall not be designed to transmit any forces other than in shear.

(2) Arc spot welds should not be used through connected parts or sheets with a total thickness Σt of more than 4 mm.

(3) Arc spot welds should have an interface diameter d_s of not less than 10 mm.

(4) If the connected part or sheet is less than 0,7 mm thick, a weld washer should be used, see figure 8.5.

(5) Arc spot welds should have adequate end and edge distances as given in the following:

(i) The minimum distance measured parallel to the direction of force transfer, from the centreline of an arc spot weld to the nearest edge of an adjacent weld or to the end of the connected part towards which the force is directed, should not be less than the value of e_{min} given by the following:

if $f_u / f_y \leq 1,15$

$$e_{min} = 1,8 \frac{F_{w,Sd}}{t f_u / \gamma_{M2}}$$

if $f_u / f_y \geq 1,15$

$$e_{min} = 2,1 \frac{F_{w,Sd}}{t f_u / \gamma_{M2}}$$

(ii) The minimum distance from the centreline of a circular arc spot weld to the end or edge of the connected sheet should not be less than $1,5 d_w$ where d_w is the visible diameter of the arc spot weld.

(iii) The minimum clear distance between an elongated arc spot weld and the end of the sheet and between the weld and the edge of the sheet should not be less than $1,0\ d_w$.

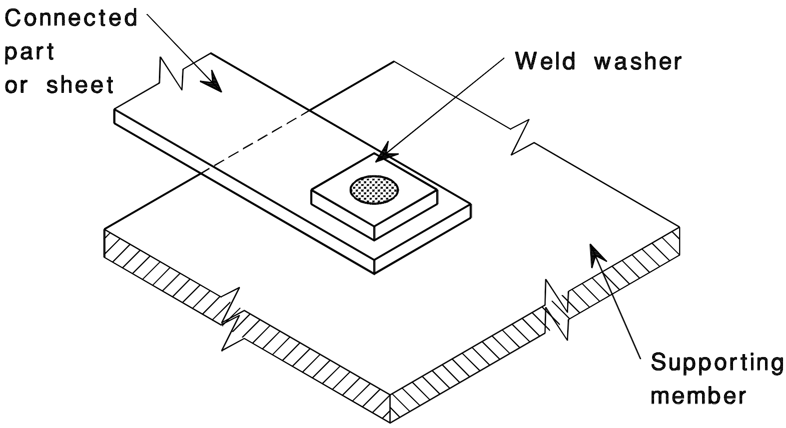


Figure 8.5: Arc spot weld with weld washer

(6) The design shear resistance $F_{w,Rd}$ of a circular arc spot weld should be determined as follows:

$$F_{w,Rd} = (\pi/4)\, d_s^2 \times 0{,}625 f_{uw} / \gamma_{M2} \quad \text{... (8.5a)}$$

where:

f_{uw} is the ultimate tensile strength of the welding electrodes;

but $F_{w,Rd}$ should not be taken as more than the peripheral resistance given by the following:

- if $d_p / \Sigma t \leq 18\,(420 / f_u)^{0,5}$:

$$F_{w,Rd} = 1{,}5\, d_p \Sigma t \ f_u / \gamma_{M2} \quad \text{... (8.5b)}$$

- if $18\,(420 / f_u)^{0,5} < d_p / \Sigma t < 30\,(420 / f_u)^{0,5}$:

$$F_{w,Rd} = 27\,(420 / f_u)^{0,5}\ (\Sigma t)^2\ f_u / \gamma_{M2} \quad \text{... (8.5c)}$$

- if $d_p / \Sigma t \geq 30\,(420 / f_u)^{0,5}$:

$$F_{w,Rd} = 0{,}9\, d_p \Sigma t\ f_u / \gamma_{M2} \quad \text{... (8.5d)}$$

(7) The interface diameter d_s of an arc spot weld, see figure 8.6, should be obtained from:

$$d_s = 0{,}7 d_w - 1{,}5 \Sigma t \quad \text{but} \quad d_s \geq 0{,}55\, d_w \quad \text{... (8.6)}$$

where:

d_w is the visible diameter of the arc spot weld, see figure 8.6.

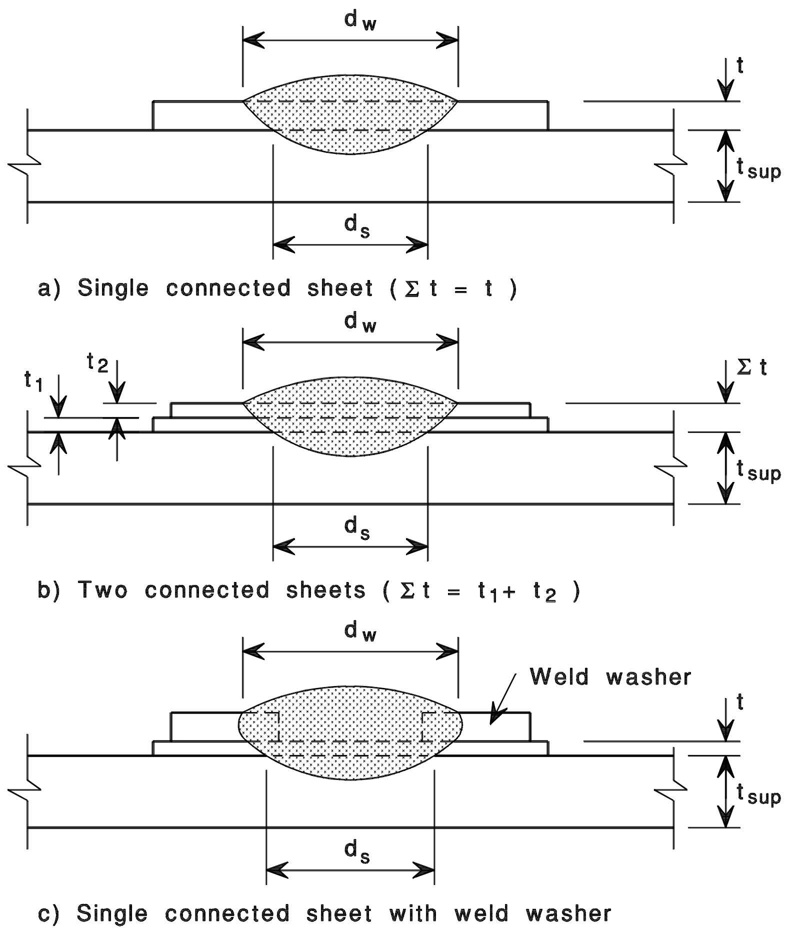


Figure 8.6: Arc spot welds

(8) The effective peripheral diameter d_p of an arc spot weld should be obtained as follows:

- for a single connected sheet or part of thickness t:

$$d_p = d_w - t \qquad \text{... (8.7a)}$$

- for multiple connected sheets or parts of total thickness Σt:

$$d_p = d_w - 2\Sigma t \qquad \text{... (8.7b)}$$

(9) The design shear resistance $F_{w,Rd}$ of an elongated arc spot weld should be determined from:

$$F_{w,Rd} = [(\pi/4) d_s^2 + L_w d_s] \times 0{,}625 f_{uw} / \gamma_{M2} \qquad \text{... (8.8a)}$$

but $F_{w,Rd}$ should not be taken as more than the peripheral resistance given by:

$$F_{w,Rd} = (0{,}5 L_w + 1{,}67 d_p) \Sigma t f_u / \gamma_{M2} \qquad \text{... (8.8b)}$$

where:

L_w is the length of the elongated arc spot weld, measured as shown in figure 8.7.

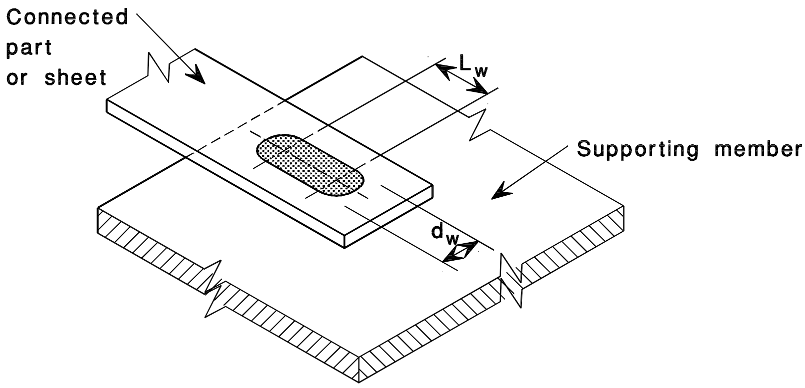


Figure 8.7: Elongated arc spot weld

9 Design assisted by testing

(1) This Section 9 may be used to apply the principles for design assisted by testing given in EN 1990 and in Section 2.5. of EN 1993-1-1, with the additional specific requirements of cold-formed thin gauge members and sheeting.

(2) Testing should be in compliance with Annex A.

NOTE: The National Annex may give informations on testing.

NOTE: Annex A gives standardised procedures for:

- tests on profiled sheets and liner trays;
- tests on cold-formed members;
- tests on structures and portions of structures;
- tests on beams torsionally restrained by sheeting;
- evaluation of test results to determine design values.

(3) Tensile testing of steel should be carried out in accordance with EN 10002-1. Testing of other steel properties should be carried out in accordance with the relevant European Standards.

(4) Testing of fasteners and connections should be carried out in accordance with the relevant European Standard or International Standard.

NOTE: Pending availability of an appropriate European or International Standard, imformation on testing procedures for fasteners may be obtained from:

ECCS Publication No. 21 (1983): *European recommendations for steel construction: the design and testing of connections in steel sheeting and sections*;

ECCS Publication No. 42 (1983): *European recommendations for steel construction: mechanical fasteners for use in steel sheeting and sections*.

10 Special considerations for purlins, liner trays and sheetings

10.1 Beams restrained by sheeting

10.1.1 General

(1) The provisions given in this clause 10.1 may be applied to beams (called purlins in this Section) of *Z*, *C*, Σ, *U*, *Zed* and *Hat* cross-section with $h / t < 233$, $c / t \leq 20$ for single fold and $d / t \leq 20$ for double edge fold (other limits as in table 5.1 and clause 5.2(5) and with continuous full lateral restraint to one flange.

NOTE The National Annex may give informations on tests. Standard tests as given in Annex A are recommended.

(2) These provisions may be used for structural systems of purlins with anti-sag bars, continuous, sleeved and overlapped systems.

(3) These provisions may also be applied to cold-formed members used as side rails, floor beams and other similar types of beam that are similarly restrained by sheeting.

(4) Side rails may be designed on the basis that wind pressure has a similar effect on them to gravity loading on purlins, and that wind suction acts on them in a similar way to uplift loading on purlins.

(5) Full continuous lateral restraint may be supplied by trapezoidal steel sheeting or other profiled steel sheeting with sufficient stiffness, continuously connected to the flange of the purlin through the troughs of the sheets. The purlin at the connection to trapetzoidal sheeting may be regarded as laterally restrained, if clause 10.1.1(6) is fulfilled. In other cases (for example, fastening through the crests of the sheets) the degree of restraint should either be validated by experience, or determined from tests.

NOTE For tests see Annex A.

(6) If the trapetzoidal sheeting is connected to a purlin and the condition expressed by the equation (10.1a) is met, the purlin at the connection may be regarded as being laterally restrained in the plane of the sheeting:

$$S \geq \left(EI_{\mathrm{w}} \frac{\pi^2}{L^2} + GI_{\mathrm{t}} + EI_{\mathrm{z}} \frac{\pi^2}{L^2} 0{,}25\, h^2 \right) \frac{70}{h^2} \qquad \ldots(10.1a)$$

where

- S is the portion of the shear stiffness provided by the sheeting for the examined member connected to the sheeting at each rib (If the sheeting is connected to a purlin every second rib only, then S should be substituted by $0{,}20\, S$);
- I_{w} is the warping constant of the purlin;
- I_{t} is the torsion constant of the purlin;
- I_{z} is the second moment of area of the cross-section about the minor axis of the cross-section of the purlin;
- L is the span of the purlin;
- h is the height of the purlin.

NOTE 1: The equation (10.1a) may also be used to determine the lateral stability of member flanges used in combination with other types of cladding than trapetzoidal sheeting, provided that the connections are of suitable design.

NOTE 2: The shear stiffness S may be calculated using ECCS guidance (see NOTE in 9.1(5)) or determined by tests.

(7) Unless alternative support arrangements may be justified from the results of tests the purlin should have support details, such as cleats, that prevent rotation and lateral displacement at its supports. The effects of forces in the plane of the sheeting, that are transmitted to the supports of the purlin, should be taken into account in the design of the support details.

(8) The behaviour of a laterally restrained purlin should be modelled as outlined in figure 10.1. The connection of the purlin to the sheeting may be assumed to partially restrain the twisting of the purlin. This partial torsional restraint may be represented by a rotational spring with a spring stiffness C_{D}. The stresses in

the free flange, not directly connected to the sheeting, should then be calculated by superposing the effects of in-plane bending and the effects of torsion, including lateral bending due to cross-sectional distortion. The rotational restraint given by the sheeting should be determined following 10.1.5.

(9) Where the free flange of a single span purlin is in compression under uplift loading, allowance should also be made for the amplification of the stresses due to torsion and distortion.

(10) The shear stiffness of trapetzoidal sheeting connected to the purlin at each rib and connected in every side overlap may be calculated as

$$S = 1000\sqrt{t^3}\,(50 + 10\sqrt[3]{b_{roof}})\,\frac{s}{h_w} \qquad \text{(N)}, \qquad t \text{ and } b_{roof} \text{ in mm} \qquad \ldots(10.1b)$$

where t is the design thickness of sheeting, b_{roof} is the width of the roof, s is the distance between the purlins and h_w is the profile depth of sheeting. All dimensions are given in mm. For liner trays the shear stiffness is S_v times distance between purlins, where S_v is calculated according to 10.3.5(6).

10.1.2 Calculation methods

(1) Unless a second order analysis is carried out, the method given in 10.1.3 and 10.1.4 should be used to allow for the tendency of the free flange to move laterally (thus inducing additional stresses) by treating it as a beam subject to a lateral load $q_{h,Ed}$, see figure 10.1.

(2) For use in this method, the rotational spring should be replaced by an equivalent lateral linear spring of stiffness K. In determining K the effects of cross-sectional distortion should also be allowed for. For this purpose, the free flange may be treated as a compression member subject to a non-uniform axial force, with a continuous lateral spring support of stiffness K.

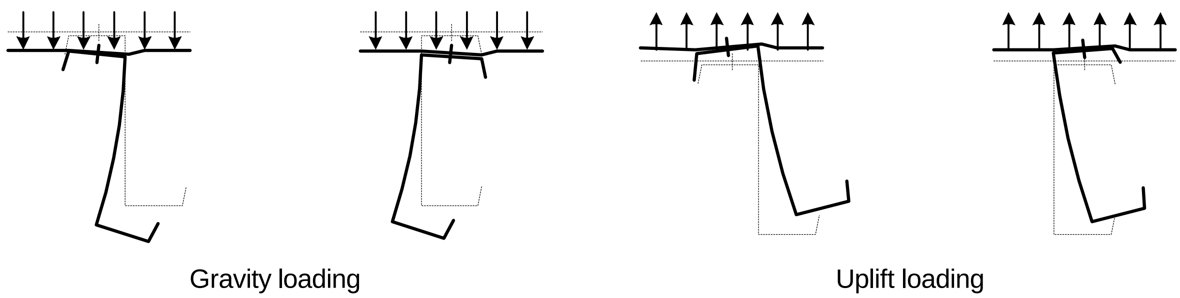
(3) If the free flange of a purlin is in compression due to in-plane bending (for example, due to uplift loading in a single span purlin), the resistance of the free flange to lateral buckling should also be verified.

(4) For a more precise calculation, a numerical analysis should be carried out, using values of the rotational spring stiffness C_D obtained from 10.1.5.2. Allowance should be made for the effects of an initial bow imperfection of (e_o) in the free flange, defined as in 5.3. The initial imperfection should be compatible with the shape of the relevant buckling mode, determined by the eigen-vectors obtained from the elastic first order buckling analysis.

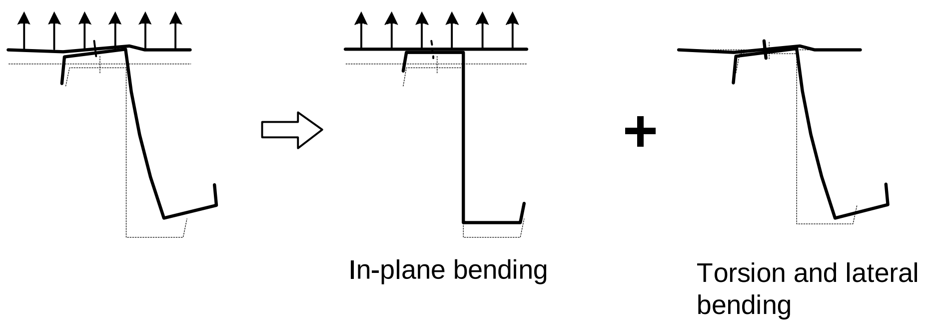
(5) A numerical analysis using the rotational spring stiffness C_D obtained from 10.1.5.2 may also be used if lateral restraint is not supplied or if its effectiveness cannot be proved. When the numerical analysis is carried out, it shall take into account the bending in two directions, torsional St Venant stiffness and warping stiffness about the imposed rotation axis.

(6) If a 2nd order analysis is carried out, effective sections and stiffness, due to local buckling, shall be taken into account.

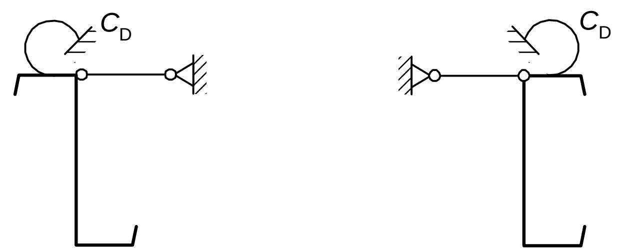
NOTE: For a simplified design of purlins made of C-, Z- and Σ- cross sections see Annex E.



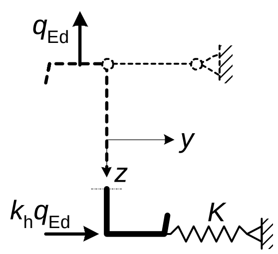
a) Z and C section purlin with upper flange connected to sheeting

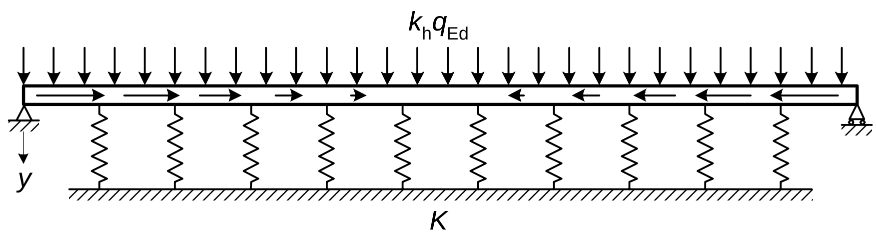


b) Total deformation split into two parts



c) Model purlin as laterally braced with rotationally spring restraint C_D from sheeting





d) As a simplification replace the rotational spring C_D by a lateral spring stiffness K

e) Free flange of purlin modelled as beam on elastic foundation. Model representing effect of torsion and lateral bending (including cross section distortion) on single span with uplift loading.

Figure 10.1: Modelling laterally braced purlins rotationally restrained by sheeting

10.1.3 Design criteria

10.1.3.1 Single span purlins

(1) For gravity loading, a single span purlin should satisfy the criteria for cross-section resistance given in 10.1.4.1. If it is subject to axial compression, it should also satisfy the criteria for stability of the free flange given in 10.1.4.2.

(2) For uplift loading, a single span purlin should satisfy the criteria for cross-section resistance given in 10.1.4.1 and the criteria for stability of the free flange given in 10.1.4.2.

10.1.3.2 Purlins continuous over two spans

(1) The moments due to gravity loading in a purlin that is physically continuous over two spans without overlaps or sleeves, may either be obtained by calculation or based on the results of tests.

(2) If the moments are calculated they should be determined using elastic global analysis. The purlin should satisfy the criteria for cross-section resistance given in 10.1.4.1. For the moment at the internal support, the criteria for stability of the free flange given in 10.1.4.2 should also be satisfied. For mid-support should be checked also for bending moment + support reaction (web crippling if cleats are not used) and for bending moment + shear forces depending on the case under consideration.

(3) Alternatively the moments may be determined using the results of tests in accordance with Section 9 and Annex A.5 on the moment-rotation behaviour of the purlin over the internal support.

NOTE: Appropriate testing procedures are given in Annex A.

(4) The design value of the resistance moment at the supports $M_{sup,Rd}$ for a given value of the load per unit length q_{Ed}, should be obtained from the intersection of two curves representing the design values of:

- the moment-rotation characteristic at the support, obtained by testing in accordance with Section 9 and Annex A.5;
- the theoretical relationship between the support moment $M_{sup,Ed}$ and the corresponding plastic hinge rotation ϕ_{Ed} in the purlin over the support.

To determine the final design value of the support moment $M_{sup,Ed}$ allowance should be made for the effect of the lateral load in the free flange and/or the buckling stability of that free flange around the mid-support, which are not fully taken into account by the internal support test as given in clause A.5.2. When the free flange is physically continued at the support and if the distance between the support and the nearest anti-sag bar is larger than 0,5 s, the lateral load $q_{h,Ed}$ according to 10.1.4.2 should be taken into account in verification of the resistance at mid-support. Alternatively, full-scale tests for two or multi-span purlins may be used to determine the effect of the lateral load in the free flange and/or the buckling stability of that free flange around the mid-support.

(5) The span moments should then be calculated from the value of the support moment.

(6) The following expressions may be used for a purlin with two equal spans:

$$\phi_{Ed} = \frac{L}{12\,E\,I_{eff}}\left[\,q_{Ed}\,L^2 - 8\,M_{sup,Ed}\,\right] \qquad \text{... (10.2a)}$$

$$M_{spn,Ed} = \frac{\left(q_{Ed}\,L^2 - 2\,M_{sup,Rd}\right)^2}{8\,q_{Ed}\,L^2} \qquad \text{... (10.2b)}$$

where:

I_{eff}	is	the effective second moment of area for the moment $M_{spn,Ed}$;
L	is	the span;
$M_{spn,Ed}$	is	the maximum moment in the span.

(7) The expressions for a purlin with two unequal spans, and for non-uniform loading (e.g. snow accumulation), and for other similar cases, the formulas (10.2a) and (10.2b) are not valid and approriate analysis should be made for these cases.

(8) The maximum span moment $M_{spn,Ed}$ in the purlin should satisfy the criteria for cross-section resistance given in 10.1.4.1. Alternatively the resistance moment in the span may be determined by testing using single span tests with a span comparable to the distance between the points of contraflexure in the span.

10.1.3.3 Two-span continuous purlins with uplift loading

(1) The moments due to uplift loading in a purlin that is physically continuous over two spans without overlaps or sleeves, should be determined using elastic global analysis.

(2) The moment over the internal support should satisfy the criteria for cross-section resistance given in 10.1.4.1. Because the support reaction is a tensile force, no account need be taken of its interaction with the support moment. The mid-support should be checked also for bending moment + shear forces.

(3) The moments in the spans should satisfy the criteria for stability of the free flange given in 10.1.4.2.

10.1.3.4 Purlins with semi-continuity given by overlaps or sleeves

(1) The moments in purlins in which continuity over two or more spans is given by overlaps or sleeves at internal supports, should be determined taking into account the effective section properties of the cross-section and the effects of the overlaps or sleeves.

(2) Tests may be carried out on the support details to determine:

- the flexural stiffness of the overlapped or sleeved part;
- the moment-rotation characteristic for the overlapped or sleeved part. Note, that only when the failure occurs at the support with cleat or similar preventing lateral displacements at the support, then the plastic redistribution of bending moments may be used for sleeved and overlapped systems;
- the resistance of the overlapped or sleeved part to combined support reaction and moment;
- the resistance of the non-overlapped unsleeved part to combined shear force and bending moment.

Alternatively the characteristics of the mid-support details may be determined by numerical methods if the design procedure is at least validated by a relevant numbers of tests.

(3) For gravity loading, the purlin should satisfy the following criteria:

- at internal supports, the resistance to combined support reaction and moment determined by testing;
- near supports, the resistance to combined shear force and bending moment determined by testing;
- in the spans, the criteria for cross-section resistance given in 10.1.4.1;
- if the purlin is subject to axial compression, the criteria for stability of the free flange given in 10.1.4.2.

(4) For uplift loading, the purlin should satisfy the following criteria:

- at internal supports, the resistance to combined support reaction and moment determined by testing, taking into account the fact that the support reaction is a tensile force in this case;
- near supports, the resistance to combined shear force and bending moment determined by testing;
- in the spans, the criteria for stability of the free flange given in 10.1.4.2;
- if the purlin is subjected to axial compression, the criteria for stability of the free flange is given in 10.1.4.2.

10.1.3.5 Serviceability criteria

(1) The serviceability criteria relevant to purlins should also be satisfied.

10.1.4 Design resistance

10.1.4.1 Resistance of cross-sections

(1) For a purlin subject to axial force and transverse load the resistance of the cross-section should be verified as indicated in figure 10.2 by superposing the stresses due to:

- the in-plane bending moment $M_{y,Ed}$;

- the axial force N_{Ed};

- an equivalent lateral load $q_{h,Ed}$ acting on the free flange, due to torsion and lateral bending, see (3).

(2) The maximum stresses in the cross-section should satisfy the following:

- restrained flange:

$$\sigma_{max,Ed} = \frac{M_{y,Ed}}{W_{eff,y}} + \frac{N_{Ed}}{A_{eff}} \leq f_y / \gamma_M \qquad \text{... (10.3a)}$$

- free flange:

$$\sigma_{max,Ed} = \frac{M_{y,Ed}}{W_{eff,y}} + \frac{N_{Ed}}{A_{eff}} + \frac{M_{fz,Ed}}{W_{fz}} \leq f_y / \gamma_M \qquad \text{... (10.3b)}$$

where:

A_{eff} is the effective area of the cross-section for only uniform compression;

f_y is the yield strength as defined in 3.2.1(5);

$M_{fz,Ed}$ is the bending moment in the free flange due to the lateral load $q_{h,Ed}$;

$W_{eff,y}$ is the effective section modulus of the cross-section for only bending about the y - y axis;

W_{fz} is the gross elastic section modulus of the free flange plus 0,27 of the web height for the point of web-flange intersection, for bending about the z - z axis;

and $\gamma_M = \gamma_{M0}$ if $A_{eff} = A_g$ or if $W_{eff,y} = W_{el,y}$ and $N_{Ed} = 0$, otherwise $\gamma_M = \gamma_{M1}$.

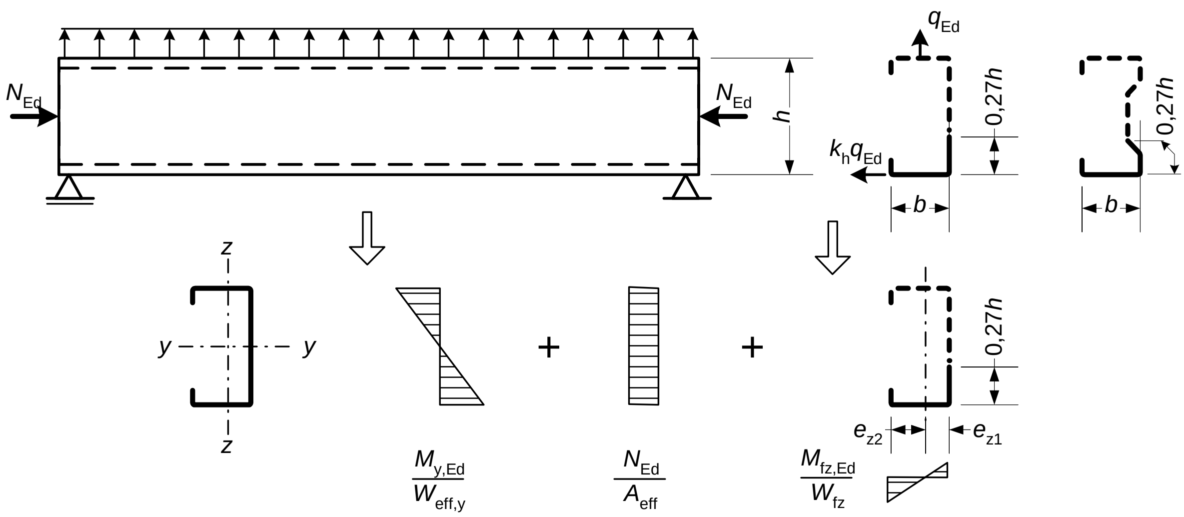
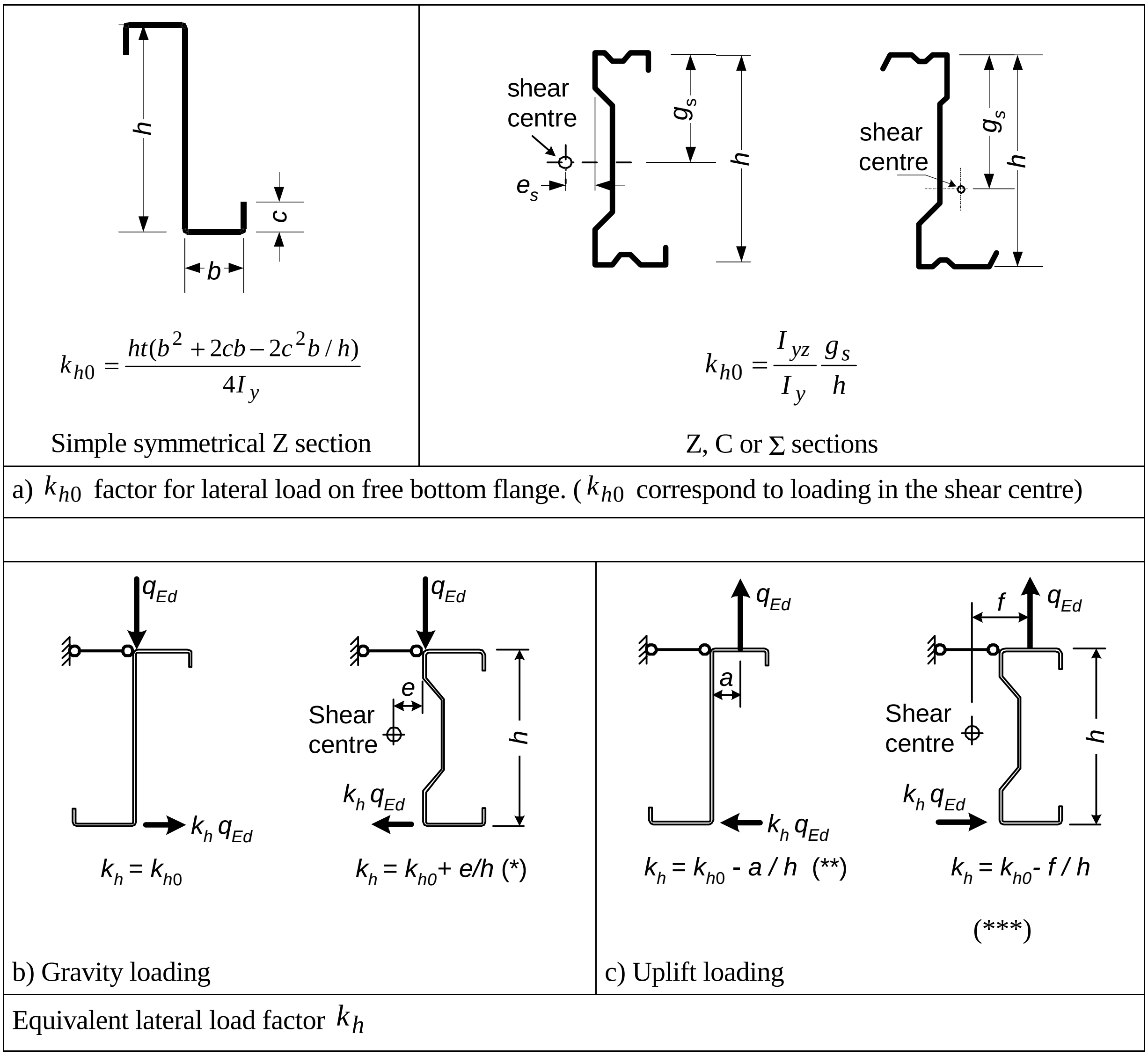


Figure 10.2: Superposition of stresses

(3) The equivalent lateral load $q_{h,Ed}$ acting on the free flange, due to torsion and lateral bending, should be obtained from:

$$q_{h,Ed} = k_h \, q_{Ed} \qquad \text{... (10.4)}$$

(4) The coefficient k_h should be obtained as indicated in figure 10.3 for common types of cross-section.



(*) If the shear centre is at the right hand side of the load q_{Ed} then the load is acting in the opposite direction.

(**) If $a/h > k_{h0}$ then the load is acting in the opposite direction.

(***) The value of f is limited to the position of the load q_{Ed} between the edges of the top flange.

Figure 10.3: Conversion of torsion and lateral bending into an equivalent lateral load $k_h \, q_{Ed}$

(5) The lateral bending moment $M_{fz,Ed}$ should be determined from expression (10.5) except for a beam with the free flange in tension, where, due to positive influence of flange curling and second order effect moment $M_{fz,Ed}$ may be taken equal to zero:

$$M_{fz,Ed} = \kappa_R \, M_{0,fz,Ed} \qquad \text{... (10.5)}$$

where:

$M_{0,fz,Ed}$ is the initial lateral bending moment in the free flange without any spring support;

κ_R is a correction factor for the effective spring support.

(6) The initial lateral bending moment in the free flange $M_{0,fz,Ed}$ should be determined from table 10.1 for the critical locations in the span, at supports, at anti-sag bars and between anti-sag bars. The validity of the table 10.1 is limited to the range $R \leq 40$.

(7) The correction factor κ_R for the relevant location and boundary conditions, should be determined from table 10.1 (or using the theory of beams on the elastic Winkler foundation), using the value of the coefficient R of the spring support given by:

$$R = \frac{K L_a^4}{\pi^4 E I_{fz}} \qquad \text{... (10.6)}$$

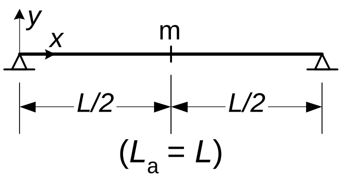
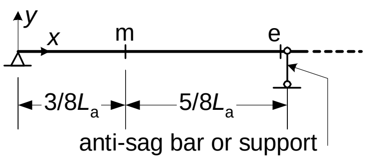
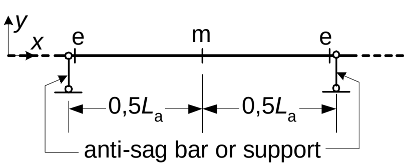
where:

I_{fz} is the second moment of area of the gross cross-section of the free flange plus 0,27 of the web height, for bending about the z - z axis, when numerical analysis is carried out, see 10.1.2(5);

K is the lateral spring stiffness per unit length from 10.1.5.1;

L_a is the distance between anti-sag bars, or if none are present, the span L of the purlin.

Table 10.1: Values of initial moment $M_{0,fz,Ed}$ and correction factor κ_R

System	Location	$M_{0,fz,Ed}$	κ_R
	m	$\frac{1}{8} q_{h,Ed} L_a^2$	$\kappa_R = \frac{1 - 0{,}0225R}{1 + 1{,}013R}$
	m	$\frac{9}{128} q_{h,Ed} L_a^2$	$\kappa_R = \frac{1 - 0{,}0141R}{1 + 0{,}416R}$
	e	$-\frac{1}{8} q_{h,Ed} L_a^2$	$\kappa_R = \frac{1 + 0{,}0314R}{1 + 0{,}396R}$
	m	$\frac{1}{24} q_{h,Ed} L_a^2$	$\kappa_R = \frac{1 - 0{,}0125R}{1 + 0{,}198R}$
	e	$-\frac{1}{12} q_{h,Ed} L_a^2$	$\kappa_R = \frac{1 + 0{,}0178R}{1 + 0{,}191R}$

10.1.4.2 Buckling resistance of free flange

(1) If the free flange is in compression, its buckling resistance should be verified using:

$$\frac{1}{\chi_{LT}}\left(\frac{M_{y,Ed}}{W_{eff,y}}+\frac{N_{Ed}}{A_{eff}}\right)+\frac{M_{fz,Ed}}{W_{fz}} \leq f_{yb}/\gamma_{M1} \qquad \text{... (10.7)}$$

in which χ_{LT} is the reduction factor for lateral torsional buckling (flexural buckling of the free flange), obtained from 6.2.3. using buckling curve b (imperfection factor $\alpha_{LT} = 0{,}34$) for the relative slenderness $\bar{\lambda}_{fz}$ given in (2). In the case of an axial compression force N_{Ed}, when the reduction factor for buckling around the strong axis is smaller than the reduction factor for lateral flange buckling, e.g. in the case of many anti-sag bars, this failure mode should also be checked following clause 6.2.2 and 6.2.4.

(2) The relative slenderness $\bar{\lambda}_{fz}$ for flexural buckling of the free flange should be determined from:

$$\bar{\lambda}_{fz} = \frac{l_{fz}/i_{fz}}{\lambda_1} \qquad \text{... (10.8)}$$

with:

$$\lambda_1 = \pi\left[E / f_{yb}\right]^{0,5}$$

where:

l_{fz} is the buckling length for the free flange from (3) to (7);

i_{fz} is the radius of gyration of the gross cross-section of the free flange plus 0,27 of the web height, about the z - z axis.

(3) For gravity loading, provided that $0 \leq R \leq 200$, the buckling length of the free flange for a variation of the compressive stress over the length L as shown in figure 10.4 may be obtained from:

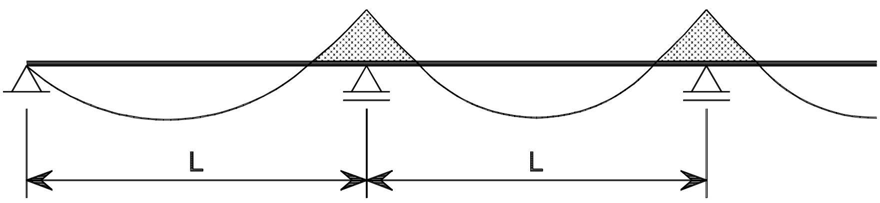
$$l_{fz} = \eta_1 L_a \left(1+\eta_2 R^{\eta_3}\right)^{\eta_4} \qquad \text{... (10.9)}$$

where:

L_a is the distance between anti-sag bars, or if none are present, the span L of the purlin;

R is as given in 10.1.4.1(7);

and η_1 to η_4 are coefficients that depend on the number of anti-sag bars, as given in table 10.2a.. The tables 10.2a and 10.2b are valid only for equal spans uniformly loaded beam systems without overlap or sleeve and with anti-sag bars that provide lateral rigid support for the free flange. Due to rotations in overlap or sleeve connection, the field moment may be much larger than support moment which results also longer buckling lengths in span. Neglecting the real moment distribution may lead to unsafe design. The tables may be used for systems with sleeves and overlaps provided that the connection system may be considered as fully continuous. In other cases the buckling length should be determined by more appropriate calculations or, except cantilevers, the values of the table 10.2a for the case of 3 anti-sag bars per field may be used.



[Dotted areas show regions in compression]

Figure 10.4: Varying compressive stress in free flange for gravity load cases

Table 10.2a : Coefficients η_i for down load with 0, 1, 2, 3, 4 anti-sag bars

Situation	Anti sag-bar Number	η_1	η_2	η_3	η_4
End span	0	0.414	1.72	1.11	-0.178
Intermediate span		0.657	8.17	2.22	-0.107
End span	1	0.515	1.26	0.868	-0.242
Intermediate span		0.596	2.33	1.15	-0.192
End and intermediate span	2	0.596	2.33	1.15	-0.192
End and intermediate span	3 and 4	0.694	5.45	1.27	-0.168

Table 10.2b : Coefficients η_i for uplift load with 0, 1, 2, 3, 4 anti-sag bars

Situation	Anti sag-bar Number	η_1	η_2	η_3	η_4
Simple span	0	0.694	5.45	1.27	-0.168
End span		0.515	1.26	0.868	-0.242
Intermediate span		0.306	0.232	0.742	-0.279
Simple and end spans	1	0.800	6.75	1.49	-0.155
Intermediate span		0.515	1.26	0.868	-0.242
Simple span	2	0.902	8.55	2.18	-0.111
End and intermediate spans		0.800	6.75	1.49	-0.155
Simple and end spans	3 and 4	0.902	8.55	2.18	-0.111
Intermediate span		0.800	6.75	1.49	-0.155

(4) For gravity loading, if there are more than three equally spaced anti-sag bars, under other conditions specified in (3), the buckling length need not be taken as greater than the value for two anti-sag bars, with $L_a = L/3$. This clause is valid only if there is no axial compressive force.

(5) If the compressive stress over the length L is almost constant, due to the application of a relatively large axial force, the buckling length should be determined using the values of η_i from table 10.2a for the case shown as more than three anti-sag bars per span, but the actual spacing L_a.

(6) For uplift loading, when anti-sag bars are not used, provided that $0 \le R_0 \le 200$, the buckling length of the free flange for variations of the compressive stress over the length L_0 as shown in figure 10.5, may be obtained from:

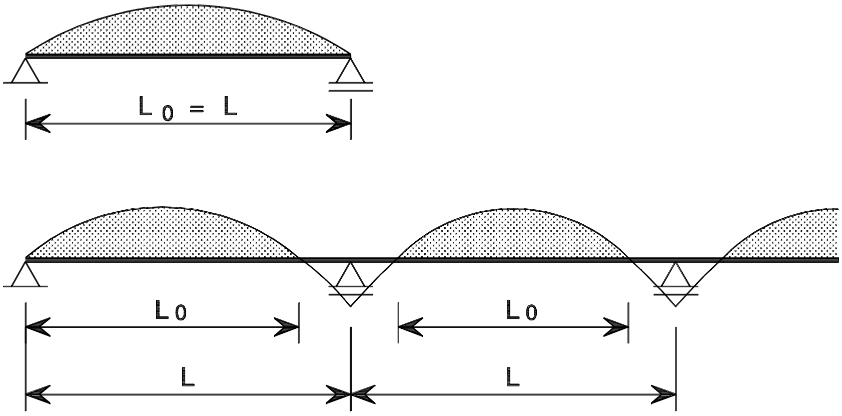
$$l_{fz} = 0{,}7\, L_0\, (1 + 13{,}1\, R_0^{1{,}6})^{-0{,}125} \qquad \text{... (10.10a)}$$

with:

$$R_0 = \frac{K\, L_0^4}{\pi^4\, E\, I_{fz}} \qquad \text{... (10.10b)}$$

in which I_{fz} and K are as defined in 10.1.4.1(7). Alternatively, the buckling length of the free flange may be determined using the table 10.2b in combination with the equation given in 10.1.4.2(3).

(7) For uplift loading, if the free flange is effectively held in position laterally at intervals by anti-sag bars, the buckling length may conservatively be taken as that for a uniform moment, determined as in (5).The formula (10.10a) may be applied under conditions specified in (3). If there are no appropriate calculations, reference should be made to 10.1.4.2(5).



[Dotted areas show regions in compression]

Figure 10.5: Varying compressive stress in free flange for uplift cases

10.1.5 Rotational restraint given by the sheeting

10.1.5.1 Lateral spring stiffness

(1) The lateral spring support given to the free flange of the purlin by the sheeting should be modelled as a lateral spring acting at the free flange, see figure 10.1. The total lateral spring stiffness K per unit length should be determined from:

$$\frac{1}{K} = \frac{1}{K_A} + \frac{1}{K_B} + \frac{1}{K_C} \qquad \text{... (10.11)}$$

where:

K_A is the lateral stiffness corresponding to the rotational stiffness of the connection between the sheeting and the purlin;

K_B is the lateral stiffness due to distortion of the cross-section of the purlin;

K_C is the lateral stiffness due to the flexural stiffness of the sheeting.

(2) Normally it may be assumed to be safe as well as acceptable to neglect $1/K_C$ because K_C is very large compared to K_A and K_B. The value of K should then be obtained from:

$$K = \frac{1}{(1/K_A + 1/K_B)} \qquad \text{... (10.12)}$$

(3) The value of $(1 / K_A + 1 / K_B)$ may be obtained either by testing or by calculation.

NOTE: Appropriate testing procedures are given in Annex A.

(4) The lateral spring stiffness K per unit length may be determined by calculation using:

$$\frac{1}{K} = \frac{4(1-\nu^2)\, h^2\, (h_d + b_{mod})}{E t^3} + \frac{h^2}{C_D} \qquad \text{... (10.13)}$$

in which the dimension b_{mod} is determined as follows:

- for cases where the equivalent lateral force bringing the purlin into contact with the sheeting at the purlin web:

$b_{mod} = a$

- for cases where the equivalent lateral force bringing the purlin into contact with the sheeting at the tip of the purlin flange:

$b_{mod} = \quad 2a + b$

where:

a is the distance from the sheet-to-purlin fastener to the purlin web, see figure 10.6;

b is the width of the purlin flange connected to the sheeting, see figure 10.6;

C_D is the total rotational spring stiffness from 10.1.5.2;

h is the overall height of the purlin;

h_d is the developed height of the purlin web, see figure 10.6.

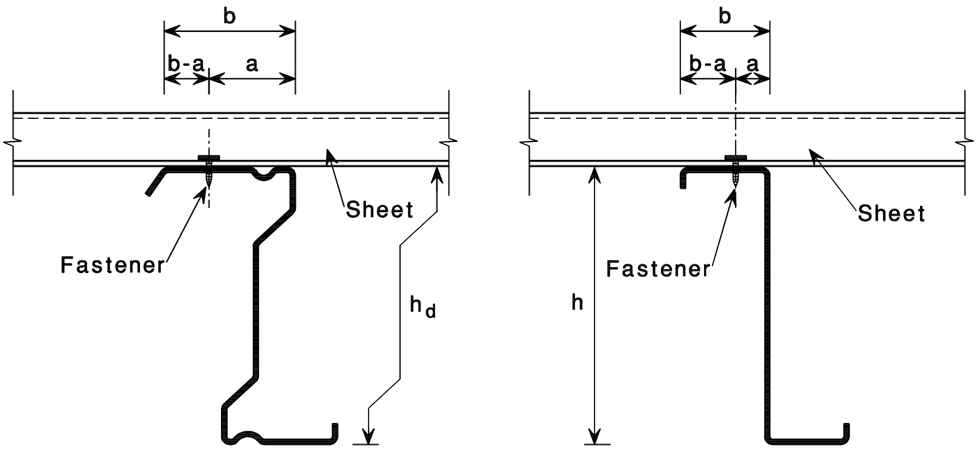


Figure 10.6: Purlin and attached sheeting

10.1.5.2 Rotational spring stiffness

(1) The rotational restraint given to the purlin by the sheeting that is connected to its top flange, should be modelled as a rotational spring acting at the top flange of the purlin, see figure 10.1. The total rotational spring stiffness C_D should be determined from:

$$C_D = \frac{1}{\left(1 / C_{D,A} + 1/C_{D,C}\right)} \qquad \text{...(10.14)}$$

where:

$C_{D,A}$ is the rotational stiffness of the connection between the sheeting and the purlin;

$C_{D,C}$ is the rotational stiffness corresponding to the flexural stiffness of the sheeting.

(2) Generally $C_{D,A}$ may be calculated as given in (5) and (7). Alternatively $C_{D,A}$ may be obtained by testing, see (9).

(3) The value of $C_{D,C}$ may be taken as the minimum value obtained from calculational models of the type shown in figure 10.7, taking account of the rotations of the adjacent purlins and the degree of continuity of the sheeting, using:

$$C_{D,C} = m/\theta \qquad \text{... (10.15)}$$

where:

I_{eff} is the effective second moment of area per unit width of the sheeting;

m is the applied moment per unit width of sheeting, applied as indicated in figure 10.7;

θ is the resulting rotation, measured as indicated in figure 10.7 [radians].

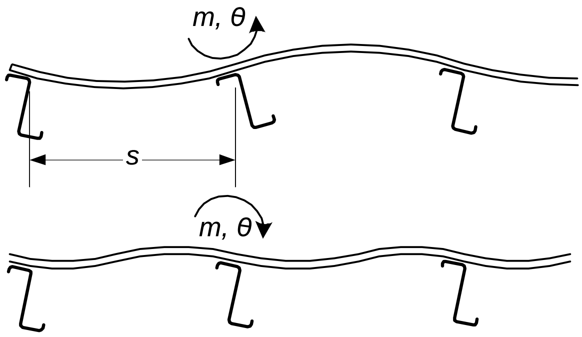


Figure 10.7: Model for calculating $C_{D,C}$

(4) Alternatively a conservative value of $C_{D,C}$ may be obtained from:

$$C_{D,C} = \frac{k\ E\ I_{eff}}{s} \qquad \text{... (10.16)}$$

in which k is a numerical coefficient, with values as follows:

- end, upper case of figure 10.7 $k = 2$;
- end, lower case of figure 10.7 $k = 3$;
- mid, upper case of figure 10.7 $k = 4$;
- mid, lower case of figure 10.7 $k = 6$;

where:

s is the spacing of the purlins.

(5) Provided that the sheet-to-purlin fasteners are positioned centrally on the flange of the purlin, the value of $C_{D,A}$ for trapezoidal sheeting connected to the top flange of the purlin may be determined as follows (see table 10.3):

$$C_{D,A} = C_{100} \cdot k_{ba} \cdot k_t \cdot k_{bR} \cdot k_A \cdot k_{bT} \qquad \text{...(10.17)}$$

where

$k_{ba} = (b_a / 100)^2$ if $b_a < 125\text{mm}$;

$k_{ba} = 1{,}25(b_a / 100)$ if $125\text{mm} \le b_a < 200\text{mm}$;

$k_t = (t_{nom} / 0{,}75)^{1{,}1}$ if $t_{nom} \ge 0{,}75\text{mm}$; positive position;

$k_t = (t_{nom} / 0{,}75)^{1{,}5}$ if $t_{nom} \ge 0{,}75\text{mm}$; negative position;

$k_t = (t_{nom} / 0{,}75)^{1{,}5}$ if $t_{nom} < 0{,}75\text{mm}$;

$k_{bR} = 1{,}0$ if $b_R \le 185\text{mm}$;

$k_{bR} = 185 / b_R$ if $b_R > 185\text{mm}$;

for gravity load:

$k_A = 1{,}0 + (A - 1{,}0) \cdot 0{,}08$ if $t_{nom} = 0{,}75\text{mm}$; positive position;

$k_A = 1{,}0 + (A - 1{,}0) \cdot 0{,}16$ if $t_{nom} = 0{,}75\text{mm}$; negative position;

$k_A = 1{,}0 + (A - 1{,}0) \cdot 0{,}095$ if $t_{nom} = 1{,}00\text{mm}$; positive position;

$k_A = 1{,}0 + (A - 1{,}0) \cdot 0{,}095$ if $t_{nom} = 1{,}00\text{mm}$; negative position;

for uplift load:

$k_A = 1{,}0$;

$k_{bT} = \sqrt{\dfrac{b_{T,max}}{b_T}}$ if $b_T > b_{T,max}$, otherwise $k_{bT} = 1$;

$A \leq 12\ kN/m$ load introduced from sheeting to beam;

where:

b_a is the width of the purlin flange [in mm];

b_R is the corrugation width [in mm];

b_T is the width of the sheeting flange through which it is fastened to the purlin;

C_{100} is a rotation coefficient, representing the value of $C_{D,A}$ if $b_a = 100$ mm.

(6) Provided that there is no insulation between the sheeting and the purlins, the value of the rotation coefficient C_{100} may be obtained from table 10.3.

(7) Alternatively $C_{D,A}$ may be taken as equal to $130\,p$ [Nm/m], where p is the number of sheet-to-purlin fasteners per metre length of purlin (but not more than one per rib of sheeting), provided that:

- the flange width b of the sheeting through which it is fastened does not exceed 120 mm;
- the nominal thickness t of the sheeting is at least 0,66 mm;
- the distance a or b - a between the centreline of the fastener and the centre of rotation of the purlin (depending on the direction of rotation), as shown in figure 10.6, is at least 25 mm.

(8) If the effects of cross-section distortion have to be taken into account, see 10.1.5.1, it may be assumed to be realistic to neglect $C_{D,C,}$ because the spring stiffness is mainly influenced by the value of $C_{D,A}$ and the cross-section distortion.

(9) Alternatively, values of $C_{D,A}$ may be obtained from a combination of testing and calculation.

(10)If the value of $(1 / K_A + 1 / K_B)$ is obtained by testing (in *mm/N* in accordance with A.5.3(3)), the values of $C_{D,A}$ for gravity loading and for uplift loading should be determined from:

$$C_{D,A} = \frac{h^2 / l_A}{(1/K_A + 1/K_B) - 4\,(1-\nu^2)\,h^2\,(h_d + b_{mod})/(E\,t^3\,l_B)} \quad \text{... (10.18)}$$

in which b_{mod}, h and h_d are as defined in 10.1.5.1(4) and l_A is the modular width of tested sheeting and l_B is the length of tested beam.

NOTE For testing see Annex A.5.3(3).

Table 10.3: Rotation coefficient C_{100} for trapezoidal steel sheeting

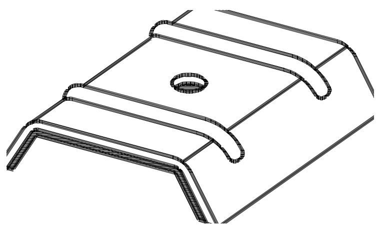
Positioning of sheeting		Sheet fastened through		Pitch of fasteners		Washer diameter [mm]	C_{100}	$b_{T,max}$
Positive 1)	Negative1)	Trough	Crest	$e = b_R$	$e = 2b_R$		[kNm/m]	[mm]
For gravity loading:								
×		×		×		22	5,2	40
×		×			×	22	3,1	40
	×		×	×		K_a	10,0	40
	×		×		×	K_a	5,2	40
	×	×		×		22	3,1	120
	×	×			×	22	2,0	120
For uplift loading:								
×		×		×		16	2,6	40
×		×			×	16	1,7	40

Key:

b_R is the corrugation width;

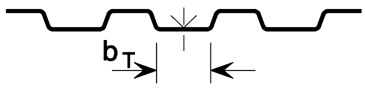
b_T is the width of the sheeting flange through which it is fastened to the purlin.

K_a indicates a steel saddle washer as shown below with $t \geq 0,75$ mm

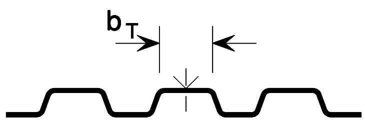


Sheet fastened:

- through the trough:



- through the crest:



The values in this table are valid for:

- sheet fastener screws of diameter: $\phi = 6,3$ mm;
- steel washers of thickness: $t_w \geq 1,0$ mm.

1) The position of sheeting in positive when the narrow flange is on the purlin and negative when the wide flange is on the purlin.

10.1.6 Forces in sheet/purlin fasteners and reaction forces

(1) Fasteners fixing the sheeting to the purlin shall be checked for a combination of shear force $q_s\, e$, perpendicular to the flange, and tension force $q_t\, e$ where q_s and q_t may be calculated using table 10.4 and e is the pitch of the fasteners. Shear force due to stabilising effect, see EN1993-1-1, shall be added to the shear force. Furthermore, shear force due to diaphragm action, acting parallel to the flange, shall be vectorially added to q_s.

Table 10.4 Shear force and tensile force in fastener along the beam

Beam and loading	Shear force per unite length q_s	Tensile force per unit length q_t
Z-beam, gravity loading	$(1+\xi)k_h q_{Ed}$, may be taken as 0	0
Z-beam, uplift loading	$(1+\xi)(k_h - a/h)q_{Ed}$	$\lvert \xi k_h q_{Ed} h/a \rvert + q_{Ed}$ $(a \cong b/2)$
C-beam, gravity loading	$(1-\xi)k_h q_{Ed}$	$\xi k_h q_{Ed} h/a$
C-beam, uplift loading	$(1-\xi)(k_h - a/h)q_{Ed}$	$\xi k_h q_{Ed} h/(b-a) + q_{Ed}$

(2) The fasteners fixing the purlins to the supports shall be checked for the reaction force R_w in the plane of the web and the transverse reaction forces R_1 and R_2 in the plane of the flanges, see figure 10.8. Forces R_1 and R_2 may be calculated using table 10.5. Force R2 shall also include loads parallel to the roof for sloped roofs. If R_1 is positive there is no tension force on the fastener. R_2 should be transferred from the sheeting to the top flange of the purlin and further on to the rafter (main beam) through the purlin/rafter connection (support cleat) or via special shear connectors or directly to the base or similar element. The reaction forces at an inner support of a continuous purlin may be taken as 2,2 times the values given in table 10.5.

NOTE: For sloped roofs the transversal loads to the purlins are the perpendicular (to the roof plane) components of the vertical loads and parallel components of the vertical loads are acting on the roof plane.

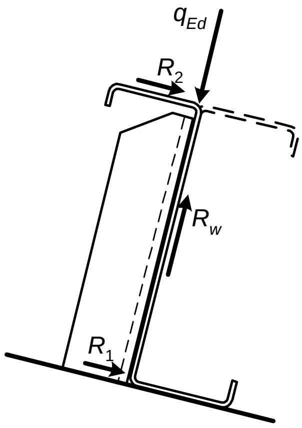


Figure 10.8: Reaction forces at support

Table 10.5 Reaction force at support for simply supported beam

Beam and loading	Reaction force on bottom flange R_1	Reaction force on top flange R_2
Z-beam, gravity loading	$(1-\varsigma)k_h q_{Ed} L/2$	$(1+\varsigma)k_h q_{Ed} L/2$
Z-beam, uplift loading	$-(1-\varsigma)k_h q_{Ed} L/2$	$-(1+\varsigma)k_h q_{Ed} L/2$
C-beam, gravity loading	$-(1-\varsigma)\, k_h q_{Ed} L/2$	$(1-\varsigma)\, k_h q_{Ed} L/2$
C-beam, uplift loading	$(1-\varsigma)\, k_h q_{Ed} L/2$	$-(1-\varsigma)\, k_h q_{Ed} L/2$

(3) The factor ζ may be taken as $\zeta = \sqrt[3]{\kappa_R}$, where κ_R = correction factor given in table 10.1, and the factor ξ may be taken as $\xi = \sqrt[3]{\zeta}$.

10.2 Liner trays restrained by sheeting

10.2.1 General

(1) Liner trays should be large channel-type sections, with two narrow flanges, two webs and one wide flange, generally as shown in figure 10.9. The two narrow flanges should be laterally restrained by attached profiled steel sheeting.

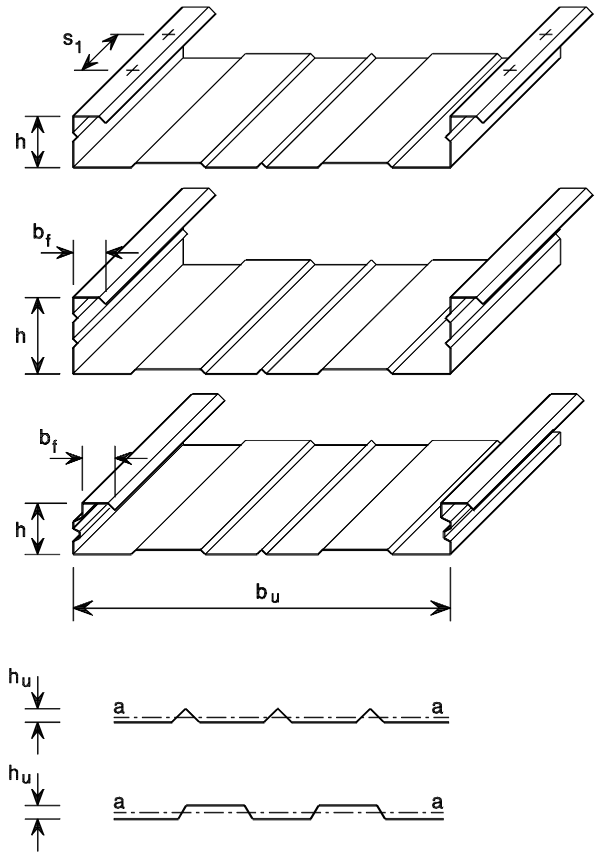


Figure 10.9: Typical geometry for liner trays

(2) The resistance of the webs of liner trays to shear forces and to local transverse forces should be obtained using 6.1.5 to 6.1.11, but using the value of $M_{c,Rd}$ given by (3) or (4).

(3) The moment resistance $M_{c,Rd}$ of a liner tray may be obtained using 10.2.2 provided that:

- the geometrical properties are within the range given in table 10.6;

- the depth h_u of the corrugations of the wide flange does not exceed $h/8$, where h is the overall depth of the liner tray.

(4) Alternatively the moment resistance of a liner tray may be determined by testing provided that it is ensured that the local behaviour of the liner tray is not affected by the testing equipment.

NOTE: Appropriate testing procedures are given in annex A.

Table 10.6: Range of validity of 10.2.2

0,75 mm	≤	t_{nom}	≤	1,5 mm
30 mm	≤	b_f	≤	60 mm
60 mm	≤	h	≤	200 mm
300 mm	≤	b_u	≤	600 mm
		I_a / b_u	≤	10 mm^4 / mm
		s_1	≤	1000 mm

10.2.2 Moment resistance

10.2.2.1 Wide flange in compression

(1) The moment resistance of a liner tray with its wide flange in compression should be determined using the step-by-step procedure outlined in figure 10.10 as follows:

- **Step 1:** Determine the effective areas of all compression elements of the cross-section, based on values of the stress ratio $\psi = \sigma_2 / \sigma_1$ obtained using the effective widths of the compression flanges but the gross areas of the webs;
- **Step 2:** Find the centroid of the effective cross-section, then obtain the moment resistance $M_{c,Rd}$ from:

$$M_{c,Rd} = 0{,}8\, W_{eff,min} f_{yb} / \gamma_{M0} \qquad \text{... (10.19)}$$

with:

$$W_{eff,min} = I_{y,eff} / z_c \quad \text{but} \quad W_{eff,min} \leq I_{y,eff} / z_t;$$

where z_c and z_t are as indicated in figure 10.10.

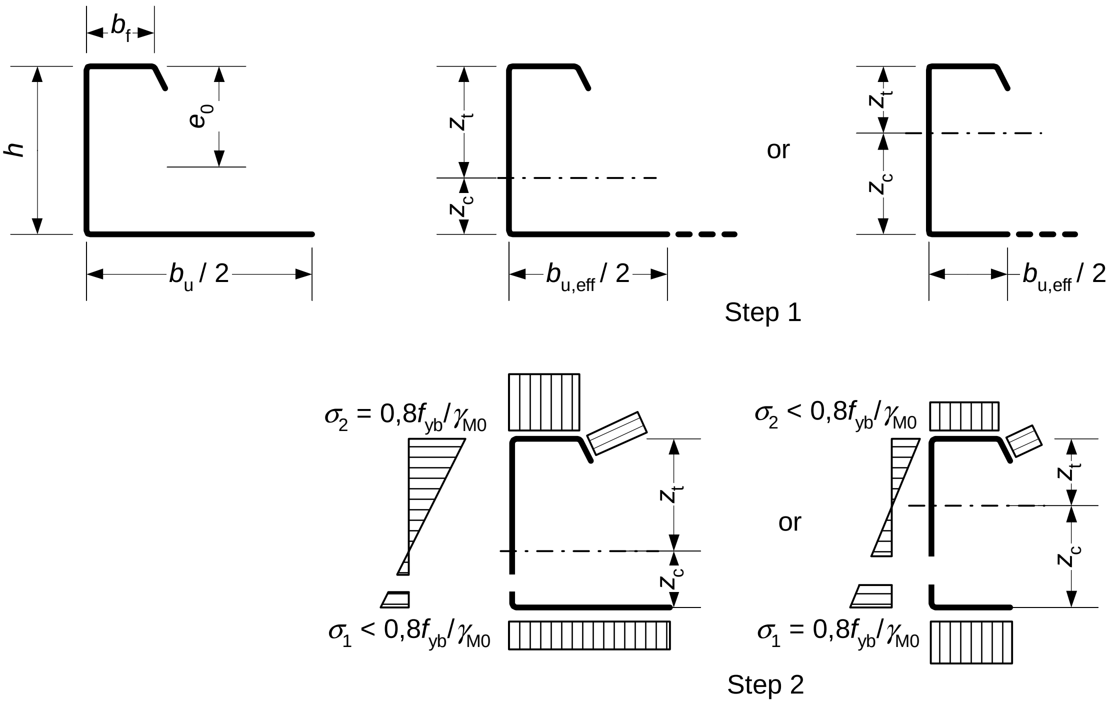


Figure 10.10: Determination of moment resistance — wide flange in compression

10.2.2.2 Wide flange in tension

(1) The moment resistance of a liner tray with its wide flange in tension should be determined using the step-by-step procedure outlined in figure 10.11 as follows:

- **Step 1:** Locate the centroid of the gross cross-section;
- **Step 2:** Obtain the effective width of the wide flange $b_{u,eff}$, allowing for possible flange curling, from:

$$b_{u,eff} = \frac{53{,}3 \cdot 10^{10}\, e_o^{\,2}\, t^3\, t_{eq}}{h\, L\, b_u^{\,3}} \qquad \text{... (10.20)}$$

where:

b_u is the overall width of the wide flange;

e_o is the distance from the centroidal axis of the gross cross-section to the centroidal axis of the narrow flanges;

h is the overall depth of the liner tray;

L is the span of the liner tray;

t_{eq} is the equivalent thickness of the wide flange, given by:

$$t_{eq} = (12\, I_a / b_u)^{1/3}$$

I_a is the second moment of area of the wide flange, about its own centroid, see figure 10.9.

- **Step 3:** Determine the effective areas of all the compression elements, based on values of the stress ratio $\psi = \sigma_2 / \sigma_1$ obtained using the effective widths of the flanges but the gross areas of the webs;
- **Step 4:** Find the centroid of the effective cross-section, then obtain the buckling resistance moment $M_{b,Rd}$ using:

$$M_{b,Rd} = 0{,}8\, \beta_b\, W_{eff,com} f_{yb} / \gamma_{M0} \quad \text{but} \quad M_{b,Rd} \leq 0{,}8\, W_{eff,t} f_{yb} / \gamma_{M0} \qquad \text{... (10.21)}$$

with:

$$W_{eff,com} = I_{y,eff} / z_c$$

$$W_{eff,t} = I_{y,eff} / z_t$$

in which the correlation factor β_b is given by the following:

- if $s_1 \leq 300$ mm:

$$\beta_b = 1{,}0$$

- if $300 \text{ mm} \leq s_1 \leq 1000$ mm:

$$\beta_b = 1{,}15 - s_1 / 2000$$

where:

s_1 is the longitudinal spacing of fasteners supplying lateral restraint to the narrow flanges, see figure 10.9.

(2) The effects of shear lag need not be considered if $L / b_{u,eff} \geq 20$. Otherwise a reduced value of ρ should be determined as specified in 6.1.4.3.

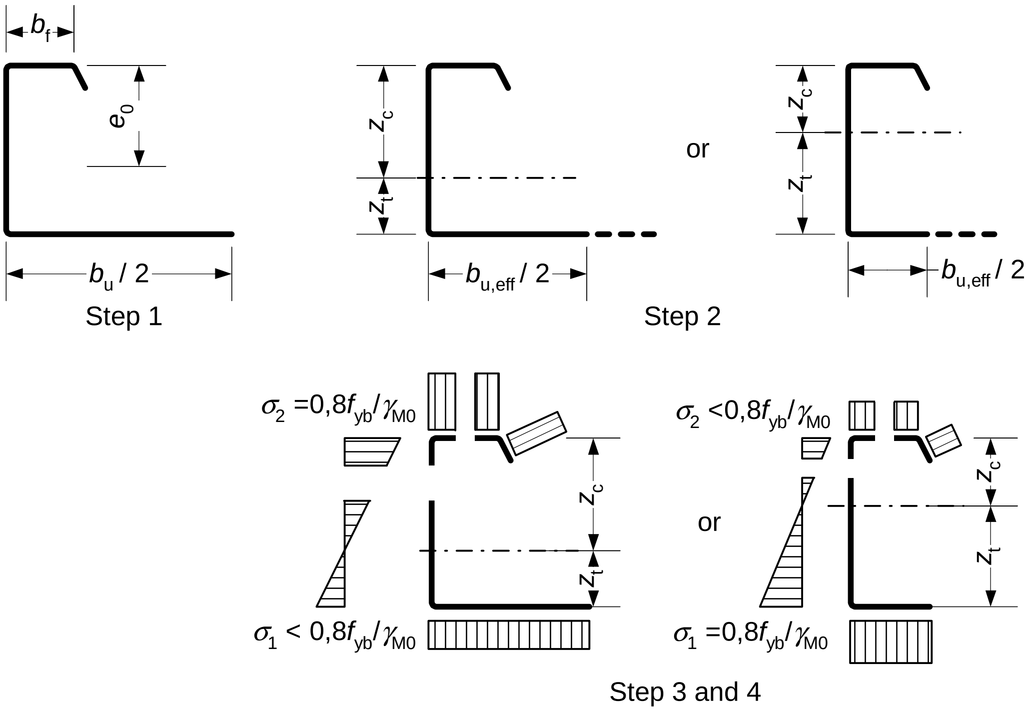


Figure 10.11: Determination of moment resistance — wide flange in tension

(3) Flange curling need not be taken into account in determining deflections at serviceability limit states.

(4) As a simplified alternative, the moment resistance of a liner tray with an unstiffened wide flange may be approximated by taking the same effective area for the wide flange in tension as for the two narrow flanges in compression combined.

10.3 Stressed skin design

10.3.1 General

(1) The interaction between structural members and sheeting panels that are designed to act together as parts of a combined structural system, may be allowed for as described in this clause 10.3.

(2) The provisions given in this clause shall be applied only to sheet diaphragms that are made of steel.

(3) Diaphragms may be formed from profiled sheeting used as roof or wall cladding or for floors. They may also be formed from wall or roof structures based upon liner trays.

NOTE: Information on the verification of such diaphragms may be obtained from:

ECCS Publication No. 88 (1995): *European recommendations for the application of metal sheeting acting as a diaphragm.*

10.3.2 Diaphragm action

(1) In stressed skin design, advantage may be taken of the contribution that diaphragms of sheeting used as roofing, flooring or wall cladding make to the overall stiffness and strength of the structural frame, by means of their stiffness and strength in shear.

(2) Roofs and floors may be treated as deep plate girders extending throughout the length of a building, resisting transverse in-plane loads and transmitting them to end gables, or to intermediate stiffened frames. The panel of sheeting may be treated as a web that resists in-plane transverse loads in shear, with the edge members acting as flanges that resist axial tension and compression forces, see figures 10.12 and 10.13.

(3) Similarly, rectangular wall panels may be treated as bracing systems that act as shear diaphragms to resist in-plane forces.

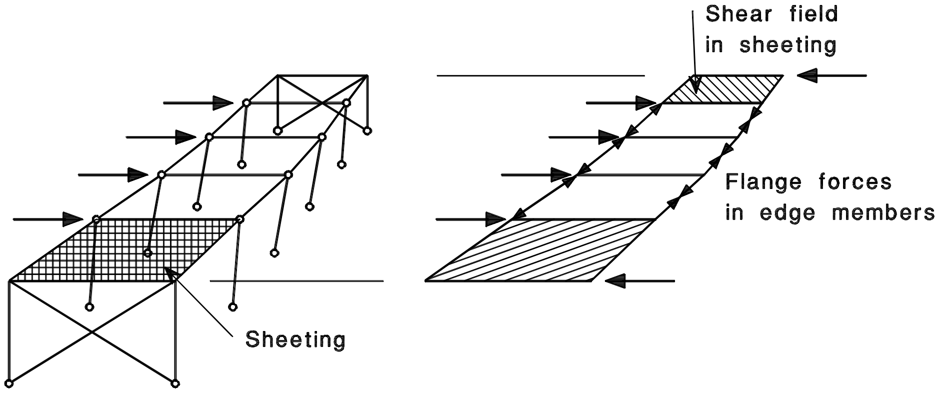


Figure 10.12: Stressed skin action in a flat-roof building

10.3.3 Necessary conditions

(1) Methods of stressed skin design that utilize sheeting as an integral part of a structure, may be used only under the following conditions:

- the use made of the sheeting, in addition to its primary purpose, is limited to the formation of shear diaphragms to resist structural displacement in the plane of that sheeting;
- the diaphragms have longitudinal edge members to carry flange forces arising from diaphragm action;
- the diaphragm forces in the plane of a roof or floor are transmitted to the foundations by means of braced frames, further stressed-skin diaphragms, or other methods of sway resistance;
- suitable structural connections are used to transmit diaphragm forces to the main steel framework and to join the edge members acting as flanges;
- the sheeting is treated as a structural component that cannot be removed without proper consideration;
- the project specification, including the calculations and drawings, draws attention to the fact that the building is designed to utilize stressed skin action;
- in sheeting with the corrugation oriented in the longitudinal direction of the roof the flange forces due to diaphragm action may be taken up by the sheeting.

(2) Stressed skin design may be used predominantly in low-rise buildings, or in the floors and facades of high-rise buildings.

(3) Stressed skin diaphragms may be used predominantly to resist wind loads, snow loads and other loads that are applied through the sheeting itself. They may also be used to resist small transient loads, such as surge from light overhead cranes or hoists on runway beams, but may not be used to resist permanent external loads, such as those from plant.

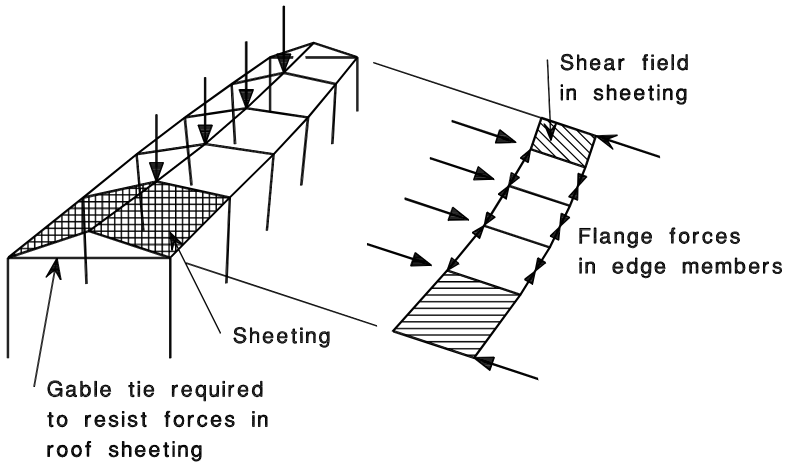


Figure 10.13: Stressed skin action in a pitched roof building

10.3.4 Profiled steel sheet diaphragms

(1) In a profiled steel sheet diaphragm, see figure 10.14, both ends of the sheets shall be attached to the supporting members by means of self-tapping screws, cartridge fired pins, welding, bolts or other fasteners of a type that will not work loose in service, pull out, or fail in shear before causing tearing of the sheeting. All such fasteners shall be fixed directly through the sheeting into the supporting member, for example through the troughs of profiled sheets, unless special measures are taken to ensure that the connections effectively transmit the forces assumed in the design.

(2) The seams between adjacent sheets should be fastened by rivets, self-drilling screws, welds, or other fasteners of a type that will not work loose in service, pull out, or fail in shear before causing tearing of the sheeting. The spacing of such fasteners should not exceed 500 mm.

(3) The distances from all fasteners to the edges and ends of the sheets shall be adequate to prevent premature tearing of the sheets.

(4) Small randomly arranged openings, up to 3% of the relevant area, may be introduced without special calculation, provided that the total number of fasteners is not reduced. Openings up to 15% of the relevant area (the area of the surface of the diaphragm taken into account for the calculations) may be introduced if justified by detailed calculations. Areas that contain larger openings should be split into smaller areas, each with full diaphragm action.

(5) All sheeting that also forms part of a stressed-skin diaphragm shall first be designed for its primary purpose in bending. To ensure that any deterioration of the sheeting would be apparent in bending before the resistance to stressed skin action is affected, it should then be verified that the shear stress due to diaphragm action does not exceed $0,25 f_{yb}/\gamma_{M1}$.

(6) The shear resistance of a stressed-skin diaphragm shall be based on the least tearing strength of the seam fasteners or the sheet-to-member fasteners parallel to the corrugations or, for diaphragms fastened only to longitudinal edge members, the end sheet-to-member fasteners. The calculated shear resistance for any other type of failure should exceed this minimum value by at least the following:

- for failure of the sheet-to-purlin fasteners under combined shear and wind uplift, by at least 40%;

- for any other type of failure, by at least 25%.

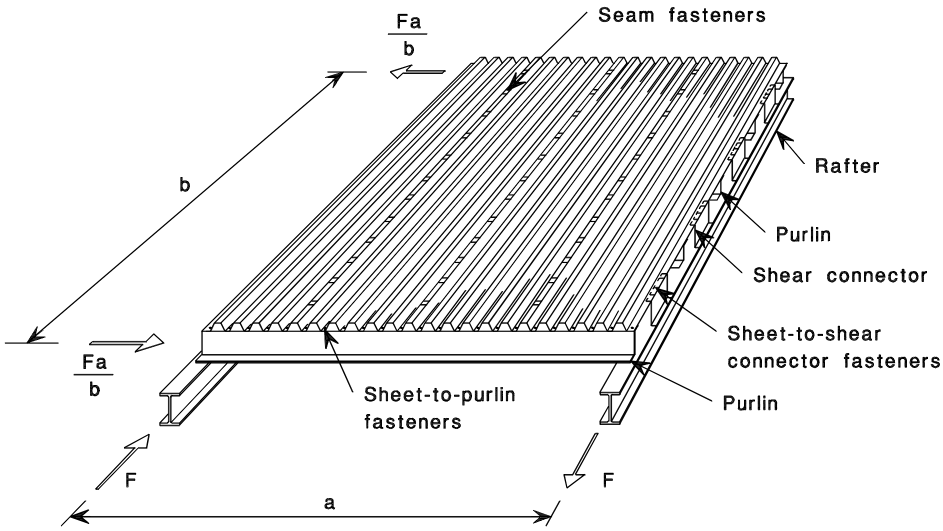


Figure 10.14: Arrangement of an individual panel

10.3.5 Steel liner tray diaphragms

(1) Liner trays used to form shear diaphragms should have stiffened wide flanges.

(2) Liner trays in shear diaphragms should be inter-connected by seam fasteners through the web at a spacing e_s of not more than 300 mm by seam fasteners (normally blind rivets) located at a distance e_u from the wide flange of not more than 30 mm, all as shown in figure 10.15.

(3) An accurate evaluation of deflections due to fasteners may be made using a similar procedure to that for trapezoidal profiled sheeting.

(4) The shear flow $T_{v,Ed}$ due to ultimate limit states design loads should not exceed $T_{v,Rd}$ given by:

$$T_{V,Rd} = 8{,}43\ E \sqrt[4]{I_a \left(t / b_u \right)^9} \quad \text{... (10.22)}$$

where:

I_a is the second moment of area of the wide flange about it own centroid, see figure 10.9;

b_u is the overall width of the wide flange.

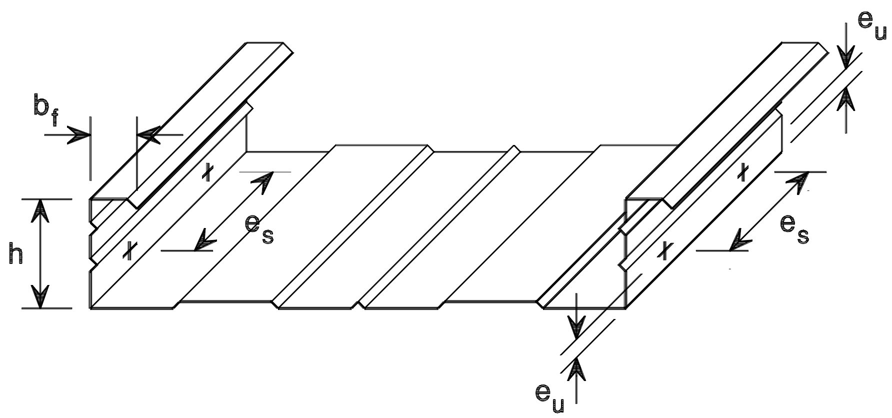


Figure 10.15: Location of seam fasteners

(5) The shear flow $T_{v,ser}$ due to serviceability design loads should not exceed $T_{v,Cd}$ given by:

$$T_{v,Cd} = S_v / 375 \qquad \text{... (10.23)}$$

where:

S_v is the shear stiffness of the diaphragm, per unit length of the span of the liner trays.

(6) The shear stiffness S_v per unit length may be obtained from:

$$S_V = \frac{\alpha\ L\ b_u}{e_s\ (b - b_u)} \qquad \text{... (10.24)}$$

where:

L is the overall length of the shear diaphragm (in the direction of the span of the liner trays);

b is the overall width of the shear diaphragm ($b = \Sigma\, b_u$);

α is the stiffness factor.

(7) The stiffness factor α may be conservatively be taken as equal to 2000 N/mm unless more accurate values are derived from tests.

10.4 Perforated sheeting

(1) Perforated sheeting may be designed by calculation, provided that the rules for non-perforated sheeting are modified by introducing the effective thicknesses given below.

NOTE: These calculation rules tend to give rather conservative values. More economical solutions might be obtained from design assisted by testing, see Section 9.

(2) Provided that $0{,}2 \leq d / a \leq 0{,}8$ gross section properties may be calculated using 5.1.2, but replacing t by $t_{a,eff}$ obtained from:

$$t_{a,eff} = 1{,}18\, t\, \left(1 - 0{,}9\, d\ / a\right) \qquad \text{... (10.25)}$$

where:

d is the diameter of the perforations;

a is the spacing between the centres of the perforations.

(3) Provided that $0{,}2 \leq d / a \leq 1{,}0$ effective section properties may be calculated using Section 4, but replacing t by $t_{b,eff}$ obtained from:

$$t_{b,eff} = t\ \sqrt[3]{1{,}18\left(1 - d\ / a\right)} \qquad \text{... (10.26)}$$

(4) The resistance of a single web to local transverse forces may be calculated using 6.1.9, but replacing t by $t_{c,eff}$ obtained from:

$$t_{c,eff} = t\ \left[1 - \left(d\ / a\right)^2 s_{per}\ / s_w\right]^{3/2} \qquad \text{... (10.27)}$$

where:

s_{per} is the slant height of the perforated portion of the web;

s_w is the total slant height of the web.

Annex A [normative] – Testing procedures

A.1 General

(1) This annex A gives appropriate standardized testing and evaluation procedures for a number of tests that are required in design.

NOTE 1: In the field of cold-formed members and sheeting, many standard products are commonly used for which design by calculation might not lead to economical solutions, so it is frequently desirable to use design assisted by testing.

NOTE 2: The National Annex may give further information on testing.

NOTE 3: The National Annex may give conversion factors for existing test results to be equivalent to the outcome of standardised tests according to this annex.

(2) This annex covers:

- tests on profiled sheets and liner trays, see A.2;

- tests on cold-formed members, see A.3;

- tests on structures and portions of structures, see A.4;

- tests on torsionally restrained beams, see A.5;

- evaluation of test results to determine design values, see A.6.

A.2 Tests on profiled sheets and liner trays

A.2.1 General

(1) Although these test procedures are presented in terms of profiled sheets, similar test procedures based on the same principles may also be used for liner trays and other types of sheeting (e.g. sheeting mentioned in EN 508).

(2) Loading may be applied through air bags or in a vacuum chamber or by steel or timber cross beams arranged to approximate uniformly distributed loading.

(3) To prevent spreading of corrugations, transverse ties or other appropriate test accessories such as timber blocks may be applied to the test specimen. Some examples are given in figure A.1.

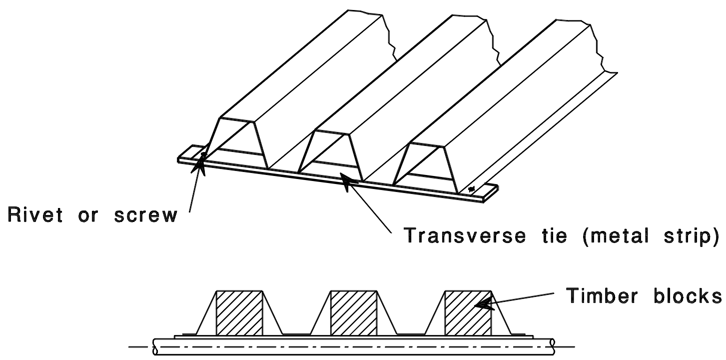


Figure A.1: Examples of appropriate test accessories

(4) For uplift tests, the test set-up should realistically simulate the behaviour of the sheeting under practical conditions. The type of connections between the sheet and the supports should be the same as in the connections to be used in practice.

(5) To give the results a wide range of applicability, hinged and roller supports should preferably be used, to avoid any influence of torsional restraint at the supports on the test results,

(6) It should be ensured that the direction of the loading remains perpendicular to the initial plane of the sheet throughout the test procedure.

(7) To eliminate the deformations of the supports, the deflections at both ends of the test specimen should also be measured.

(8) The test result should be taken as the maximum value of the loading applied to the specimen either coincident with failure or immediately prior to failure as appropriate.

A.2.2 Single span test

(1) A test set-up equivalent to that shown in figure A.2 may be used to determine the midspan moment resistance (in the absence of shear force) and the effective flexural stiffness.

(2) The span should be chosen such that the test results represent the moment resistance of the sheet.

(3) The moment resistance should be determined from the test result.

(4) The flexural stiffness should be determined from a plot of the load-deflection behaviour.

A.2.3 Double span test

(1) The test set-up shown in figure A.3 may be used to determine the resistance of a sheet that is continuous over two or more spans to combinations of moment and shear at internal supports, and its resistance to combined moment and support reaction for a given support width.

(2) The loading should preferably be uniformly distributed (applied using an air bag or a vacuum chamber, for example).

(3) Alternatively any number of line loads (transverse to the span) may be used, arranged to produce internal moments and forces that are appropriate to represent the effects of uniformly distributed loading. Some examples of suitable arrangements are shown in figure A.4.

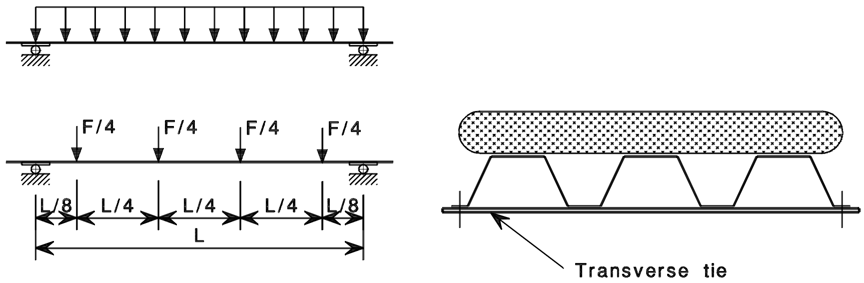
A.2.4 Internal support test

(1) As an alternative to A.2.3, the test set-up shown in figure A.5 may be used to determine the resistance of a sheet that is continuous over two or more spans to combinations of moment and shear at internal supports, and its resistance to combined moment and support reaction for a given support width.

(2) The test span s used to represent the portion of the sheet between the points of contraflexure each side of the internal support, in a sheet continuous over two equal spans L may be obtained from:

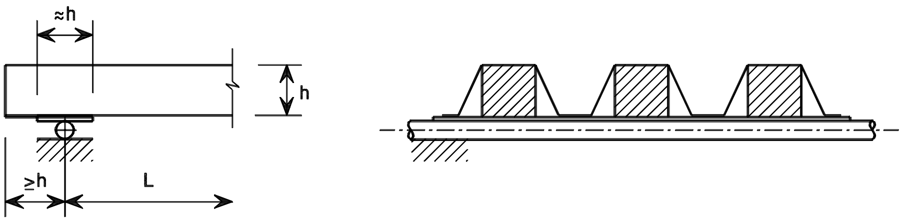
$$s = 0{,}4L \qquad \text{... (A.1)}$$

(3) If plastic redistribution of the support moment is expected, the test span s should be reduced to represent the appropriate ratio of support moment to shear force.

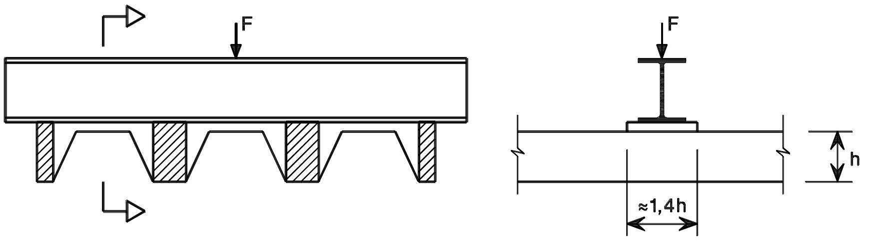


a) Uniformly distributed loading and an example of alternative equivalent line loads

b) Distributed loading applied by an airbag (alternatively by a vacuum test rig)



c) Example of support arrangements for preventing distortion



d) Example of method of applying a line load

Figure A.2: Test set-up for single span tests

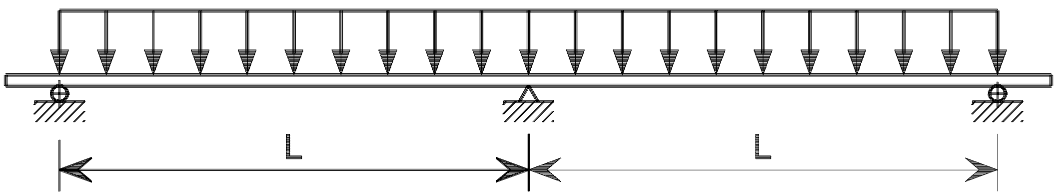


Figure A.3: Test set-up for double span tests

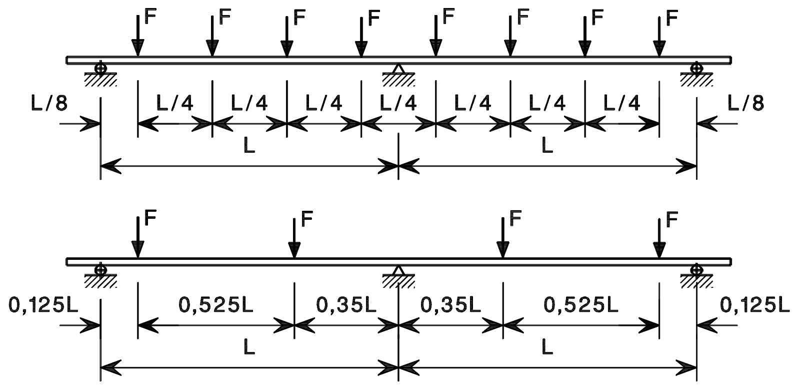


Figure A.4: Examples of suitable arrangements of alternative line loads

(4) The width b_B of the beam used to apply the test load should be selected to represent the actual support width to be used in practice.

(5) Each test result may be used to represent the resistance to combined bending moment and support reaction (or shear force) for a given span and a given support width. To obtain information about the interaction of bending moment and support reaction, tests should be carried out for several different spans.

(6) Interpretation of test results, see A.5.2.3.

A.2.5 End support test

(1) The test set-up shown in figure A.6 may be used to determine the shear resistance of a sheet at an end support.

(2) Separate tests should be carried out to determine the shear resistance of the sheet for different lengths u from the contact point at the inner edge of the end support, to the actual end of the sheet, see figure A.6.

NOTE: Value of maximum support reaction measured during a bending test may be used as a lower bound for section resistance to both shear and local transverse force.

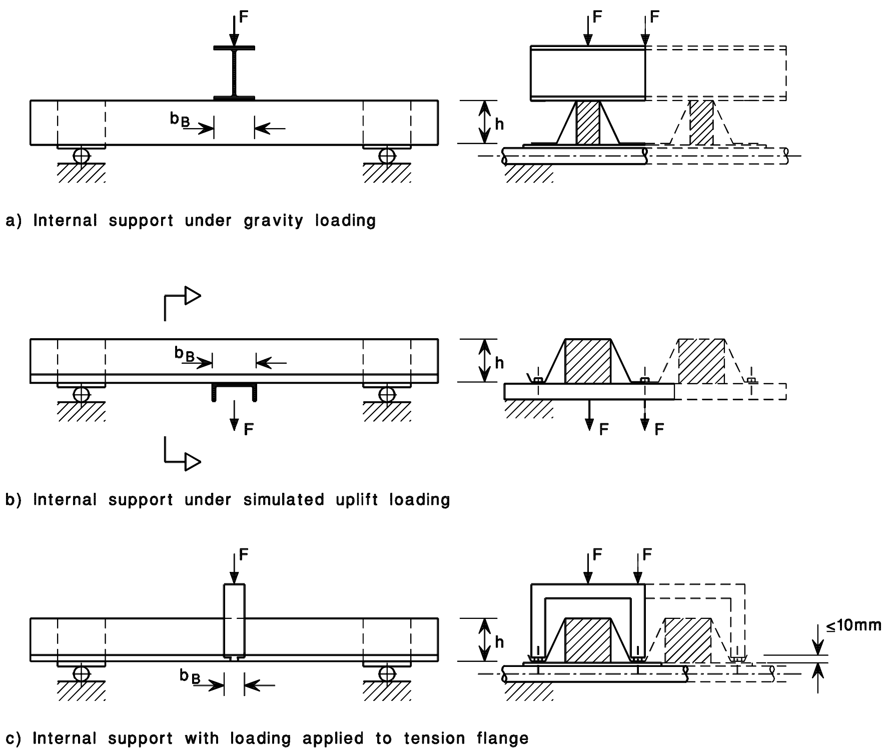
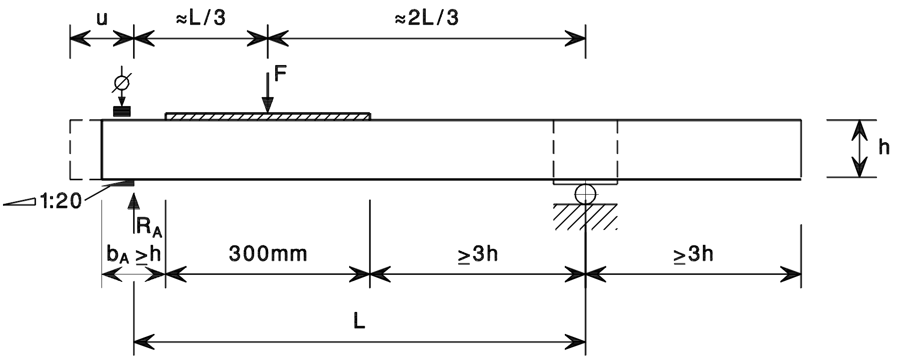


Figure A.5: Test set-up for internal support tests



Key:
b_A = support width
u = length from internal edge of end support to end of sheet

Figure A.6: Test set-up for end support tests

A.3 Tests on cold-formed members

A.3.1 General

(1) Each test specimen should be similar in all respects to the component or structure that it represents.

(2) The supporting devices used for tests should preferably provide end conditions that closely reproduce those supplied by the connections to be used in service. Where this cannot be achieved, less favourable end conditions that decrease the load carrying capacity or increase the flexibility should be used, as relevant.

(3) The devices used to apply the test loads should reproduce the way that the loads would be applied in service. It should be ensured that they do not offer more resistance to transverse deformations of the cross-section than would be available in the event of an overload in service. It should also be ensured that they do not localize the applied forces onto the lines of greatest resistance.

(4) If the given load combination includes forces on more than one line of action, each increment of the test loading should be applied proportionately to each of these forces.

(5) At each stage of the loading, the displacements or strains should be measured at one or more principal locations on the structure. Readings of displacements or strains should not be taken until the structure has completely stabilized after a load increment.

(6) Failure of a test specimen should be considered to have occurred in any of the following cases:

- at collapse or fracture;

- if a crack begins to spread in a vital part of the specimen;

- if the displacement is excessive.

(7) The test result should be taken as the maximum value of the loading applied to the specimen either coincident with failure or immediately prior to failure as appropriate.

(8) The accuracy of all measurements should be compatible with the magnitude of the measurement concerned and should in any case not exceed ± 1% of the value to be determined. The following magnitudes (in clause (9)) must also be fulfilled.

(9) The measurements of the cross-sectional geometry of the test specimen should include:

- the overall dimensions (width, depth and length) to an accuracy of ± 1,0 mm;

- widths of plane elements of the cross-section to an accuracy of ± 1,0 mm;

- radii of bends to an accuracy of ± 1,0 mm;

- inclinations of plane elements to an accuracy of ± 2,0°;

- angles between flat surfaces to an accuracy of $\pm 2{,}0°$;

- locations and dimensions of intermediate stiffeners to an accuracy of $\pm 1{,}0$ mm;

- the thickness of the material to an accuracy of $\pm 0{,}01$ mm;

- accuracy of all measurements of the cross-section has to be taken as equal to maximum 0,5 % of the nominal values.

(10)All other relevant parameters should also be measured, such as:

- locations of components relative to each other;

- locations of fasteners;

- the values of torques etc. used to tighten fasteners.

A.3.2 Full cross-section compression tests

A.3.2.1 Stub column test

(1) Stub column tests may be used to allow for the effects of local buckling in thin gauge cross-sections, by determining the value of the ratio $\beta_A = A_{eff} / A_g$ and the location of the effective centroidal axis.

(2) If local buckling of the plane elements governs the resistance of the cross-section, the specimen should have a length of at least 3 times the width of the widest plate element.

(3) The lengths of specimens with perforated cross-sections should include at least 5 pitches of the perforations, and should be such that the specimen is cut to length midway between two perforations.

(4) In the case of a cross-section with edge or intermediate stiffeners, it should be ensured that the length of the specimen is not less than the expected buckling lengths of the stiffeners.

(5) If the overall length of the specimen exceeds 20 times the least radius of gyration of its gross cross-section i_{min}, intermediate lateral restraints should be supplied at a spacing of not more than $20\, i_{min}$.

(6) Before testing, the tolerances of the cross-sectional dimensions of the specimen should be checked to ensure that they are within the permitted deviations.

(7) The cut ends of the specimen should be flat, and should be perpendicular to its longitudinal axis.

(8) An axial compressive force should be applied to each end of the specimen through pressure pads at least 30 mm thick, that protrude at least 10 mm beyond the perimeter of the cross-section.

(9) The test specimen should be placed in the testing machine with a ball bearing at each end. There should be small drilled indentations in the pressure pads to receive the ball bearings. The ball bearings should be located in line with the centroid of the calculated effective cross-section. If the calculated location of this effective centroid proves not to be correct, it may be adjusted within the test series.

(10)In the case of open cross-sections, possible spring-back may be corrected.

(11)Stub column tests may be used to determine the compression resistance of a cross-section. In interpreting the test results, the following parameters should be treated as variables:

- the thickness;

- the ratio b_p / t;

- the ratio f_u / f_{yb};

- the ultimate strength f_u and the yield strength f_{yb};

- the location of the centroid of the effective cross-section;

- imperfections in the shape of the elements of the cross-section;

- the method of cold forming (for example increasing the yield strength by introducing a deformation that is subsequently removed).

A.3.2.2 Member buckling test

(1) Member buckling tests may be used to determine the resistance of compression members with thin gauge cross-sections to overall buckling (including flexural buckling, torsional buckling and torsional-flexural buckling) and the interaction between local buckling and overall buckling.

(2) The method of carrying out the test should be generally as given for stub column tests in A.3.2.1.

(3) A series of tests on axially loaded specimens may be used to determine the appropriate buckling curve for a given type of cross-section and a given grade of steel, produced by a specific process. The values of relative slenderness $\overline{\lambda}$ to be tested and the minimum number of tests n at each value, should be as given in table A.1.

Table A.1: Relative slenderness values and numbers of tests

$\overline{\lambda}$	0,2	0,5	0,7	1,0	1,3	1,6	2,0	3,0
N	3	5	5	5	5	5	5	5

(4) Similar tests may also be used to determine the effect of introducing intermediate restraints on the torsional buckling resistance of a member.

(5) For the interpretation of the test results the following parameters should be taken into account:

- the parameters listed for stub column tests in A.3.2.1(11);

- overall lack of straightness imperfections compared to standard production output, see (6);

- type of end or intermediate restraint (flexural, torsional or both).

(6) Overall lack of straighness may be taken into account as follows:

a) Determine the critical compression load of the member by an appropriate analysis with initial bow equal to test sample: $F_{\mathrm{cr,bow,test}}$

b) As a) but with an initial bow equal to the maximum allowed according to the product specification: $F_{\mathrm{cr,bow,max,nom}}$

c) Additional correction factor: $F_{\mathrm{cr,bow,max,nom}} / F_{\mathrm{cr,bow,test}}$

A.3.3 Full cross-section tension test

(1) This test may be used to determine the average yield strength f_{ya} of the cross-section.

(2) The specimen should have a length of at least 5 times the width of the widest plane element in the cross-section.

(3) The load should be applied through end supports that ensure a uniform stress distribution across the cross-section.

(4) The failure zone should occur at a distance from the end supports of not less than the width of the widest plane element in the cross-section.

A.3.4 Full cross-section bending test

(1) This test may be used to determine the moment resistance and rotation capacity of a cross-section.

(2) The specimen should have a length of at least 15 times its greatest transversal dimension. The spacing of lateral restraints to the compression flange should not be less than the spacing to be used in service.

(3) A pair of point loads should be applied to the specimen to produce a length under uniform bending moment at midspan of at least $0{,}2 \times$ (span) but not more than $0{,}33 \times$ (span). These loads should be applied through the shear centre of the cross-section. If necessary, local buckling of the specimen should be prevented at the points of load application, to ensure that failure occurs within the central portion of the span. The deflection

should be measured at the load positions, at midspan and at the ends of the specimen.

(4) In interpreting the test results, the following parameters should be treated as variables:

- the thickness;
- the ratio b_p / t;
- the ratio f_u / f_{yb};
- the ultimate strength f_u and the yield strength f_{yb};
- differences between restraints used in the test and those available in service;
- the support conditions.

A.4 Tests on structures and portions of structures

A.4.1 Acceptance test

(1) This acceptance test may be used as a non-destructive test to confirm the structural performance of a structure or portion of a structure.

(2) The test load for an acceptance test should be taken as equal to the sum of:

- 1,0 $\times$ (the actual self-weight present during the test);
- 1,15 $\times$ (the remainder of the permanent load);
- 1,25 $\times$ (the variable loads).

but need not be taken as more than the mean of the total ultimate limit state design load and the total serviceability limit state design load for the characteristic (rare) load combination.

(3) Before carrying out the acceptance test, preliminary bedding down loading (not exceeding the characteristic values of the loads) may optionally be applied, and then removed.

(4) The structure should first be loaded up to a load equal to the total characteristic load. Under this load it should demonstrate substantially elastic behaviour. On removal of this load the residual deflection should not exceed 20% of the maximum recorded. If these criteria are not satisfied this part of the test procedure should be repeat. In this repeat load cycle, the structure should demonstrate substantially linear behaviour up to the characteristic load and the residual deflection should not exceed 10% of the maximum recorded.

(5) During the acceptance test, the loads should be applied in a number of regular increments at regular time intervals and the principal deflections should be measured at each stage. When the deflections show significant non-linearity, the load increments should be reduced.

(6) On the attainment of the acceptance test load, the load should be maintained for being no changes between a set of adjacent readings and deflection measurements should be taken to establish whether the structure is subject to any time-dependent deformations, such as deformations of fasteners or deformations arising from creep in the zinc layer.

(7) Unloading should be completed in regular decrements, with deflection readings taken at each stage.

(8) The structure should prove capable of sustaining the acceptance test load, and there should be no significant local distortion or defects likely to render the structure unserviceable after the test.

A.4.2 Strength test

(1) This strength test may be used to confirm the calculated load carrying capacity of a structure or portion of a structure. Where a number of similar items are to be constructed to a common design, and one or more prototypes have been submitted to and met all the requirements of this strength test, the others may be accepted without further testing provided that they are similar in all relevant respects to the prototypes.

(2) Before carrying out a strength test the specimen should first pass the acceptance test detailed in A.4.1.

(3) The load should then be increased in increments up to the strength test load and the principal deflections should be measured at each stage. The strength test load should be maintained for at least one hour and deflection measurements should be taken to establish whether the structure is subject to creep.

(4) Unloading should be completed in regular decrements with deflection readings taken at each stage.

(5) The total test load (including self-weight) for a strength test F_{str} should be determined from the total design load F_{Ed} specified for ultimate limit state verifications by calculation, using:

$$F_{str} = \gamma_M \mu_F F_{Ed} \qquad \text{... (A.2)}$$

in which μ_F is the load adjustment coefficient and γ_M is the partial coefficient of the ultimate limit state.

(6) The load adjustment coefficient μ_F should take account of variations in the load carrying capacity of the structure, or portion of a structure, due to the effects of variation in the material yield strength, local buckling, overall buckling and any other relevant parameters or considerations.

(7) Where a realistic assessment of the load carrying capacity of the structure, or portion of a structure, may be made using the provisions of this Part 1-3 of EN 1993 for design by calculation, or another proven method of analysis that takes account of all buckling effects, the load adjustment coefficient μ_F may be taken as equal to the ratio of (the value of the assessed load carrying capacity based on the averaged basic yield strength f_{ym}) compared to (the corresponding value based on the nominal basic yield strength f_{yb}).

(8) The value of f_{ym} should be determined from the measured basic strength $f_{yb,obs}$ of the various components of the structure, or portion of a structure, with due regard to their relative importance.

(9) If realistic theoretical assessments of the load carrying capacity cannot be made, the load adjustment coefficient μ_F should be taken as equal to the resistance adjustment coefficient μ_R defined in A.6.2.

(10)Under the test load there should be no failure by buckling or rupture in any part of the specimen.

(11)On removal of the test load, the deflection should be reduced by at least 20%.

A.4.3 Prototype failure test

(1) A test to failure may be used to determine the real mode of failure and the true load carrying capacity of a structure or assembly. If the prototype is not required for use, it may optionally be used to obtain this additional information after completing the strength test described in A.4.2.

(2) Alternatively a test to failure may be carried out to determine the true design load carrying capacity from the ultimate test load. As the acceptance and strength test procedures should preferably be carried out first, an estimate should be made of the anticipated design load carrying capacity as a basis for such tests.

(3) Before carrying out a test to failure, the specimen should first pass the strength test described in A.4.2. Its estimated design load carrying capacity may then be adjusted based on its behaviour in the strength test.

(4) During a test to failure, the loading should first be applied in increments up to the strength test load. Subsequent load increments should then be based on an examination of the plot of the principal deflections.

(5) The ultimate load carrying capacity should be taken as the value of the test load at that point at which the structure or assembly is unable to sustain any further increase in load.

NOTE: At this point gross permanent distortion is likely to have occurred. In some cases gross deformation might define the test limit.

A.4.4 Calibration test

(1) A calibration test may be used to:

- verify load bearing behaviour relative to analytical design models;
- quantify parameters derived from design models, such as strength or stiffness of members or joints.

A.5 Tests on torsionally restrained beams

A.5.1 General

(1) These test procedures may be used for beams that are partially restrained against torsional displacement, by means of trapezoidal profiled steel sheeting or other suitable cladding.

(2) These procedures may be used for purlins, side rails, floor beams and other similar types of beams that have relevant restraint conditions.

A.5.2 Internal support test

A.5.2.1 Test set-up

(1) The test set-up shown in figure A.7 may be used to determine the resistance of a beam that is continuous over two or more spans, to combinations of bending moment and shear force at internal supports.

NOTE: The same test set-up may be used for sleeved and overlap systems.

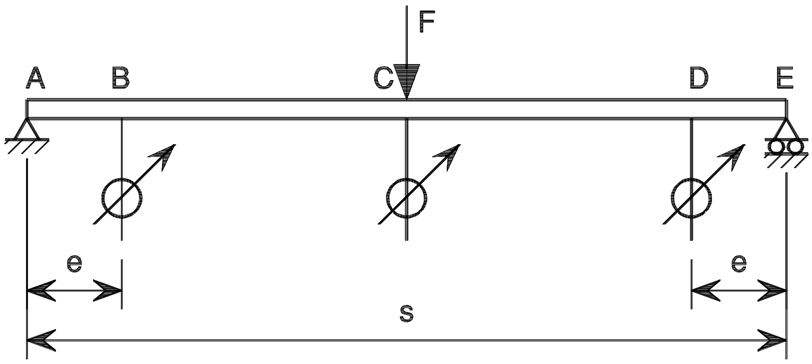


Figure A.7: Test set-up for internal support tests

(2) The supports at **A** and **E** should be hinged and roller supports respectively. At these supports, rotation about the longitudinal axis of the beam may be prevented, for example by means of cleats.

(3) The method of applying the load at **C** should correspond with the method to be used in service.

NOTE: In many cases this will mean that lateral displacement of both flanges is prevented at **C**.

(4) The displacement measurements at points **B** and **D** located at a distance e from each support, see figure A.7, should be recorded to allow these displacements to be eliminated from the results analysis

(5) The test span s should be chosen to produce combinations of bending moment and shear force that represent those expected to occur in practical application under the design load for the relevant limit state.

(6) For double span beams of span L subject to uniformly distributed loads, the test span s should normally be taken as equal to $0,4L$. However, if plastic redistribution of the support moment is expected, the test span s should be reduced to represent the appropriate ratio of support moment to shear force.

A.5.2.2 Execution of tests

(1) In addition to the general rules for testing, the following specific aspects should be taken into account.

(2) Testing should continue beyond the peak load and the recording of the deflections should be continued either until the applied load has reduced to between 10% and 15% of its peak value or until the deflection has reached a value 6 times the maximum elastic displacement.

A.5.2.3 Interpretation of test results

(1) The actual measured test results $R_{obs,i}$ should be adjusted as specified in A.6.2 to obtain adjusted values $R_{adj,i}$ related to the nominal basic yield strength f_{yb} and design thickness t of the steel, see 3.2.4.

(2) For each value of the test span s the support reaction R should be taken as the mean of the adjusted values of the peak load F_{max} for that value of s. The corresponding value of the support moment M should then be determined from:

$$M = \frac{s\ R}{4} \qquad \text{... (A.3)}$$

Generally the influence of the dead load should be added when calculating the value of moment M following the expression (A.3).

(3) The pairs of values of M and R for each value of s should be plotted as shown in figure A.8. Pairs of values for intermediate combinations of M and R may then be determined by linear interpolation.

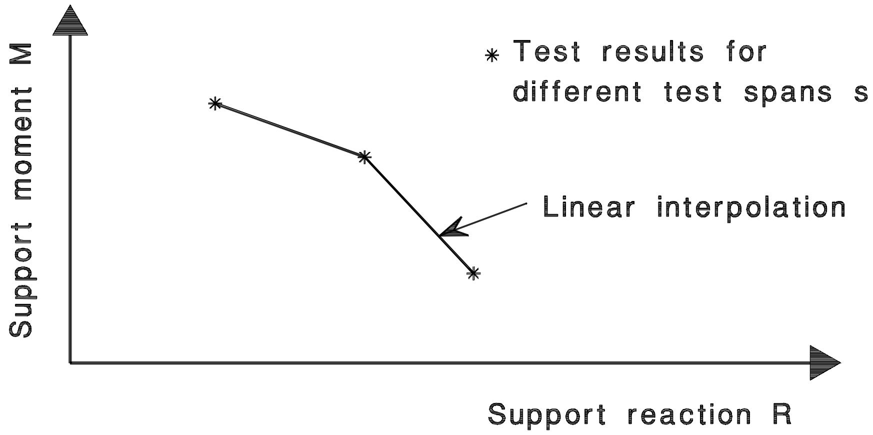


Figure A.8: Relation between support moment and support reaction

(4) The net deflection at the point of load application **C** in figure A.7 should be obtained from the gross measured values by deducting the mean of the corresponding deflections measured at the points **B** and **D** located at a distance e from the support points **A** and **E**, see figure A.7.

(5) For each test the applied load should be plotted against the corresponding net deflection, see figure A.9. From this plot, the rotation θ should be obtained for a range of values of the applied load using:

$$\theta = \frac{2(\delta_{pl} - \delta_e - \delta_{el})}{0{,}5\ s - e} \qquad \text{... (A.4a)}$$

$$\theta = \frac{2(\delta_{pl} - \delta_e - \delta_{lin})}{0{,}5\ s - e} \qquad \text{... (A.4b)}$$

where:

δ_{el} is the net deflection for a given load on the rising part of the curve, before F_{max};

δ_{pl} is the net deflection for the same load on the falling part of the curve, after F_{max};

δ_{lin} is the fictive net deflection for a given load, that would be obtained with a linear behaviour, see figure A.9;

δ_e is the average deflection measured at a distance e from the support, see figure A.7;

s is the test span;

e is the distance between a deflection measurement point and a support, see figure A.7.

The expression (A.4a) is used when analyses are done based on the effective cross-section. The expression (A.4b) is used when analyses are done based on the gross cross-section.

(6) The relationship between M and θ should then be plotted for each test at a given test span s corresponding to a given value of beam span L as shown in figure A.10. The design M - θ characteristic for the moment resistance of the beam over an internal support should then be taken as equal to 0,9 times the mean value of M for all the tests corresponding to that value of the beam span L.

NOTE: Smaller value than 0,9 for reduction should be used, if the full-scale tests are used to determine effect of lateral load and buckling of free flange around the mid-support, see 10.1.3.2(4).

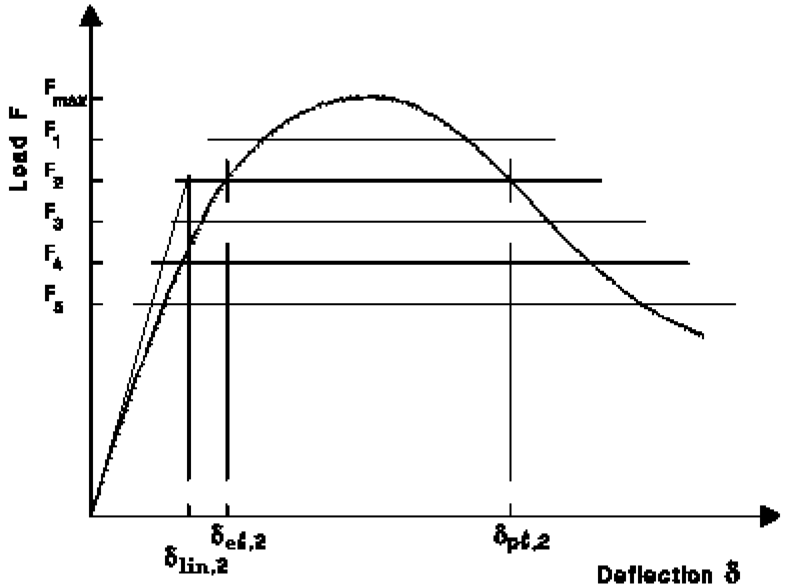


Figure A.9: Relationship between load and net deflection

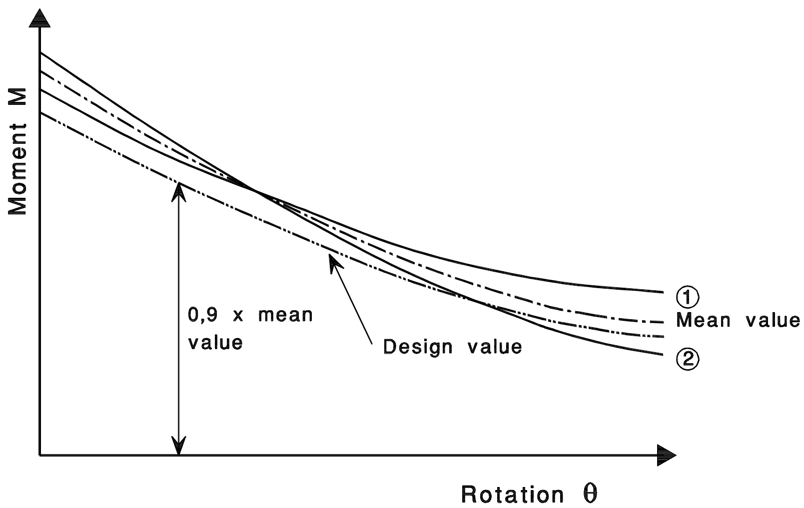


Figure A.10: Derivation of the design moment-rotation characteristic

A.5.3 Determination of torsional restraint

(1) The test set-up shown in figure A.11 may be used to determine the amount of torsional restraint given by adequately fastened sheeting or by another member perpendicular to the span of the beam.

(2) This test set-up covers two different contributions to the total amount of restraint as follows:

a) The lateral stiffness K_A per unit length corresponding to the rotational stiffness of the connection between the sheeting and the beam;

b) The lateral stiffness K_B per unit length due to distortion of the cross-section of the purlin.

(3) The combined restraint per unit length may be determined from:

$$\left(1 / K_A + 1 / K_B\right) = \delta / F \qquad \text{... (A.5)}$$

where:

F is the load per unit length of the test specimen to produce a lateral deflection of $h/10$;

h is the overall depth of the specimen;

δ is the lateral displacement of the top flange in the direction of the load F.

(4) In interpreting the test results, the following parameters should be treated as variables:

- the number of fasteners per unit length of the specimen;
- the type of fasteners;
- the flexural stiffness of the beam, relative to its thickness;
- the flexural stiffness of the bottom flange of the sheeting, relative to its thickness;
- the positions of the fasteners in the flange of the sheeting;
- the distance from the fasteners to the centre of rotation of the beam;
- the overall depth of the beam;
- the possible presence of insulation between the beam and the sheeting.

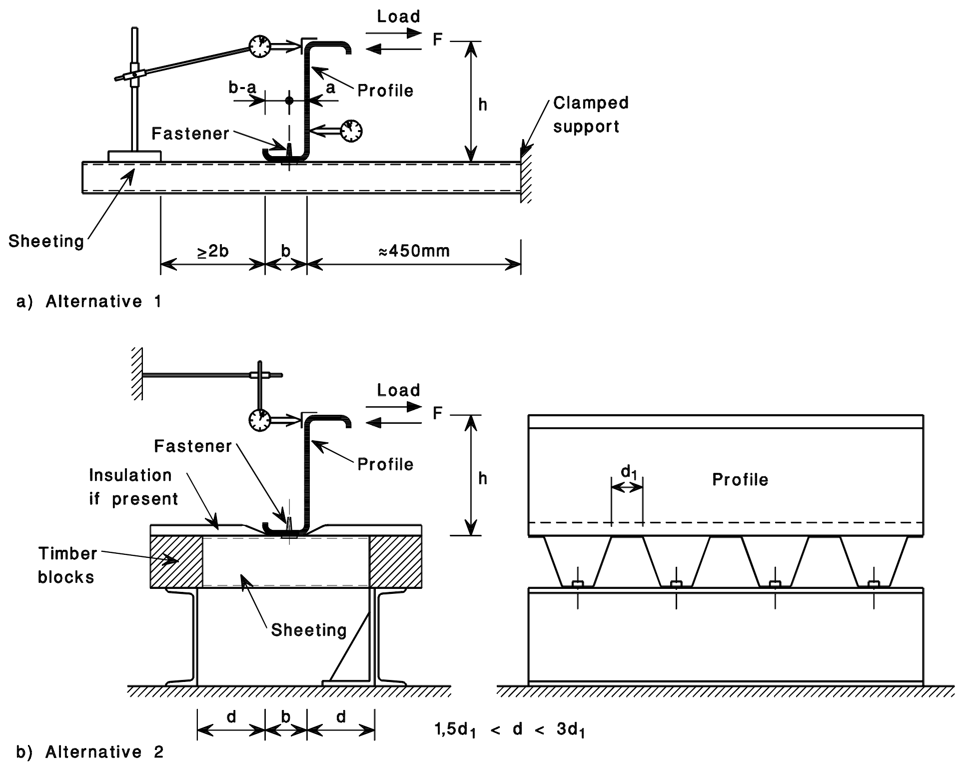


Figure A.11: Experimental determination of spring stiffnesses K_A and K_B

A.6 Evaluation of test results

A.6.1 General

(1) A specimen under test should be regarded as having failed if the applied test loads reach their maximum values, or if the gross deformations exceed specified limits.

(2) The gross deformations of members should generally satisfy:

$$\delta \leq L/50 \quad \text{... (A.6)}$$

$$\phi \leq 1/50 \quad \text{... (A.7)}$$

where:

δ is the maximum deflection of a beam of span L;

ϕ is the sway angle of a structure.

(3) In the testing of connections, or of components in which the examination of large deformations is necessary for accurate assessment (for example, in evaluating the moment-rotation characteristics of sleeves), no limit need be placed on the gross deformation during the test.

(4) An appropriate margin of safety should be available between a ductile failure mode and possible brittle failure modes. As brittle failure modes do not usually appear in large scale tests, additional detail tests should be carried out where necessary.

NOTE: This is often the case for connections.

A.6.2 Adjustment of test results

(1) Test results should be appropriately adjusted to allow for variations between the actual measured properties of the test specimens and their nominal values.

(2) The actual measured basic yield strength $f_{yb,obs}$ should not deviate by more than -25% from the nominal basic yield strength f_{yb} i.e. $f_{yb,obs} \geq 0{,}75\, f_{yb.}$

(3) The actual measured thickness t_{obs} should not exceed the nominal material thickness t_{nom} (see 3.2.4) by more than 12%.

(4) Adjustments should be made in respect of the actual measured values of the core material thickness $t_{obs,cor}$ and the basic yield strength $f_{yb,obs}$ for all tests, except if values measured in tests are used to calibrate a design model then provisions of (5) need not be applied.

(5) The adjusted value $R_{adj,i}$ of the test result for test i should be determined from the actual measured test result $R_{obs,i}$ using:

$$R_{adj,i} = R_{obs,i} / \mu_R \quad \text{... (A.8)}$$

in which μ_R is the resistance adjustment coefficient given by:

$$\mu_R = \left(\frac{f_{yb,obs}}{f_{yb}}\right)^{\alpha} \left(\frac{t_{obs,cor}}{t_{cor}}\right)^{\beta} \quad \text{... (A.9)}$$

(6) The exponent α for use in expression (A.9) should be obtained as follows:

- if $f_{yb,obs} \leq f_{yb}$: $\alpha = 0$
- if $f_{yb,obs} > f_{yb}$:
  - generally: $\alpha = 1$

For profiled sheets or liner trays in which compression elements have such large b_p / t ratios that local buckling is clearly the failure mode: $\alpha = 0{,}5$.

(7) The exponent β for use in expression (A.9) should be obtained as follows:

- if $t_{obs,cor} \leq t_{cor}$: $\beta = 1$
- if $t_{obs,cor} > t_{cor}$:
  - for tests on profiled sheets or liner trays: $\beta = 2$
- for tests on members, structures or portions of structures:
  - if $b_p / t \leq (b_p / t)_{lim}$: $\beta = 1$
  - if $b_p / t > 1{,}5(b_p / t)_{lim}$: $\beta = 2$
  - if $(b_p / t)_{lim} < b_p / t < 1{,}5(b_p / t)_{lim}$: obtain β by linear interpolation.

in which the limiting width-to thickness ratio $(b_p / t)_{lim}$ given by:

$$(b_p / t)_{lim} = 0{,}64 \sqrt{\frac{E\, k_\sigma}{f_{yb}}} \cdot \sqrt{\frac{f_{yb} / \gamma_{M1}}{\sigma_{com,Ed}}} \cong 19{,}1 \varepsilon \sqrt{k_\sigma} \cdot \sqrt{\frac{f_{yb} / \gamma_{M1}}{\sigma_{com,Ed}}} \quad \text{... (A.10)}$$

where:

b_p is the notional flat width of a plane element;

k_σ is the relevant buckling factor from table 5.3 or 5.4;

$\sigma_{com,Ed}$ is the largest calculated compressive stress in that element, when the resistance of the cross-section is reached.

NOTE: In the case of available test report concerning sheet specimens with $t_{obs,cor} / t_{cor} \leq 1{,}06$ readjustment of existing value not exceeding 1,02 times the $R_{adj,i}$ value according to A.6.2 may be ommitted.

A.6.3 Characteristic values

A.6.3.1 General

(1) Characteristic values may be determined statistically, provided that there are at least 4 test results.

NOTE: A larger number is generally preferable, particularly if the scatter is relatively wide.

(2) If the number of test results available is 3 or less, the method given in A.6.3.3 may be used.

(3) The characteristic minimum value should be determined using the following provisions. If the characteristic maximum value or the characteristic mean value is required, it should be determined by using appropriate adaptations of the provisions given for the characteristic minimum value.

(4) The characteristic value R_k determined on the basis of at least 4 tests may be obtained from:

$$R_k = R_m \pm k\, s \quad \text{... (A.11)}$$

where:

s is the standard deviation;

k is the appropriate coefficient from table A.2;

R_m is the mean value of the adjusted test results R_{adj};

The unfavourable sign "+" or "-" shall be adopted for given considered value.

NOTE: As general rule, for resistance characteristic value, the sign "-" should be taken and e.g. for rotation characteristic value, both are to be considered.

(5) The standard deviation s may be determined using:

$$s = \left[\sum_{i=1}^{n} \left(R_{\text{adj.i}} - R_{\text{m}} \right)^2 \Big/ (n-1) \right]^{0,5} \equiv \left[\left[\sum_{i=1}^{n} \left(R_{\text{adj.i}} \right)^2 - (1/n) \left(\sum_{i=1}^{n} R_{\text{adj.i}} \right)^2 \right] \Big/ (n-1) \right]^{0,5} \quad \text{... (A.12)}$$

where:

$R_{\text{adj,i}}$ is the adjusted test result for test i ;

n is the number of tests.

Table A.2: Values of the coefficient k

N	4	5	6	8	10	20	30	∞
k	2,63	2,33	2,18	2,00	1,92	1,76	1,73	1,64

A.6.3.2 Characteristic values for families of tests

(1) A series of tests carried out on a number of otherwise similar structures, portions of structures, members, sheets or other structural components, in which one or more parameters is varied, may be treated as a single family of tests, provided that they all have the same failure mode. The parameters that are varied may include cross-sectional dimensions, spans, thicknesses and material strengths.

(2) The characteristic resistances of the members of a family may be determined on the basis of a suitable design expression that relates the test results to all the relevant parameters. This design expression may either be based on the appropriate equations of structural mechanics, or determined on an empirical basis.

(3) The design expression should be modified to predict the mean measured resistance as accurately as practicable, by adjusting the coefficients to optimize the correlation.

NOTE: Information on this process is given Annex D of EN 1990.

(4) In order to calculate the standard deviation s each test result should first be normalized by dividing it by the corresponding value predicted by the design expression. If the design expression has been modified as specified in (3), the mean value of the normalized test results will be unity. The number of tests n should be taken as equal to the total number of tests in the family.

(5) For a family of at least four tests, the characteristic resistance R_{k} should then be obtained from expression (A.11) by taking R_{m} as equal to the value predicted by the design expression, and using the value of k from table A.2 corresponding to a value of n equal to the total number of tests in the family.

A.6.3.3 Characteristic values based on a small number of tests

(1) If only one test is carried out, then the characteristic resistance R_{k} corresponding to this test should be obtained from the adjusted test result R_{adj} using:

$$R_{\text{k}} = 0,9\, \eta_{\text{k}} R_{\text{adj}} \quad \text{... (A.13)}$$

in which η_{k} should be taken as follows, depending on the failure mode:

- yielding failure: $\eta_{\text{k}} = 0,9$;
- gross deformation: $\eta_{\text{k}} = 0,9$;
- local buckling: $\eta_{\text{k}} = 0,8$ ... 0,9 depending on effects on global behaviour in tests ;
- overall instability: $\eta_{\text{k}} = 0,7$.

(2) For a family of two or three tests, provided that each adjusted test result $R_{adj,i}$ is within $\pm 10\%$ of the mean value R_m of the adjusted test results, the characteristic resistance R_k should be obtained using:

$$R_k = \eta_k R_m \qquad \text{... (A.14)}$$

(3) The characteristic values of stiffness properties (such as flexural or rotational stiffness) may be taken as the mean value of at least two tests, provided that each test result is within $\pm 10\%$ of the mean value.

(4) In the case of one single test the characteristic value of the stiffness is reduced by 0,95 for favourable value and increased by 1,05 for non-favourable value.

A.6.4 Design values

(1) The design value of a resistance R_d should be derived from the corresponding characteristic value R_k determined by testing, using:

$$R_d = \eta_{sys} \frac{R_k}{\gamma_M} \qquad \text{... (A.15)}$$

where:

γ_M is the partial factor for resistance;

η_{sys} is a conversion factor for differences in behaviour under test conditions and service conditions.

(2) The appropriate value for η_{sys} should be determined in dependance of the modelling for testing.

(3) For sheeting and for other well defined standard testing procedures (including A.3.2.1 stub column tests, A.3.3 tension tests and A.3.4 bending tests) η_{sys} may be taken as equal to 1,0. For tests on torsionally restrained beams conformed to the section A.5, η_{sys} = 1,0 may also be taken.

(4) For other types of tests in which possible instability phenomena, or modes of behaviour, of structures or structural components might not be covered sufficiently by the tests, the value of η_{sys} should be assessed taking into account the actual testing conditions, in order to achieve the necessary reliability.

NOTE: The partial factor γ_M may be given in the National Annex. It is recommended to use the γ_M-values as chosen in the design by calculation given in section 2 or section 8 of this part unless other values result from the use of Annex D of EN 1990.

A.6.5 Serviceability

(1) The provisions given in Section 7 should be satisfied.

Annex B [informative] – Durability of fasteners

B.1 Durability of fasteners

(1) In Construction Classes I, II and III table B.1 may be applied.

Table B.1: Fastener material with regard to corrosion environment (and sheeting material only for information). Only the risk of corrosion is considered. Classification of environment according to EN ISO 12944-2.

Classification of environment	**Sheet material**	**Material of fastener**					
		Aluminium	Electro galvanized steel. Coat thickness > 7μm	Hot-dip zinc coated steel[b]. Coat thickness >45μm	Stainless steel, case hardened. 1.4006 [d]	Stainless steel, 1.4301 [d] 1.4436 [d]	Monel[a]
C1	A, B, C	X	X	X	X	X	X
	D, E, S	X	X	X	X	X	X
C2	A	X	-	X	X	X	X
	C, D, E	X	-	X	X	X	X
	S	X	-	X	X	X	X
C3	A	X	-	X	-	X	X
	C, E	X	-	X	(X)[C]	(X)[C]	-
	D	X	-	X	-	(X)[C]	X
	S	-	-	X	X	X	X
C4	A	X	-	(X)[C]	-	(X)[C]	-
	D	-	-	X	-	(X)[C]	-
	E	X	-	X	-	(X)[C]	-
	S	-	-	X	-	X	X
C5-I	A	X	-	-	-	(X)[C]	-
	D[f]	-	-	X	-	(X)[C]	-
	S	-	-	-	-	X	-
C5-M	A	X	-	-	-	(X)[C]	-
	D[f]	-	-	X	-	(X)[C]	-
	S	-	-	-	-	X	-

Anm. Fastener of steel without coating may be used in corrosion classification class C1.

A = Aluminium irrespective of surface finish
B = Un-coated steel sheet
C = Hot-dip zinc coated (Z275) or aluzink coated (AZ150) steel sheet
D = Hot-dip zinc coated steel sheet + coating of paint or plastics
E = Aluzink coated (AZ185) steel sheet
S = Stainless steel
X = Type of material recommended from the corrosion standpoint
(X) = Type of material recommended from the corrosion standpoint under the specified condition only
- = Type of material not recommended from the corrosion standpoint
a Refers to rivets only
b Refers to screws and nuts only
c Insulating washer, of material resistant to ageing, between sheeting and fastener
d Stainless steel EN 10 088
e Risk of discoloration.
f Always check with sheet supplier

(2) The environmental classification following EN-ISO 12944-2 is presented in table B.2.

Table B.2: Atmospheric-corrosivity categories according to EN ISO 12944-2 and examples of typical environments

Corro-sivity category	Corro-sivity level	Examples of typical environments in a temperate climate (informative))	
		Exterior	Interior
C1	Very low	-	Heated buildings with clean atmospheres, e. g. offices, shops, schools and hotels.
C2	Low	Atmospheres with low level of pollution. Mostly rural areas	Unheated buildings where condensation may occur, e. g. depots, sport halls.
C3	Medium	Urban and industrial atmospheres, moderate sulphur dioxide pollution. Coastal areas with low salinity.	Production rooms with high humidity and some air pollution, e. g. food-processing plants, laundries, breweries and dairies.
C4	High	Industrial areas and coastal areas with moderate salinity.	Chemical plants, swimming pools, coastal ship- and boatyards.
C5-I	Very high (in-dustrial)	Industrial areas with high humidity and aggressive atmosphere.	Building or areas with almost permanent condensation and with high pollution.
C5-M	Very high (marine)	Coastal and offshore areas with high salinity.	Building or areas with almost permanent condensation and with high pollution.

Annex C [informative] – Cross section constants for thin-walled cross sections

Drafting note: Update of Annex C received from Prof. Höglund on 1 December 2003.

C.1 Open cross sections

(1) Divide the cross section into n parts. Number the parts 1 to n.

Insert nodes between the parts. Number the nodes 0 to n.

Part i is then defined by nodes i - 1 and i.

Give nodes, co-ordinates and (effective) thickness.

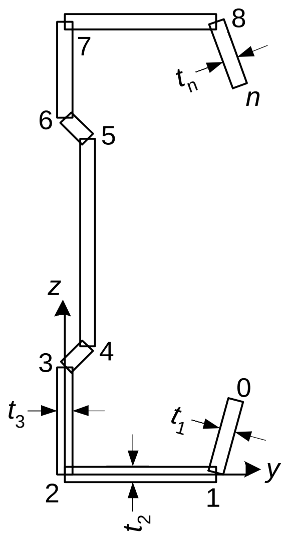


Figure C.1 Cross section nodes

Nodes and parts $\quad j = 0..n \quad i = 1..n$

Area of cross section parts

$$dA_i = \overrightarrow{\left[t_i \cdot \sqrt{(y_i - y_{i-1})^2 + (z_i - z_{i-1})^2} \right]}$$

Cross section area

$$A = \sum_{i=1}^{n} dA_i$$

First moment of area with respect to y-axis and coordinate for gravity centre

$$S_{y0} = \sum_{i=1}^{n} (z_i + z_{i-1}) \cdot \frac{dA_i}{2} \qquad z_{gc} = \frac{S_{y0}}{A}$$

Second moment of area with respect to original y-axis and new y-axis through gravity centre

$$I_{y0} = \sum_{i=1}^{n} \left[(z_i)^2 + (z_{i-1})^2 + z_i \cdot z_{i-1} \right] \cdot \frac{dA_i}{3} \qquad I_y = I_{y0} - A \cdot z_{gc}{}^2$$

First moment of area with respect to z-axis and gravity centre

$$S_{z0} = \sum_{i=1}^{n} (y_i + y_{i-1}) \cdot \frac{dA_i}{2} \qquad y_{gc} = \frac{S_{z0}}{A}$$

Second moment of area with respect to original z-axis and new z-axis through gravity centre

$$I_{z0} = \sum_{i=1}^{n} \left[(y_i)^2 + (y_{i-1})^2 + y_i \cdot y_{i-1} \right] \cdot \frac{dA_i}{3} \qquad I_z = I_{z0} - A \cdot y_{gc}{}^2$$

Product moment of area with respect of original *y*- and *z*-axis and new axes through gravity centre

$$I_{yz0} = \sum_{i=1}^{n} \left(2 \cdot y_{i-1} \cdot z_{i-1} + 2 \cdot y_i \cdot z_i + y_{i-1} \cdot z_i + y_i \cdot z_{i-1}\right) \cdot \frac{dA_i}{6} \quad I_{yz} = I_{yz0} - \frac{S_{y0} \cdot S_{z0}}{A}$$

Principal axis

$$\alpha = \frac{1}{2} \arctan\left(\frac{2I_{yz}}{I_z - I_y}\right) \text{ if } (I_z - I_y) \neq 0 \text{ otherwise } \alpha = 0$$

$$I_\xi = \frac{1}{2} \cdot \left[I_y + I_z + \sqrt{\left(I_z - I_y\right)^2 + 4 \cdot I_{yz}{}^2}\right]$$

$$I_\eta = \frac{1}{2} \cdot \left[I_y + I_z - \sqrt{\left(I_z - I_y\right)^2 + 4 \cdot I_{yz}{}^2}\right]$$

Sectorial co-ordinates

$$\omega_0 = 0 \qquad \omega_{0_i} = y_{i-1} \cdot z_i - y_i \cdot z_{i-1} \qquad \omega_i = \omega_{i-1} + \omega_{0_i}$$

Mean of sectorial coordinate

$$I_\omega = \sum_{i=1}^{n} \left(\omega_{i-1} + \omega_i\right) \cdot \frac{dA_i}{2} \qquad \omega_{mean} = \frac{I_\omega}{A}$$

Sectorial constants

$$I_{y\omega 0} = \sum_{i=1}^{n} \left(2 \cdot y_{i-1} \cdot \omega_{i-1} + 2 \cdot y_i \cdot \omega_i + y_{i-1} \cdot \omega_i + y_i \cdot \omega_{i-1}\right) \cdot \frac{dA_i}{6} \qquad I_{y\omega} = I_{y\omega 0} - \frac{S_{z0} \cdot I_\omega}{A}$$

$$I_{z\omega 0} = \sum_{i=1}^{n} \left(2 \cdot \omega_{i-1} \cdot z_{i-1} + 2 \cdot \omega_i \cdot z_i + \omega_{i-1} \cdot z_i + \omega_i \cdot z_{i-1}\right) \cdot \frac{dA_i}{6} \qquad I_{z\omega} = I_{z\omega 0} - \frac{S_{y0} \cdot I_\omega}{A}$$

$$I_{\omega\omega 0} = \sum_{i=1}^{n} \left[\left(\omega_i\right)^2 + \left(\omega_{i-1}\right)^2 + \omega_i \cdot \omega_{i-1}\right] \cdot \frac{dA_i}{3} \qquad I_{\omega\omega} = I_{\omega\omega 0} - \frac{I_\omega{}^2}{A}$$

Shear centre

$$y_{sc} = \frac{I_{z\omega} I_z - I_{y\omega} I_{yz}}{I_y \cdot I_z - I_{yz}{}^2} \qquad z_{sc} = \frac{-I_{y\omega} I_y + I_{z\omega} I_{yz}}{I_y \cdot I_z - I_{yz}{}^2} \qquad (I_y I_z - I_{yz}^2 \neq 0)$$

Warping constant

$$I_w = I_{\omega\omega} + z_{sc} \cdot I_{y\omega} - y_{sc} \cdot I_{z\omega}$$

Torsion constants

$$I_t = \sum_{i=1}^{n} dA_i \cdot \frac{\left(t_i\right)^2}{3} \qquad W_t = \frac{I_t}{min(t)}$$

Sectorial co-ordinate with respect to shear centre

$$\omega_{s_j} = \omega_j - \omega_{mean} + z_{sc} \cdot (y_j - y_{gc}) - y_{sc} \cdot (z_j - z_{gc})$$

Maximum sectorial co-ordinate and warping modulus

$$\omega_{max} = max(|\omega_s|) \qquad W_w = \frac{I_w}{\omega_{max}}$$

Distance between shear centre and gravity centre

$$y_s = y_{sc} - y_{gc} \qquad z_s = z_{sc} - z_{gc}$$

Polar moment of area with respect to shear centre

$$I_p = I_y + I_z + A(y_s^2 + z_s^2)$$

Non-symmetry factors z_j and y_j according to Annex F

$$z_j = z_s - \frac{0.5}{I_y} \cdot \sum_{i=1}^{n} \left[(z_{c_i})^3 + z_{c_i} \cdot \left[\frac{(z_i - z_{i-1})^2}{4} + (y_{c_i})^2 + \frac{(y_i - y_{i-1})^2}{12} \right] + y_{c_i} \cdot \frac{(y_i - y_{i-1}) \cdot (z_i - z_{i-1})}{6} \right] \cdot dA_i$$

$$y_j = y_s - \frac{0.5}{I_z} \cdot \sum_{i=1}^{n} \left[(y_{c_i})^3 + y_{c_i} \cdot \left[\frac{(y_i - y_{i-1})^2}{4} + (z_{c_i})^2 + \frac{(z_i - z_{i-1})^2}{12} \right] + z_{c_i} \cdot \frac{(z_i - z_{i-1}) \cdot (y_i - y_{i-1})}{6} \right] \cdot dA_i$$

where the coordinates for the centre of the cross section parts with respect to shear center are

$$y_{c_i} = \frac{y_i + y_{i-1}}{2} - y_{gc} \qquad z_{c_i} = \frac{z_i + z_{i-1}}{2} - z_{gc}$$

NOTE: $z_j = 0$ ($y_j = 0$) for cross sections with *y*-axis (*z*-axis) being axis of symmetry, see Figure C.1.

C.2 Cross section constants for open cross section with branches

(1) In cross sections with branches, formulae in C.1 can be used. However, follow the branching back (with thickness $t = 0$) to the next part with thickness $t \neq 0$, see branch 3 - 4 - 5 and 6 - 7 in Figure C.2.

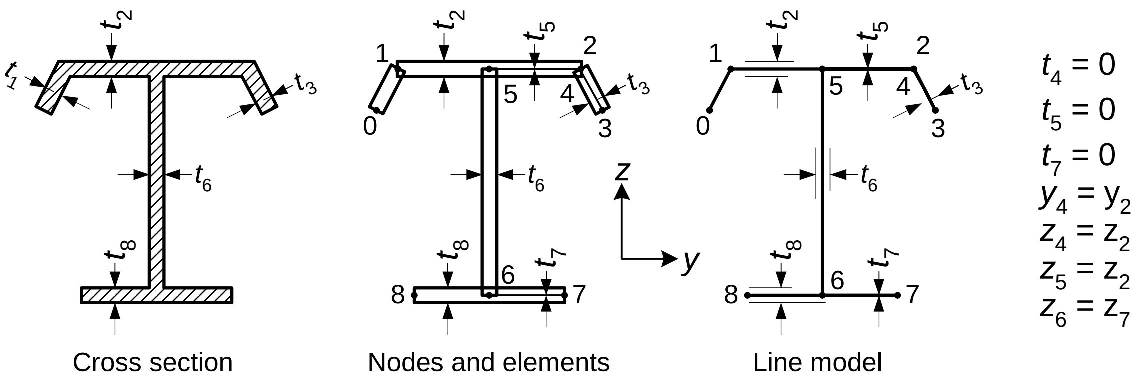


Figure C.2 Nodes and parts in a cross section with branches

C.3 Torsion constant and shear centre of cross section with closed part

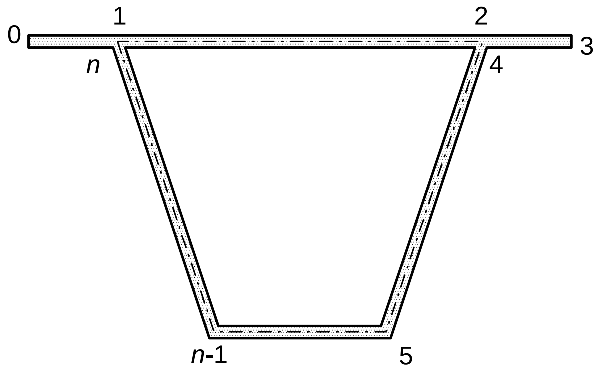


Figure C.3 Cross section with closed part

(1) For a symmetric or non-symmetric cross section with a closed part, Figure C.3, the torsion constant is given by

$$I_t = \frac{4A_t^2}{S_t} \text{ and } W_t = 2\,A_t \min(t_i)$$

where

$$A_t = 0{,}5 \sum_{i=2}^{n} (y_i - y_{i-1})(z_i + z_{i-1})$$

$$S_t = \sum_{i=2}^{n} \frac{\sqrt{(y_i - y_{i-1})^2 + (z_i - z_{i-1})^2}}{t_i} \qquad (t_i \neq 0)$$

Annex D [informative] – Mixed effective width/effective thickness method for outstand elements

Drafting note: Update of Annex D received from Prof. Höglund on 1 December 2003; to be checked with background material, see Prof. Höglund / Dr. Brune.

D.1 The method

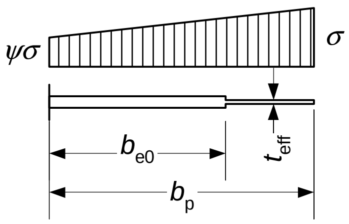
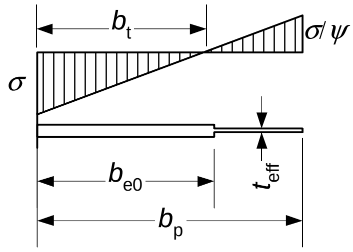
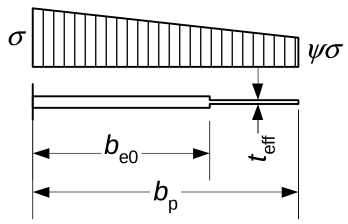
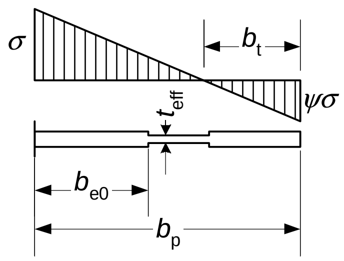
(1) This annex gives an alternative to the effective width method in 5.5.2 for outstand elements in compression. The effective area of the element is composed of the element thickness times an effective width b_{e0} and an effective thickness t_{eff} times the rest of the element width b_p . See Table C.1.

(2) The slenderness parameter $\bar{\lambda}_p$ and reduction factor ρ is found in 5.5.2 for the buckling factor k_σ in Table C.1.

(3) The stress relation factor ψ in the buckling factor k_σ may be based on the stress distribution for the gross cross section.

(4) The resistance of the section shall be based on elastic stress distribution over the section.

Table D.1: Outstand compression elements

Maximum compression at free longitudinal edge		
Stress distribution	Effective width and thickness	Buckling factor
	$1 \geq \psi \geq 0$ $b_{e0} = 0{,}42 b_p$ $t_{eff} = (1{,}75\rho - 0{,}75)t$	$1 \geq \psi \geq -2$ $k_\sigma = \dfrac{1{,}7}{3+\psi}$
	$\psi < 0$ $b_{e0} = \dfrac{0{,}42 b_p}{(1-\psi)} + b_t < b_p$ $b_t = \dfrac{\psi b_p}{(\psi - 1)}$ $t_{eff} = (1{,}75\rho - 0{,}75 - 0{,}15\psi)t$	$-2 > \psi \geq -3$ $k_\sigma = 3{,}3(1+\psi) + 1{,}25\psi^2$ $\psi < -3$ $k_\sigma = 0{,}29(1-\psi)^2$
Maximum compression at supported longitudinal edge		
Stress distribution	Effective width and thickness	Buckling factor
	$1 \geq \psi \geq 0$ $b_{e0} = 0{,}42 b_p$ $t_{eff} = (1{,}75\rho - 0{,}75)t$	$1 \geq \psi \geq 0$ $k_\sigma = \dfrac{1{,}7}{1+3\psi}$
	$\psi < 0$ $b_{e0} = \dfrac{0{,}42 b_p}{(1-\psi)}$ $b_t = \dfrac{\psi b_p}{(\psi - 1)}$ $t_{eff} = (1{,}75\rho - 0{,}75)t$	$0 \geq \psi \geq -1$ $k_\sigma = 1{,}7 - 5\psi + 17{,}1\psi^2$ $\psi < -1$ $k_\sigma = 5{,}98(1-\psi)^2$

Annex E [Informative] – Simplified design for purlins

(1) Purlins with C-, Z- and Σ-cross-sections with or without additional stiffeners in web or flange may be designed due to (2) to (4) if the following conditions are fulfilled :

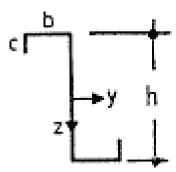
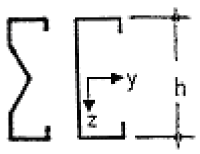
- the cross-section dimension are within the range of table 10.7;
- the purlins are horizontally restraint by trapezoidal sheeting where the horizontal restraint fulfill the conditions of the equation E.1;
- the purlins are restraint rotationally by trapezoidal sheeting and the conditions of table 10.3 are met.
- the purlins have equal spans and uniform loading

This method should not be used:

- for systems using anti-sag bars;
- for sleeve or overlapping systems;
- for application of axial forces N.

Table E.1: Limitations to be fulfilled if the simplified design method is used and other limits as in Table 5.1 and section 5.2

(the axis y and z are parallel respect rectangular to the top flange)

purlins	t [mm]	b/t	h/t	h/b	c/t	b/c	L/h
	≥ 1,25	≤ 55	≤ 160	≤ 3,43	≤ 20	≤ 4,0	≥ 15
	≥ 1,25	≤ 55	≤ 160	≤ 3,43	≤ 20	≤ 4,0	≥ 15

(2) The design value of the bending moment M_{Ed} should satisfy

$$\frac{M_{\mathrm{Ed}}}{M_{\mathrm{LT,Rd}}} \leq 1 \qquad \text{…(E.1)}$$

where

$$M_{\mathrm{LT,Rd}} = \left(\frac{f_{\mathrm{y}}}{\gamma_{\mathrm{M1}}}\right) W_{\mathrm{eff,y}} \frac{\chi_{\mathrm{LT}}}{k_{\mathrm{d}}} \qquad \text{…(E.2)}$$

and

$W_{\mathrm{eff,y}}$ is section modulus of the effective cross-section with regard to the axis y;

χ_{LT} is reduction factor for lateral torsional buckling in dependency of $\bar{\lambda}_{\mathrm{LT}}$ due to 6.2.3, where α_{LT} is substituted by $\alpha_{\mathrm{LT,eff}}$;

and

$$\bar{\lambda}_{\mathrm{LT}} = \sqrt{\frac{W_{\mathrm{eff,y}}\ f_{\mathrm{y}}}{M_{\mathrm{cr}}}} \quad \ldots(E.3)$$

$$\alpha_{\mathrm{LT,eff}} = \alpha_{\mathrm{LT}} \sqrt{\frac{W_{\mathrm{el,y}}}{W_{\mathrm{eff,y}}}} \quad \ldots(E.4)$$

and

α_{LT} is imperfection factor due to 6.2.3;

$W_{\mathrm{el,y}}$ is section modulus of the gross cross-section with regard to the axis y;

k_{d} is coefficient for consideration of the non restraint part of the purlin due to the equation (E.5) and table E.2;

$$k_{\mathrm{d}} = \left(a_1 - a_2 \frac{L}{h}\right), \text{ but } \geq 1.0 \quad \ldots(E.5)$$

a_1, a_2 coefficients from table E.2;

L span of the purlin;

h overall depth of the purlin.

Table E.2: Coefficients a_1, a_2 for equation (E.5)

System	Z-purlins		C-purlins		Σ-purlins	
	a_1	a_2	a_1	a_2	a_1	a_2
single span beam gravity load	1.0	0	1.1	0.002	1.1	0.002
single span beam uplift load	1.3	0	3.5	0.050	1.9	0.020
continuous beam gravity load	1.0	0	1.6	0.020	1.6	0.020
continuous beam uplift load	1.4	0.010	2.7	0.040	1.0	0

(3) The reduction factor χ_{LT} may be chosen by equation (E.6), if a single span beam under gravity load is present or if equation (E.7) is met

$$\chi_{LT} = 1{,}0 \qquad \ldots(E.6)$$

$$C_D \geq \frac{M_{el,u}^2}{E\, I_v} k_\vartheta \qquad \ldots(E.7)$$

where

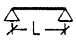
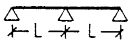
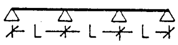
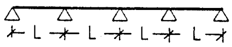
$M_{el,u} = W_{el,u}\, f_y$ elastic moment of the gross cross-section with regard to the major axis u; …(E.8)

I_v moment of inertia of the gross cross-section with regard to the minor axis v:

k_ϑ factor for considering the static system of the purlin due to table E.3.

NOTE: For equal flanged C-purlins and Σ-purlins $I_v = I_z$, $W_u = W_y$, and $M_{el,u} = M_{el,y}$. Conventions used for cross section axes are shown in Figure 1.7 and section 1.6.4.

Table E.3: Factors k_ϑ

Statical system	Gravity load	Uplift load
	-	0.210
	0.07	0.029
	0.15	0.066
	0.10	0.053

(4) The reduction factor χ_{LT} should be calculated by equation (6.36) using $\bar{\lambda}_{LT}$ and $\alpha_{LT,eff}$ in cases which are not met by (3). The elastic critical moment for lateral-torsional buckling M_{cr} may be calculated by the equation (E.9):

$$M_{cr} = \frac{k}{L}\sqrt{G\, I_t^*\, E\, I_v} \qquad \ldots(E.9)$$

where

I_t^* is the fictitious St. Venant torsion constant considering the effective rotational restraint by equation (E.10) and (E.11):

$$I_t^* = I_t + C_D \frac{L^2}{\pi^2\, G} \qquad \ldots(E.10)$$

I_t is St. Venant torsion constant of the purlin;

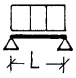
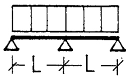
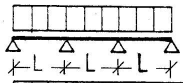
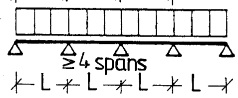
$$1/C_D = \frac{1}{C_{D,A}} + \frac{1}{C_{D,B}} + \frac{1}{C_{D,C}} \qquad \ldots(E.11)$$

$C_{D,A}$, $C_{D,C}$ rotational stiffnesses due to 10.1.5.2;

$C_{D,B}$ rotational stiffnesses due to distorsion of the cross-section of the purlin due to 10.1.5.1, $C_{D,B} = K_B\, h^2$, where h = depth of the purlin and K_B according to 10.1.5.1;

k lateral torsional buckling coefficient due to table E.4.

Table E.4: Lateral torsional buckling coefficients *k* for beams restraint horizontally at the upper flange

Statical system	Gravity load	Uplift load
	∞	10.3
	17.7	27.7
	12.2	18.3
	14.6	20.5

EUROPEAN STANDARD
NORME EUROPÉENNE
EUROPÄISCHE NORM

prEN 1993-1-5 : 2004

~~11 June~~December 2004

UDC

Descriptors:

English version

Eurocode 3 : Design of steel structures

Part 1.5 : Plated structural elements

Calcul des structures en acier

Partie 1.5 :

Plaques planes ~~Bauteile~~

Bemessung und Konstruktion von Stahlbauten

Teil 1.5 :

Plattenbeulen ~~Aus Blechen zusammengesetzte~~

Stage 49 draft

CEN

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

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

Ref. No. EN 1993-1.5 : 2004. E

Content

Foreword

This document (prEN 1993-1-5: 2004) has been prepared by Technical Committee CEN/TC 250 "Structural Eurocodes", the secretariat of which is held be BSI.

This document is currently submitted to the Formal Vote.

This document will supersede ENV 1993-1-5.

National annex for EN 1993-1-5

This standard gives alternative procedures, values and recommendations with notes indicating where national choices may have to be made. The National Standard implementing EN 1993-1-5 should have a National Annex containing all Nationally Determined Parameters to be used for the design of steel structures to be constructed in the relevant country.

National choice is allowed in EN 1993-1-5 through:

- 2.2(5)
- 3.3(1)
- 4.3(6)
- 5.1(2)
- 6.4(2)
- 8(2)
- 9.1(1)
- 9.2.1(8)
- 10(1)
- 10(5)
- C.2(1)
- C.5(2)
- C.8(1)
- C.9(3)

1 Introduction

1.1 Scope

(1) EN 1993-1-5 gives design requirements of stiffened and unstiffened plates which are subject to in-plane forces.

(2) Effects due to shear lag, in-plane load introduction and plate buckling for I-section girders and box girders are covered. Also covered are plated structural components subject to in-plane loads as in tanks and silos. The effects of out-of-plane loading are outside the scope of this document.

NOTE 1 The rules in this part complement the rules for class 1, 2, 3 and 4 sections, see EN 1993-1-1.

NOTE 2 For the design of slender plates which are subject to repeated direct stress and/or shear and also fatigue due to out-of-plane bending of plate elements (breathing) see EN 1993-2 and EN 1993-6.

NOTE 3 For the effects of out-of-plane loading and for the combination of in-plane effects and out-of-plane loading effects see EN 1993-2 and EN 1993-1-7.

NOTE 4 Single plate elements may be considered as flat where the curvature radius r satisfies:

$$r \geq \frac{b^2}{t} \qquad (1.1)$$

where b is the panel width

t is the plate thickness

1.2 Normative references

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

EN 1993 Eurocode 3: Design of steel structures:

Part 1.1: General rules and rules for buildings;

1.3 Terms and definitions

For the purpose of this standard, the following terms and definitions apply:

1.3.1

elastic critical stress

stress in a component at which the component becomes unstable when using small deflection elastic theory of a perfect structure

1.3.2

membrane stress

stress at mid-plane of the plate

1.3.3

gross cross-section

the total cross-sectional area of a member but excluding discontinuous longitudinal stiffeners

1.3.4
effective cross-section and effective width
the gross cross-section or width reduced for the effects of plate buckling or shear lag or both; to distinguish between their effects the word "effective" is clarified as follows:
"effective^p" denotes effects of plate buckling
"effective^s" denotes effects of shear lag
"effective" denotes effects of plate buckling and shear lag

1.3.5
plated structure
a structure built up from nominally flat plates which are ~~joined~~ connected together; the plates may be stiffened or unstiffened

1.3.6
stiffener
a plate or section attached to a plate to resist buckling or to strengthen the plate; a stiffener is denoted:

- longitudinal if its direction is parallel to the member;
- transverse if its direction is perpendicular to the member.

1.3.7
stiffened plate
plate with transverse or longitudinal stiffeners or both

1.3.8
subpanel
unstiffened plate portion surrounded by flanges and/or stiffeners

1.3.9
hybrid girder
girder with flanges and web made of different steel grades; this standard assumes higher steel grade in flanges compared to webs

1.3.10
sign convention
unless otherwise stated compression is taken as positive

1.4 Symbols

(1) In addition to those given in EN 1990 and EN 1993-1-1, the following symbols are used:

$A_{s\ell}$ total area of all the longitudinal stiffeners of a stiffened plate;

A_{st} gross cross sectional area of one transverse stiffener;

A_{eff} effective cross sectional area;

$A_{c,eff}$ effective^p cross sectional area;

$A_{c,eff,loc}$ effective^p cross sectional area for local buckling;

a length of a stiffened or unstiffened plate;

b width of a stiffened or unstiffened plate;

b_w clear width between welds;

b_{eff} effective^s width for elastic shear lag;

F_{Ed} design transverse force;

h_w clear web depth between flanges;

L_{eff} effective length for resistance to transverse forces, see 6;

$M_{f.Rd}$ design plastic moment of resistance of a cross-section consisting of the flanges only;

$M_{pl.Rd}$ design plastic moment of resistance of the cross-section (irrespective of cross-section class);

M_{Ed} design bending moment;

N_{Ed} design axial force;

t thickness of the plate;

V_{Ed} design shear force including shear from torque;

W_{eff} effective elastic section modulus;

β effective^s width factor for elastic shear lag;

(2) Additional symbols are defined where they first occur.

2 Basis of design and modelling

2.1 General

(1) The effects of shear lag and plate buckling should be taken into account at the ultimate, serviceability or fatigue limit states.

NOTE Partial factors γ_{M0} and γ_{M1} used in this part are defined for different applications in the National Annexes of EN 1993-1 to EN 1993-6.

2.2 Effective width models for global analysis

(1) The effects of shear lag and of plate buckling on the stiffness of members and joints should be taken into account in the global analysis.

(2) The effects of shear lag of flanges in global analysis may be taken into account by the use of an effective^s width. For simplicity this effective^s width may be assumed to be uniform over the length of the span.

(3) For each span of a ~~beam~~ member the effective^s width of flanges should be taken as the lesser of the full width and L/8 per side of the web, where L is the span or twice the distance from the support to the end of a cantilever.

(4) The effects of plate buckling in elastic global analysis may be taken into account by effective^p cross sectional areas of the elements in compression, see 4.3.

(5) For global analysis the effect of plate buckling on the stiffness may be ignored when the effective^p cross-sectional area of an element in compression is larger than ρ_{lim} times the gross cross-sectional area of the same element.

NOTE 1 The parameter ρ_{lim} may be given in the National Annex. The value ρ_{lim} = 0,5 is recommended.

NOTE 2 For determining the stiffness when (5) is not fulfilled, see Annex E.

2.3 Plate buckling effects on uniform members

(1) Effective^p width models for direct stresses, resistance models for shear buckling and buckling due to transverse loads as well as interactions between these models for determining the resistance of uniform members at the ultimate limit state may be used when the following conditions apply:

- panels are rectangular and flanges are parallel;
- the diameter of any unstiffened open hole or cut out does not exceed 0,05b, where b is the width of the panel.

NOTE The rules may apply to non rectangular panels provided the angle α_{limit} (see Figure 2.1) is not greater than 10 degrees. If α_{limit} exceeds 10, panels may be assessed assuming it to be a rectangular panel based on the larger of b_1 and b_2 of the panel.

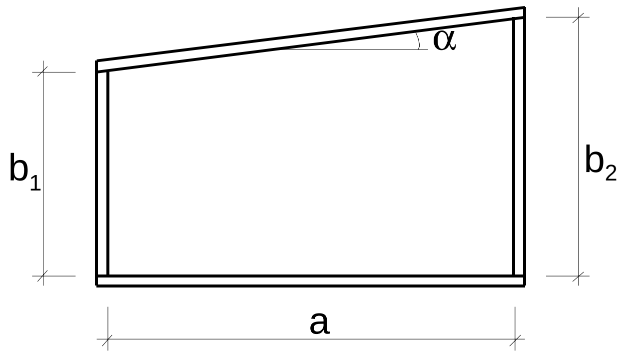


Figure 2.1: Definition of angle α

(2) For the calculation of stresses at the serviceability and fatigue limit state the effective[s] area may be used if the condition in 2.5(5) is fulfilled. For ultimate limit states the effective area according to 3.3 should be used with β replaced by β_{ult}.

2.4 Reduced stress method

(1) As an alternative to the use of the effective[p] width models for direct stresses given in sections 4 to 7, the cross sections may be assumed to be class 3 sections provided that the stresses in each panel do not exceed the limits specified in section 10.

NOTE The reduced stress method is analogous to the effective[p] width method (see 2.3) for single plated elements. However, in verifying the stress limitations no load shedding has been assumed between the plated elements of the cross section.

2.5 Non uniform members

(1) Non uniform members (e.g. haunched ~~beams~~ members, non rectangular panels) or members with regular or irregular large openings may be analysed using Finite Element (FE) methods.

NOTE 1 See Annex B for non uniform members.

NOTE 2 For FE-calculations see Annex C.

2.6 Members with corrugated webs

(1) For members with corrugated webs, the bending stiffness should be based on the flanges only and webs should be considered to transfer shear and transverse loads.

NOTE For plate buckling resistance of flanges in compression and the shear resistance of webs see Annex D.

3 Shear lag in member design

3.1 General

(1) Shear lag in flanges may be neglected if $b_0 < L_e/50$ where b_0 is taken as the flange outstand or half the width of an internal element and L_e is the length between points of zero bending moment, see 3.2.1(2).

(2) Where the above limit for b_0 is exceeded the effects due to shear lag in flanges should be considered at serviceability and fatigue limit state verifications by the use of an effective^s width according to 3.2.1 and a stress distribution according to 3.2.2. For the ultimate limit state verification an effective area according to 3.3 may be used.

(3) Stresses due to patch loading in the web applied at the flange level should be determined from 3.2.3.

3.2 Effective^s width for elastic shear lag

3.2.1 Effective width

(1) The effective^s width b_{eff} for shear lag under elastic conditions should be determined from:

$$b_{eff} = \beta\, b_0 \tag{3.1}$$

where the effective^s factor β is given in Table 3.1.

This effective width may be relevant for serviceability and fatigue limit states.

(2) Provided adjacent spans do not differ more than 50% and any cantilever span is not larger than half the adjacent span the effective lengths L_e may be determined from Figure 3.1. For all other cases L_e should be taken as the distance between adjacent points of zero bending moment.

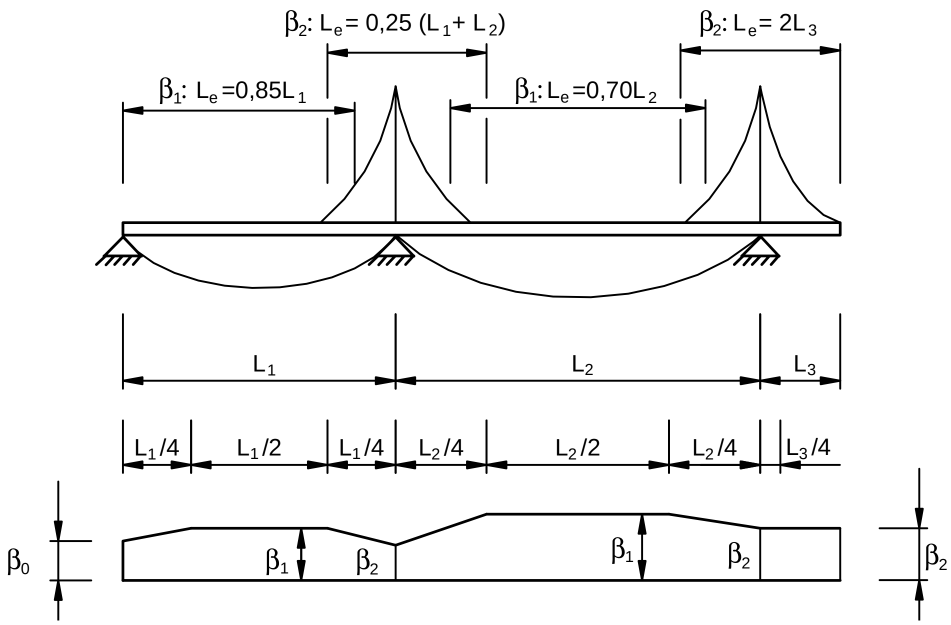
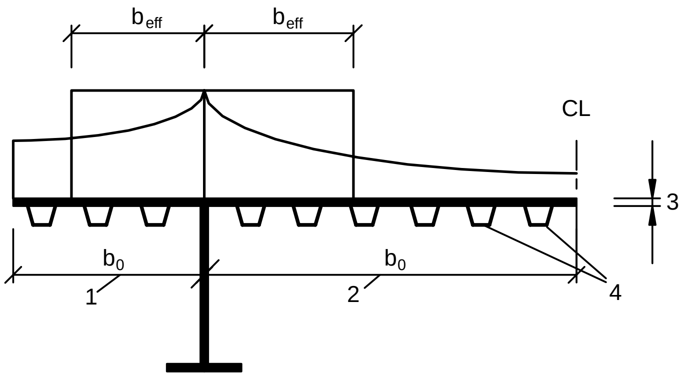


Figure 3.1: Effective length L_e for continuous beam and distribution of effective^s width



1 for flange outstand
2 for internal flange
3 plate thickness t
4 stiffeners with $A_{s\ell} = \sum A_{s\ell i}$

Figure 3.2: Notations for shear lag

Table 3.1: Effective[s] width factor β

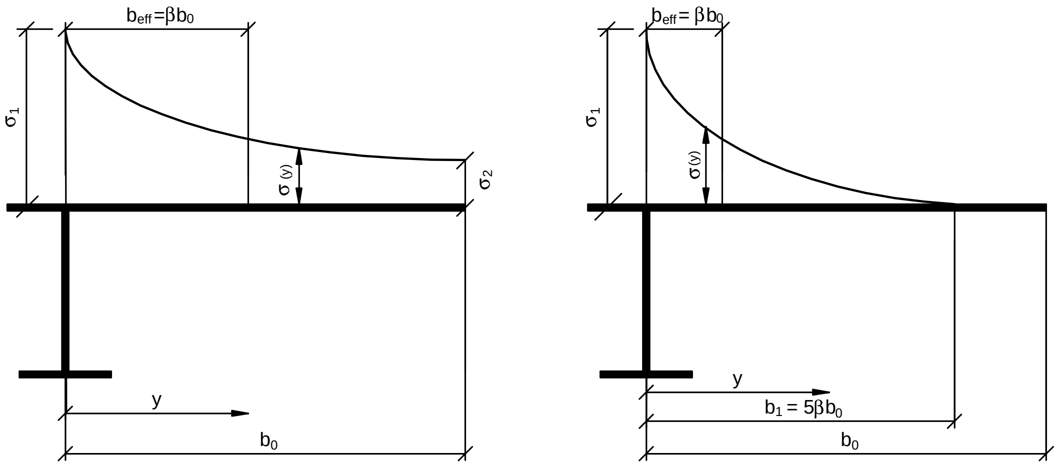
κ	verification	β – value
$\kappa \leq 0{,}02$		$\beta = 1{,}0$
$0{,}02 < \kappa \leq 0{,}70$	sagging bending	$\beta = \beta_1 = \dfrac{1}{1 + 6{,}4\,\kappa^2}$
	hogging bending	$\beta = \beta_2 = \dfrac{1}{1 + 6{,}0\left(\kappa - \dfrac{1}{2500\,\kappa}\right) + 1{,}6\,\kappa^2}$
$> 0{,}70$	sagging bending	$\beta = \beta_1 = \dfrac{1}{5{,}9\,\kappa}$
	hogging bending	$\beta = \beta_2 = \dfrac{1}{8{,}6\,\kappa}$
all κ	end support	$\beta_0 = (0{,}55 + 0{,}025 / \kappa)\,\beta_1$, but $\beta_0 < \beta_1$
all κ	cantilever	$\beta = \beta_2$ at support and at the end

$\kappa = \alpha_0\, b_0 / L_e$ with $\alpha_0 = \sqrt{1 + \dfrac{A_{s\ell}}{b_0 t}}$

in which $A_{s\ell}$ is the area of all longitudinal stiffeners within the width b_0 and other symbols are as defined in Figure 3.1 and Figure 3.2.

3.2.2 Stress distribution due to shear lag

(1) The distribution of longitudinal stresses across the flange plate due to shear lag should be obtained from Figure 3.3.



$\beta > 0{,}20:$

$\sigma_2 = 1{,}25\,(\beta - 0{,}20)\,\sigma_1$

$\sigma(y) = \sigma_2 + (\sigma_1 - \sigma_2)\,(1 - y/b_0)^4$

$\beta < 0{,}20:$

$\sigma_2 = 0$

$\sigma(y) = \sigma_1 (1 - y/b_1)^4$

σ_1 is calculated with the effective width of the flange b_{eff}

Figure 3.3: Distribution of stresses due to shear lag

3.2.3 In-plane load effects

(1) The elastic stress distribution in a stiffened or unstiffened plate due to the local introduction of in-plane forces (patch loads), see Figure 3.4, should be determined from:

$$\sigma_{z,Ed} = \frac{F_{Ed}}{b_{eff}\,(t_w + a_{st,l})} \qquad (3.2)$$

with: $b_{eff} = s_e \sqrt{1 + \left(\frac{z}{s_e\, n}\right)^2}$

$$n = 0{,}636 \sqrt{1 + \frac{0{,}878\, a_{st,1}}{t_w}}$$

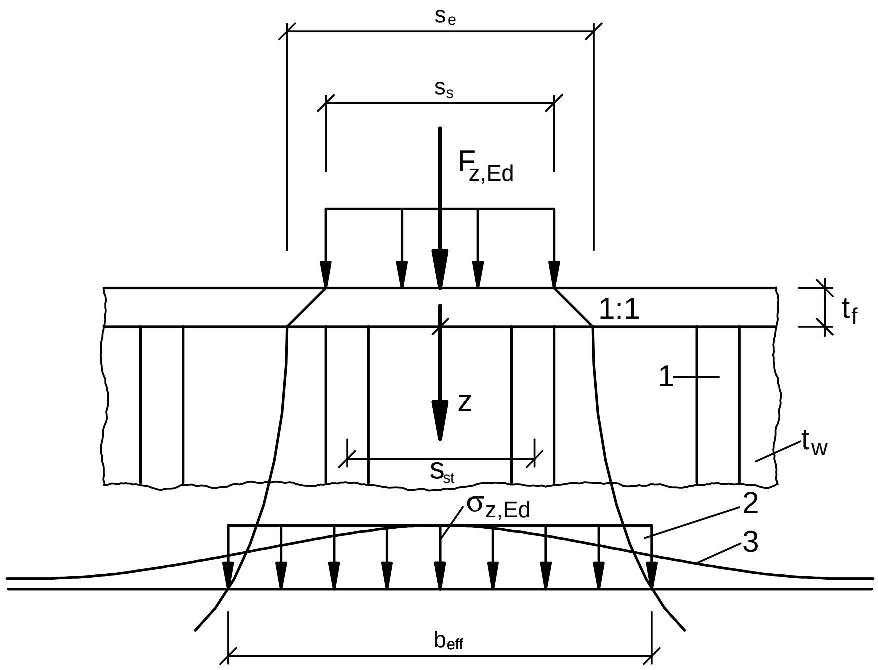
$$s_e = s_s + 2\, t_f$$

where $a_{st,1}$ is the gross cross-sectional area of the stiffeners smeared over the length s_e. This may be taken, conservatively, as the area of the stiffeners divided by the spacing s_{st};

t_w is the web thickness;

z is the distance to flange.

NOTE The equation (3.2) is valid when $s_{st}/s_e \leq 0{,}5$; otherwise the contribution of stiffeners should be neglected.



1 stiffener
2 simplified stress distribution
3 actual stress distribution

Figure 3.4: In-plane load introduction

NOTE The above stress distribution may also be used for the fatigue verification.

3.3 Shear lag at the ultimate limit states

(1) At the ultimate limit states shear lag effects may be determined as follows:

a) elastic shear lag effects as determined for serviceability and fatigue limit states,

b) combined effects of shear lag and of plate buckling,

c) elastic-plastic shear lag effects allowing for limited plastic strains.

NOTE 1 The National Annex may choose the method to be applied. Unless specified otherwise in EN 1993-2 to EN 1993-6, the method in NOTE 3 is recommended.

NOTE 2 The combined effects of plate buckling and shear lag may be taken into account by using A_{eff} as given by

$$A_{eff} = A_{c,eff}\,\beta_{ult} \tag{3.3}$$

where $A_{c,eff}$ is the effective^p area of the compression flange due to plate buckling (see 4.4 and 4.5);

β_{ult} is the effective^s width factor for the effect of shear lag at the ultimate limit state, which may be taken as β determined from Table 3.1 with α_0 replaced by

$$\alpha_0^* = \sqrt{\frac{A_{c,eff}}{b_0\ t_f}} \tag{3.4}$$

t_f is the flange thickness.

NOTE 3 Elastic-plastic shear lag effects allowing for limited plastic strains may be taken into account using A_{eff} as follows:

$$A_{eff} = A_{c,eff}\,\beta^{\kappa} \geq A_{c,eff}\,\beta \tag{3.5}$$

where β and κ are taken from Table 3.1.

The expressions in NOTE 2 and NOTE 3 may also be applied for flanges in tension in which case $A_{c,eff}$ should be replaced by the gross area of the tension flange.

4 Plate buckling effects due to direct stresses at the ultimate limit state

4.1 General

(1) This section gives rules to account for plate buckling effects from direct stresses at the ultimate limit state when the following criteria are met:

a) The panels are rectangular and flanges are parallel or nearly parallel (see 2.3);~~.~~

b) Stiffeners if any are provided in the longitudinal or transverse direction or both;~~.~~

c) Open holes ~~or~~ and cut outs are small (see 2.3);~~.~~

d) Members are of uniform cross section;~~.~~

e) No flange induced web buckling occurs.

NOTE 1 For compression flange buckling in the plane of the web see section 8.

NOTE 2 For stiffeners and detailing of plated members subject to plate buckling see section 9.

4.2 Resistance to direct stresses

(1) The resistance of plated members may be determined using the effective areas of plate elements in compression for class 4 sections using cross sectional data (A_{eff}, I_{eff}, W_{eff}) for cross sectional verifications and member verifications for column buckling and lateral torsional buckling according to EN 1993-1-1.

(2) Effective[p] areas should be determined on the basis of the linear strain distributions with the attainment of yield strain in the mid plane of the compression plate.

4.3 Effective cross section

(1) In calculating longitudinal stresses, account should be taken of the combined effect of shear lag and plate buckling using the effective areas given in 3.3.

(2) The effective cross sectional properties of members should be based on the effective areas of the compression elements and on the effective[s] area of the tension elements due to shear lag.

(3) The effective area A_{eff} should be determined assuming that the cross section is subject only to stresses due to uniform axial compression. For non-symmetrical cross sections the possible shift e_N of the centroid of the effective area A_{eff} relative to the centre of gravity of the gross cross-section, see Figure 4.1, gives an additional moment which should be taken into account in the cross section verification using 4.6.

(4) The effective section modulus W_{eff} should be determined assuming the cross section is subject only to bending stresses, see Figure 4.2. For biaxial bending effective section moduli should be determined about both main axes.

NOTE As an alternative to 4.3(3) and (4) a single effective section may be determined from N_{Ed} and M_{Ed} acting simultaneously. The effects of e_N should be taken into account as in 4.3(3). This requires an iterative procedure.

(5) The stress in a flange should be calculated using the elastic section modulus with reference to the mid-plane of the flange.

(6) Hybrid girders may have flange material with yield strength f_{yf} up to $\varphi_h \times f_{yw}$ provided that:

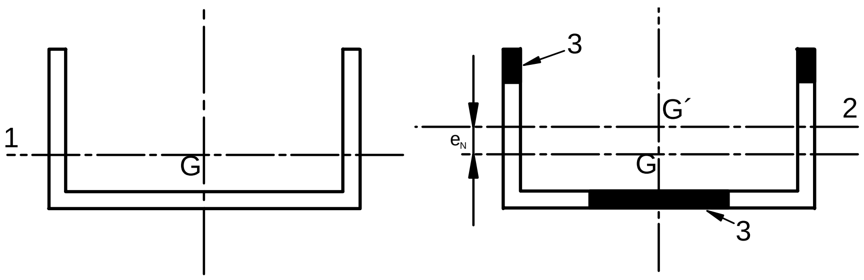
a) the increase of flange stresses caused by yielding of the web is taken into account by limiting the stresses in the web to f_{yw}

b) f_{yf} (rather than f_{yw}) is used in determining the effective area of the web.

NOTE The National Annex may specify the value φ_h. A value of $\varphi_h = 2,0$ is recommended.

(7) The increase of deformations and of stresses at serviceability and fatigue limit states may be ignored for hybrid girders complying with 4.3(6) including the NOTE.

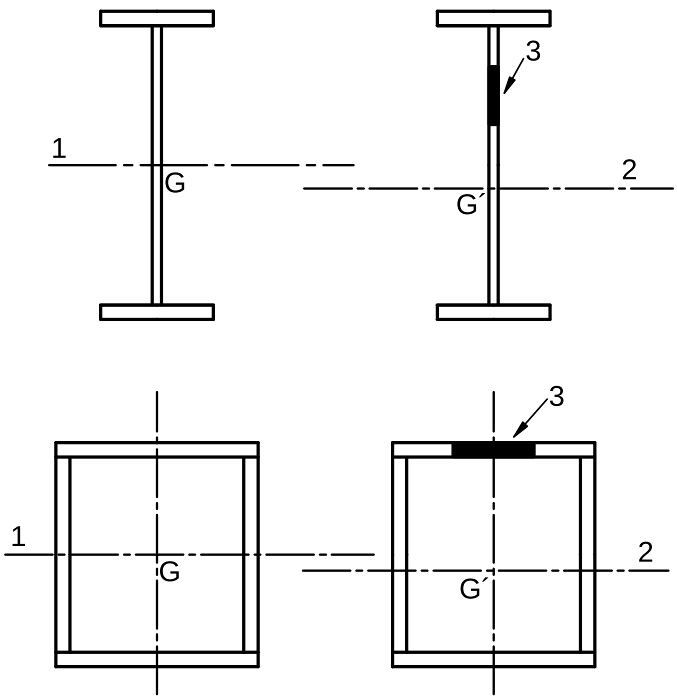
(8) For hybrid girders complying with 4.3(6) the stress range limit in EN 1993-1-9 may be taken as $1,5f_{yf}$.



Gross cross section   Effective cross section

G centroid of the gross cross section
G´ centroid of the effective cross section
1 centroidal axis of the gross cross section
2 centroidal axis of the effective cross section
3 non effective zone

Figure 4.1: Class 4 cross-sections - axial force



G centroid of the gross cross section
G´ centroid of the effective cross section
1 centroidal axis of the gross cross section
2 centroidal axis of the effective cross section
3 non effective zone

Gross cross section   Effective cross section

Figure 4.2: Class 4 cross-sections - bending moment

4.4 Plate elements without longitudinal stiffeners

(1) The effective[p] areas of flat compression elements should be obtained using Table 4.1 for internal elements and Table 4.2 for outstand elements. The effective[p] area of the compression zone of a plate with the gross cross-sectional area A_c should be obtained from:

$$A_{c,eff} = \rho\, A_c \tag{4.1}$$

where ρ is the reduction factor for plate buckling.

(2) The reduction factor ρ may be taken as follows:

– internal compression elements:

$\rho = 1{,}0$ for $\bar{\lambda}_p \le 0{,}673$

$$\rho = \frac{\bar{\lambda}_p - 0{,}055\,(3+\psi)}{\bar{\lambda}_p^2} \le 1{,}0 \quad \text{for } \bar{\lambda}_p > 0{,}673 \text{ , where } (3+\psi) \ge 0 \tag{4.2}$$

– outstand compression elements:

$\rho = 1{,}0$ for $\bar{\lambda}_p \le 0{,}748$

$$\rho = \frac{\bar{\lambda}_p - 0{,}188}{\bar{\lambda}_p^2} \le 1{,}0 \quad \text{for } \bar{\lambda}_p > 0{,}748 \tag{4.3}$$

where $\bar{\lambda}_p = \sqrt{\dfrac{f_y}{\sigma_{cr}}} = \dfrac{\bar{b}/t}{28{,}4\,\varepsilon\,\sqrt{k_\sigma}}$

ψ is the stress ratio determined in accordance with 4.4(3) and 4.4(4)

$\bar{b}$ is the appropriate width to be taken as follows (for definitions, see Table 5.2 of EN 1993-1-1)

b_w for webs;

b for internal flange elements (except RHS);

$b - 3\,t$ for flanges of RHS;

c for outstand flanges;

h for equal-leg angles;

h for unequal-leg angles;

k_σ is the buckling factor corresponding to the stress ratio ψ and boundary conditions. For long plates k_σ is given in Table 4.1 or Table 4.2 as appropriate;

t is the thickness;

σ_{cr} is the elastic critical plate buckling stress see equation (A.1) in Annex A.1(2) and Table 4.1 and Table 4.2;

$$\varepsilon = \sqrt{\frac{235}{f_y\,[N/mm^2]}}$$

(3) For flange elements of I-sections and box girders the stress ratio ψ used in Table 4.1 ~~or~~ and Table 4.2 should be based on the properties of the gross cross-sectional area, due allowance being made for shear lag in the flanges if relevant. For web elements the stress ratio ψ used in Table 4.1 should be obtained using a stress distribution based on the effective area of the compression flange and the gross area of the web.

NOTE If the stress distribution results from different stages of construction (as e.g. in a composite bridge) the stresses from the various stages may first be calculated with a cross section consisting of effective flanges and gross web and these stresses are added together. This resulting stress distribution determines an effective web section that can be used for all stages to calculate the final stress distribution for stress analysis.

(4) Except as given in 4.4(5), the plate slenderness $\bar{\lambda}_p$ of an element may be replaced by:

$$\bar{\lambda}_{p,red} = \bar{\lambda}_p \sqrt{\frac{\sigma_{com,Ed}}{f_y / \gamma_{M0}}} \qquad (4.4)$$

where $\sigma_{com,Ed}$ is the maximum design compressive stress in the element determined using the effective[P] area of the section caused by all simultaneous actions.

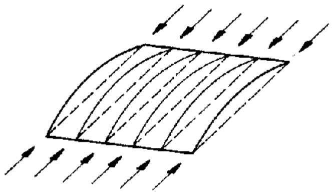
NOTE 1 The above procedure is conservative and requires an iterative calculation in which the stress ratio ψ (see Table 4.1 and Table 4.2) is determined at each step from the stresses calculated on the effective[P] cross-section defined at the end of the previous step.

NOTE 2 See also alternative procedure in Annex E.

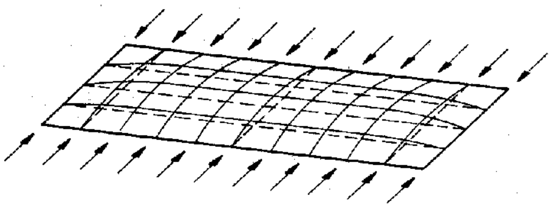
(5) For the verification of the design buckling resistance of a class 4 member using 6.3.1, 6.3.2 or 6.3.4 of EN 1993-1-1, either the plate slenderness $\bar{\lambda}_p$ or $\bar{\lambda}_{p,red}$ with $\sigma_{com,Ed}$ based on second order analysis with global imperfections should be used.

(6) For aspect ratios a/b < 1 a column type of buckling may occur and the check should be performed according to 4.5.3 using the reduction factor ρ_c.

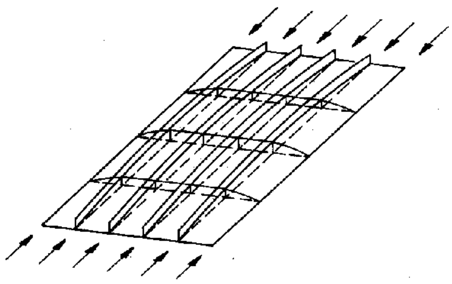
NOTE This applies e.g. for flat elements between transverse stiffeners where plate buckling could be column-like and require a reduction factor ρ_c close to χ_c as for column buckling, see Figure 4.3 a) and b). For plates with longitudinal stiffeners column type buckling may also occur for a/b ≥ 1, see Figure 4.3 c).



a) column-like behaviour of plates without longitudinal supports



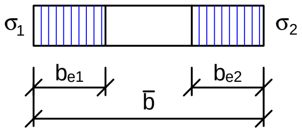
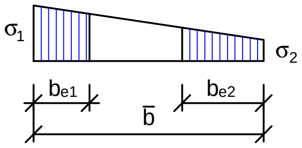
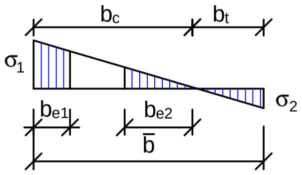
b) column-like behaviour of an unstiffened plate with a small aspect ratio α



c) column-like behaviour of a longitudinally stiffened plate with a large aspect ratio α

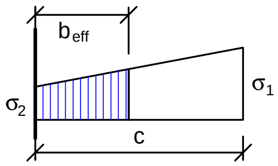
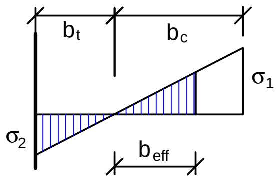
Figure 4.3: Column-like behaviour

Table 4.1: Internal compression elements

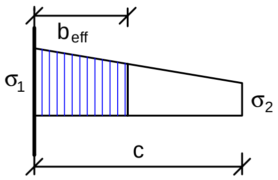
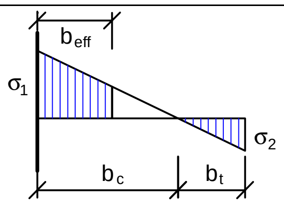
Stress distribution (compression positive)	Effective^{p} width b_{eff}
	$\underline{\psi = 1}$: $b_{eff} = \rho\ \bar{b}$ $b_{e1} = 0{,}5\ b_{eff}$ $\quad b_{e2} = 0{,}5\ b_{eff}$
	$\underline{1 > \psi \geq 0}$: $b_{eff} = \rho\ \bar{b}$ $b_{e1} = \dfrac{2}{5-\psi} b_{eff}$ $\quad b_{e2} = b_{eff} - b_{e1}$
	$\underline{\psi < 0}$: $b_{eff} = \rho\ b_c = \rho\ \bar{b} / (1-\psi)$ $b_{e1} = 0{,}4\ b_{eff}$ $\quad b_{e2} = 0{,}6\ b_{eff}$

$\psi = \sigma_2/\sigma_1$	1	$1 > \psi > 0$	0	$0 > \psi > -1$	-1	$-1 > \psi > -3$
Buckling factor k_σ	4,0	$8{,}2 / (1{,}05 + \psi)$	7,81	$7{,}81 - 6{,}29\psi + 9{,}78\psi^2$	23,9	$5{,}98\ (1 - \psi)^2$

Table 4.2: Outstand compression elements

Stress distribution (compression positive)	Effective^{p} width b_{eff}
	$\underline{1 > \psi \geq 0}$: $b_{eff} = \rho\ c$
	$\underline{\psi < 0}$: $b_{eff} = \rho\ b_c = \rho\ c / (1-\psi)$

$\psi = \sigma_2/\sigma_1$	1	0	-1	$1 \geq \psi \geq -3$
Buckling factor k_σ	0,43	0,57	0,85	$0{,}57 - 0{,}21\psi + 0{,}07\psi^2$

Stress distribution (compression positive)	Effective^{p} width b_{eff}
	$\underline{1 > \psi \geq 0}$: $b_{eff} = \rho\ c$
	$\underline{\psi < 0}$: $b_{eff} = \rho\ b_c = \rho\ c / (1-\psi)$

$\psi = \sigma_2/\sigma_1$	1	$1 > \psi > 0$	0	$0 > \psi > -1$	-1
Buckling factor k_σ	0,43	$0{,}578 / (\psi + 0{,}34)$	1,70	$1{,}7 - 5\psi + 17{,}1\psi^2$	23,8

4.5 Stiffened plate elements with longitudinal stiffeners

4.5.1 General

(1) For plates with longitudinal stiffeners the effective[p] areas from local buckling of the various subpanels between the stiffeners and the effective[p] areas from the global buckling of the stiffened panel should be accounted for.

(2) The effective[p] section area of each subpanel should be determined by a reduction factor in accordance with 4.4 to account for local plate buckling. The stiffened plate with effective[p] section areas for the stiffeners should be checked for global plate buckling (by modelling it as an equivalent orthotropic plate) and a reduction factor ρ should be determined for overall plate buckling.

(3) The effective[p] area of the compression zone of the stiffened plate should be taken as:

$$A_{c,eff} = \rho_c \, A_{c,eff,loc} + \sum b_{edge,eff}\, t \tag{4.5}$$

where $A_{c,eff,loc}$ is the effective[p] section areas of all the stiffeners and subpanels that are fully or partially in the compression zone except the effective parts supported by an adjacent plate element with the width $b_{edge,eff}$, see example in Figure 4.4.

(4) The area $A_{c,eff,loc}$ should be obtained from:

$$A_{c,eff,loc} = A_{s\ell,eff} + \sum_c \rho_{loc}\, b_{c,loc}\, t \tag{4.6}$$

where $\sum_c$ applies to the part of the stiffened panel width that is in compression except the parts $b_{edge,eff}$, see Figure 4.4;

$A_{s\ell,eff}$ is the sum of the effective[p] sections according to 4.4 of all longitudinal stiffeners with gross area $A_{s\ell}$ located in the compression zone;

$b_{c,loc}$ is the width of the compressed part of each subpanel;

ρ_{loc} is the reduction factor from 4.4(2) for each subpanel.

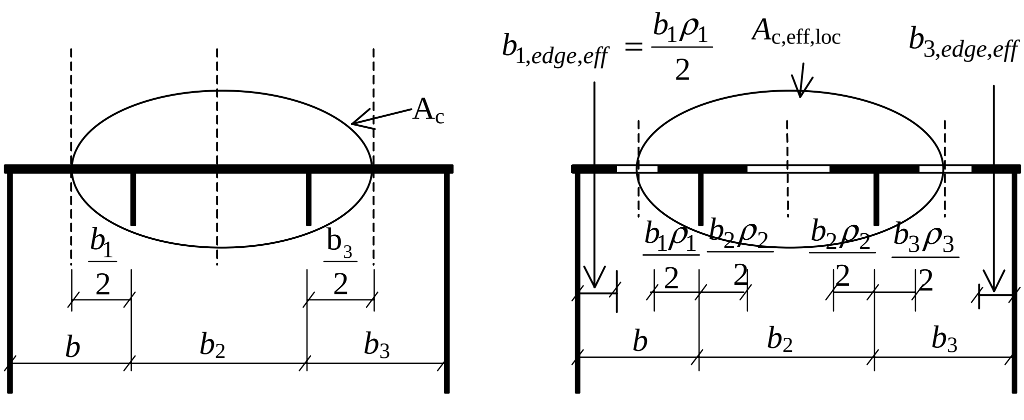


Figure 4.4: Stiffened plate under uniform compression

NOTE For non-uniform compression see Figure A.1.

(5) In determining the reduction factor ρ_c for overall buckling, the reduction factor for column-type buckling, which is more severe than the reduction factor than for plate buckling, should be considered.

(6) Interpolation should be carried out in accordance with 4.5.4(1) between the reduction factor ρ for plate buckling and the reduction factor χ_c for column buckling to determine ρ_c see 4.5.4.

(7) The reduction of the compressed area $A_{c,eff,loc}$ through ρ_c may be taken as a uniform reduction across the whole cross section.

(8) If shear lag is relevant (see 3.3), the effective cross-sectional area $A_{c,eff}$ of the compression zone of the stiffened plate should then be taken as $A^*_{c,eff}$ accounting not only for local plate buckling effects but also for shear lag effects.

(9) The effective cross-sectional area of the tension zone of the stiffened plate should be taken as the gross area of the tension zone reduced for shear lag if relevant, see 3.3.

(10) The effective section modulus W_{eff} should be taken as the second moment of area of the effective cross section divided by the distance from its centroid to the mid depth of the flange plate.

4.5.2 Plate type behaviour

(1) The relative plate slenderness $\bar{\lambda}_p$ of the equivalent plate is defined as:

$$\bar{\lambda}_p = \sqrt{\frac{\beta_{A,c}\ f_y}{\sigma_{cr,p}}} \tag{4.7}$$

with $\beta_{A,c} = \dfrac{A_{c,eff,loc}}{A_c}$

where A_c is the gross area of the compression zone of the stiffened plate except the parts of subpanels supported by an adjacent plate, see Figure 4.4 (to be multiplied by the shear lag factor if shear lag is relevant, see 3.3);

$A_{c,eff,loc}$ is the effective[p] area of the same part of the plate with due allowance made for possible plate buckling of subpanels and/or of stiffened plate.

(2) The reduction factor ρ for the equivalent orthotropic plate is obtained from 4.4(2) provided $\bar{\lambda}_p$ is calculated from equation (4.7).

NOTE For calculation of $\sigma_{cr,p}$ see Annex A.

4.5.3 Column type buckling behaviour

(1) The elastic critical column buckling stress $\sigma_{cr,c}$ of an unstiffened (see 4.4) or stiffened (see 4.5) plate should be taken as the buckling stress with the supports along the longitudinal edges removed.

(2) For an unstiffened plate the elastic critical column buckling stress $\sigma_{cr,c}$ ~~of an unstiffened plate~~ may be obtained from

$$\sigma_{cr,c} = \frac{\pi^2\ E\ t^2}{12\left(1-\nu^2\right)a^2} \tag{4.8}$$

(3) For a stiffened plate $\sigma_{cr,c}$ may be determined from the elastic critical column buckling stress $\sigma_{cr,sl}$ of the stiffener closest to the panel edge with the highest compressive stress as follows:

$$\sigma_{cr,s\ell} = \frac{\pi^2\ E\ I_{s\ell,1}}{A_{s\ell,1}\ a^2} \tag{4.9}$$

where $I_{s\ell,1}$ is the second moment of area of the stiffener, relative to the out-of-plane bending of the plate; ~~,~~

$A_{s\ell,1}$ is the gross cross-sectional area of the stiffener and the adjacent parts of the plate according to Figure A.1.

NOTE $\sigma_{cr,c}$ may be obtained from $\sigma_{cr,c} = \sigma_{cr,s\ell} \dfrac{b_c}{b_{s\ell,1}}$, where $\sigma_{cr,c}$ is related to the compressed edge of the plate, and , b_{sl1} and b_c are geometric values from the stress distribution used for the extrapolation, see Figure A.1.

(4) The relative column slenderness $\bar{\lambda}_c$ is defined as follows:

$$\bar{\lambda}_c = \sqrt{\frac{f_y}{\sigma_{cr,c}}} \quad \text{for unstiffened plates} \tag{4.10}$$

$$\bar{\lambda}_c = \sqrt{\frac{\beta_{A,c}\, f_y}{\sigma_{cr,c}}} \quad \text{for stiffened plates} \tag{4.11}$$

with $\beta_{A,c} = \dfrac{A_{s\ell,1,eff}}{A_{s\ell,1}}$;

$A_{s\ell,1}$ is defined in 4.5.3(3); ~~and~~

$A_{s\ell,1,eff}$ is the effective cross-sectional area of the stiffener with due allowance for plate buckling, see Figure A.1.

(5) The reduction factor χ_c should be obtained from 6.3.1.2 of EN 1993-1-1. For unstiffened plates $\alpha = 0{,}21$ corresponding to buckling curve a should be used. For stiffened plates its value should be increased to:

$$\alpha_e = \alpha + \frac{0{,}09}{i/e} \tag{4.12}$$

with $i = \sqrt{\dfrac{I_{s\ell,1}}{A_{s\ell,1}}}$

e = max (e_1, e_2) is the largest distance from the respective centroids of the plating and the one-sided stiffener (or of the centroids of either set of stiffeners when present on both sides) to the neutral axis of the effective column, see Figure A.1;

α = 0,34 (curve b) for closed section stiffeners;

= 0,49 (curve c) for open section stiffeners.

4.5.4 Interaction between plate and column buckling

(1) The final reduction factor ρ_c should be obtained by interpolation between χ_c and ρ as follows:

$$\rho_c = (\rho - \chi_c)\,\xi\,(2 - \xi) + \chi_c \tag{4.13}$$

where $\xi = \dfrac{\sigma_{cr,p}}{\sigma_{cr,c}} - 1 \quad \text{but} \quad 0 \le \xi \le 1$

$\sigma_{cr,p}$ is the elastic critical plate buckling stress, see Annex A.1(2);

$\sigma_{cr,c}$ is the elastic critical column buckling stress according to 4.5.3(2) and (3), respectively;

χ_c is the reduction factor due to column buckling.

4.6 Verification

(1) Member verification for uniaxial bending should be performed as follows:

$$\eta_1 = \frac{N_{Ed}}{\dfrac{f_y\, A_{eff}}{\gamma_{M0}}} + \frac{M_{Ed} + N_{Ed}\, e_N}{\dfrac{f_y\, W_{eff}}{\gamma_{M0}}} \leq 1{,}0 \tag{4.14}$$

where A_{eff} is the effective cross-section area in accordance with 4.3(3);

e_N is the shift in the position of neutral axis, see 4.3(3);

M_{Ed} is the design bending moment;

N_{Ed} is the design axial force;

W_{eff} is the effective elastic section modulus, see 4.3(4);

γ_{M0} is the partial factor, see application parts EN 1993-2 to 6.

NOTE For members subject to compression and biaxial bending the above equation (4.14) may be modified as follows:

$$\eta_1 = \frac{N_{Ed}}{\dfrac{f_y\, A_{eff}}{\gamma_{M0}}} + \frac{M_{y,Ed} + N_{Ed}\, e_{y,N}}{\dfrac{f_y\, W_{y,eff}}{\gamma_{M0}}} + \frac{M_{z,Ed} + N_{Ed}\, e_{z,N}}{\dfrac{f_y\, W_{z,eff}}{\gamma_{M0}}} \leq 1{,}0 \tag{4.15}$$

$M_{y,Ed}$, $M_{z,Ed}$ are the design bending moments with respect to y and z axes respectively;

e_{yN}, e_{zN} are the eccentricities with respect to the neutral axis.

(2) Action effects M_{Ed} and N_{Ed} should include global second order effects where relevant.

(3) The plate buckling verification of the panel should be carried out for the stress resultants at a distance 0,4a or 0,5b, whichever is the smallest, from the panel end where the stresses are the greater. In this case the gross sectional resistance needs to be checked at the end of the panel.

5 Resistance to shear

5.1 Basis

(1) This section gives rules for shear resistance of plates considering shear buckling at the ultimate limit state where the following criteria are met:

a) the panels are rectangular within the angle limit stated in 2.3;

b) stiffeners, if any, are provided in the longitudinal or transverse direction or both;

c) all holes and cut outs are small (see 2.3);

d) members are of uniform cross section.

(2) Plates with h_w/t greater than $\dfrac{72}{\eta}\varepsilon$ for an unstiffened web, or $\dfrac{31}{\eta}\varepsilon\sqrt{k_\tau}$ for a stiffened web, should be checked for resistance to shear buckling and should be provided with transverse stiffeners at the supports, where $\varepsilon = \sqrt{\dfrac{235}{f_y\left[N/mm^2\right]}}$.

NOTE 1 h_w see Figure 5.1 and for k_τ see 5.3(3).

NOTE 2 The National Annex will define η. The value η = 1,20 is recommended for steel grades up to and including S460. For higher steel grades η = 1,00 is recommended.

5.2 Design resistance

(1) For unstiffened or stiffened webs the design resistance for shear should be taken as:

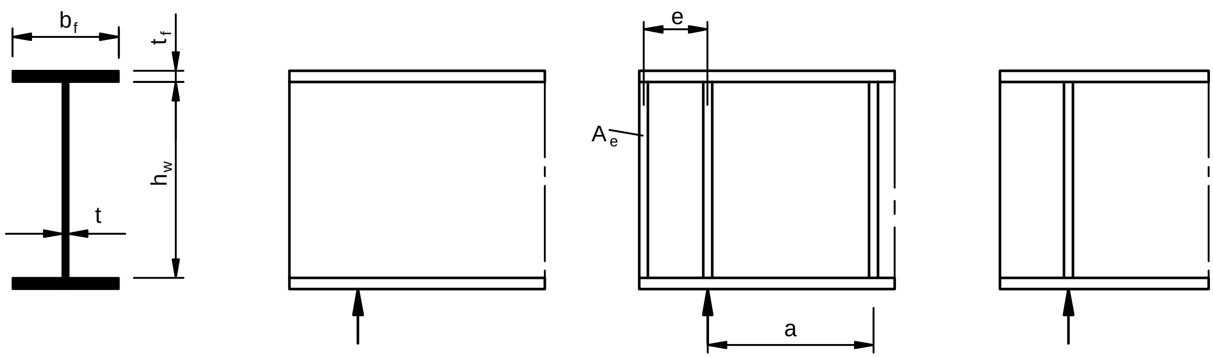
$$V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} \leq \frac{\eta\, f_{yw}\, h_w\, t}{\sqrt{3}\,\gamma_{M1}} \tag{5.1}$$

in which the contribution from the web is given by:

$$V_{bw,Rd} = \frac{\chi_w\, f_{yw}\, h_w\, t}{\sqrt{3}\,\gamma_{M1}} \tag{5.2}$$

and the contribution from the flanges $V_{bf,Rd}$ is according to 5.4.

(2) Stiffeners should comply with the requirements in 9.3 and welds should fulfil the requirement given in 9.3.5.



Cross section notations a) No end post b) Rigid end post c) Non-rigid end post

Figure 5.1: End supports

5.3 Contribution from the web

(1) For webs with transverse stiffeners at supports only and for webs with either intermediate transverse stiffeners 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 5.1 or Figure 5.2.

Table 5.1: Contribution from the web χ_w to shear buckling resistance

	Rigid end post	Non-rigid end post
$\bar{\lambda}_w < 0{,}83/\eta$	η	η
$0{,}83/\eta \leq \bar{\lambda}_w < 1{,}08$	$0{,}83/\bar{\lambda}_w$	$0{,}83/\bar{\lambda}_w$
$\bar{\lambda}_w \geq 1{,}08$	$1{,}37/(0{,}7+\bar{\lambda}_w)$	$0{,}83/\bar{\lambda}_w$

NOTE See 6.2.6 in EN 1993-1-1.

(2) Figure 5.1 shows various end supports for girders:

a) No end post, see 6.1 (2), type c);

b) Rigid end posts, see 9.3.1; this case is also applicable for panels at an intermediate support of a continuous girder;

c) Non rigid end posts, see 9.3.2.

(3) The slenderness parameter $\overline{\lambda}_w$ in Table 5.1 and Figure 5.2 should be taken as:

$$\overline{\lambda}_w = 0{,}76 \sqrt{\frac{f_{yw}}{\tau_{cr}}} \tag{5.3}$$

where $\tau_{cr} = k_\tau\ \sigma_E$ (5.4)

NOTE 1 Values for σ_E and k_τ may be taken from Annex A.

NOTE 2 The slenderness parameter $\overline{\lambda}_w$ may be taken as follows:

a) transverse stiffeners at supports only:

$$\overline{\lambda}_w = \frac{h_w}{86{,}4\ t\ \varepsilon} \tag{5.5}$$

b) transverse stiffeners at supports and intermediate transverse or longitudinal stiffeners or both:

$$\overline{\lambda}_w = \frac{h_w}{37{,}4\ t\ \varepsilon \sqrt{k_\tau}} \tag{5.6}$$

in which k_τ is the minimum shear buckling coefficient for the web panel.

NOTE 3 Where non-rigid transverse stiffeners are also used in addition to rigid transverse stiffeners, k_τ is taken as the minimum of the values from the web panels between any two transverse stiffeners (e.g. $a_2 \times h_w$ and $a_3 \times h_w$) and that between two rigid stiffeners containing non-rigid transverse stiffeners (e.g. $a_4 \times h_w$).

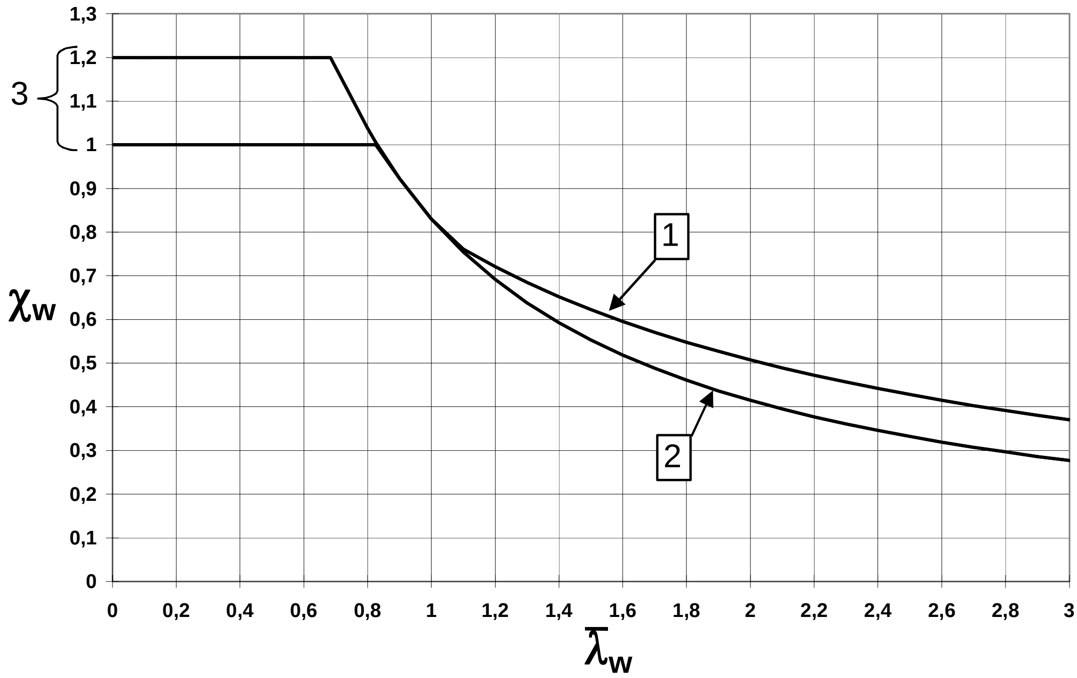
NOTE 4 Rigid boundaries may be assumed for panels bordered by flanges and rigid transverse stiffeners. The web buckling analysis can then be based on the panels between two adjacent transverse stiffeners (e.g. $a_1 \times h_w$ in Figure 5.3).

NOTE 5 For non-rigid transverse stiffeners the minimum value k_τ may be obtained from the buckling analysis of the following:

1. a combination of two adjacent web panels with one flexible transverse stiffener
2. a combination of three adjacent web panels with two flexible transverse stiffeners

For procedure to determine k_τ see Annex A.3.

(4) The second moment of area of a longitudinal stiffener should be reduced to 1/3 of their actual value when calculating k_τ. Formulae for k_τ taking this reduction into account in A.3 may be used.



1 Rigid end post
2 Non-rigid end post
3 Range of recommended η

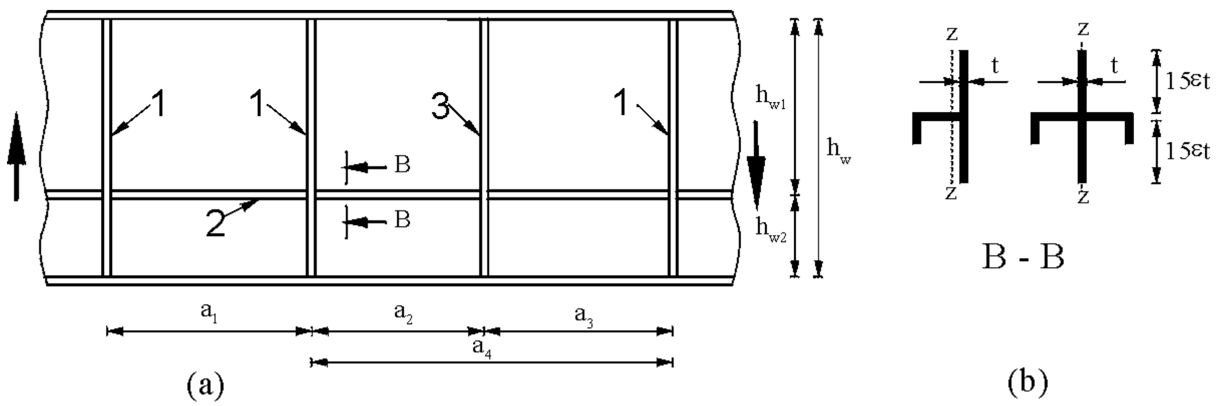
Figure 5.2: Shear buckling factor χ_w

(5) For webs with longitudinal stiffeners the slenderness parameter $\bar{\lambda}_w$ in (3) should not be taken as less than

$$\bar{\lambda}_w = \frac{h_{wi}}{37{,}4\ t\ \varepsilon \sqrt{k_{\tau i}}} \tag{5.7}$$

where h_{wi} and $k_{\tau i}$ refer to the subpanel with the largest slenderness parameter $\bar{\lambda}_w$ of all subpanels within the web panel under consideration.

NOTE To calculate $k_{\tau i}$ the expression given in A.3 may be used with $k_{\tau st} = 0$.



1 Rigid transverse stiffener
2 Longitudinal stiffener
3 Non-rigid transverse stiffener

Figure 5.3: Web with transverse and longitudinal stiffeners

5.4 Contribution from flanges

(1) When the flange resistance is not completely utilized in resisting the bending moment ($M_{Ed} < M_{f,Rd}$) the contribution from the flanges should be obtained as follows:

$$V_{bf,Rd} = \frac{b_f\, t_f^2\, f_{yf}}{c\, \gamma_{M1}} \left(1 - \left(\frac{M_{Ed}}{M_{f,Rd}}\right)^2\right) \qquad (5.8)$$

b_f and t_f are taken for the flange which provides the least axial resistance,

b_f being taken as not larger than $15\varepsilon t_f$ on each side of the web,

$M_{f,Rd} = \dfrac{M_{f,k}}{\gamma_{M0}}$ is the moment of resistance of the cross section consisting of the area of the effective flanges only,

$$c = a\left(0{,}25 + \frac{1{,}6\, b_f\, t_f^2\, f_{yf}}{t\, h_w^2\, f_{yw}}\right)$$

(2) When an axial force N_{Ed} is present, the value of $M_{f,Rd}$ should be reduced by multiplying it by the following factor:

$$\left(1 - \frac{N_{Ed}}{\dfrac{(A_{f1} + A_{f2})\, f_{yf}}{\gamma_{M0}}}\right) \cancel{\left(1 - \frac{N_{Ed}}{\dfrac{(A_{f1} + A_{f2})\, f_{yf}}{\gamma_{M2}}}\right)} \qquad (5.9)$$

where A_{f1} and A_{f2} are the areas of the top and bottom flanges respectively.

5.5 Verification

(1) The verification should be performed as follows:

$$\eta_3 = \frac{V_{Ed}}{V_{b,Rd}} \leq 1{,}0 \qquad (5.10)$$

where V_{Ed} is the design shear force including shear from torque.

6 Resistance to transverse forces

6.1 Basis

(1) The design resistance of the webs of rolled beams and welded girders should be determined in accordance with 6.2, provided that the compression flange is adequately restrained in the lateral direction.

(2) The load is applied as follows:

a) through the flange and resisted by shear forces in the web, see Figure 6.1 (a);

b) through one flange and transferred through the web directly to the other flange, see Figure 6.1 (b).

c) through one flange adjacent to an unstiffened end, see Figure 6.1 (c)

(3) For box girders with inclined webs the resistance of both the web and flange should be checked. The internal forces to be taken into account are the components of the external load in the plane of the web and flange respectively.

(4) The interaction of the transverse force, bending moment and axial force should be verified using 7.2.

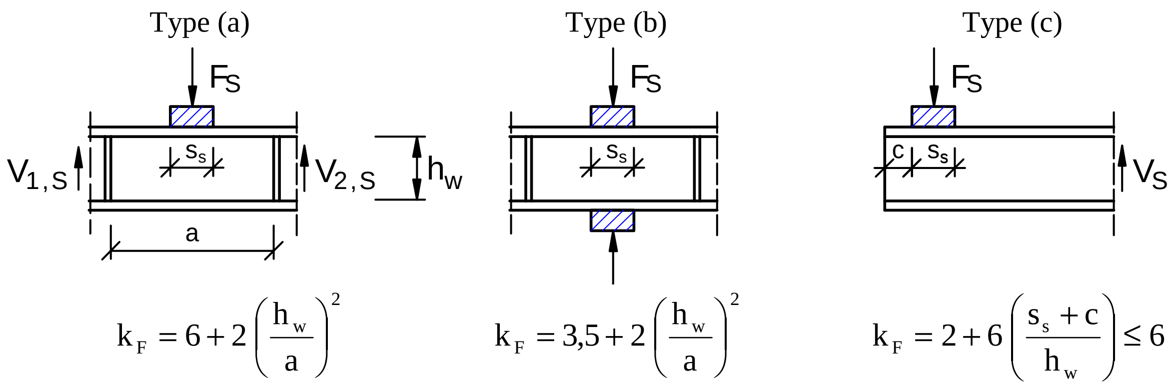


Figure 6.1: Buckling coefficients for different types of load application

6.2 Design resistance

(1) For unstiffened or stiffened webs the design resistance to local buckling under transverse forces should be taken as

$$F_{Rd} = \frac{f_{yw} L_{eff} t_w}{\gamma_{M1}} \tag{6.1}$$

where t_w is the thickness of the web;

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

L_{eff} is the effective length for resistance to transverse forces, which should be determined from

$$L_{eff} = \chi_F \ell_y \tag{6.2}$$

where ℓ_y is the effective loaded length, see 6.5, appropriate to the length of stiff bearing s_s, see 6.3;

χ_F is the reduction factor due to local buckling, see 6.4(1).

6.3 Length of stiff bearing

(1) The length of stiff bearing s_s on the flange should be taken as the distance over which the applied load is effectively distributed at a slope of 1:1, see Figure 6.2. However, s_s should not be taken as larger than h_w.

(2) If several concentrated forces are closely spaced, the resistance should be checked for each individual force as well as for the total load with s_s as the centre-to-centre distance between the outer loads.

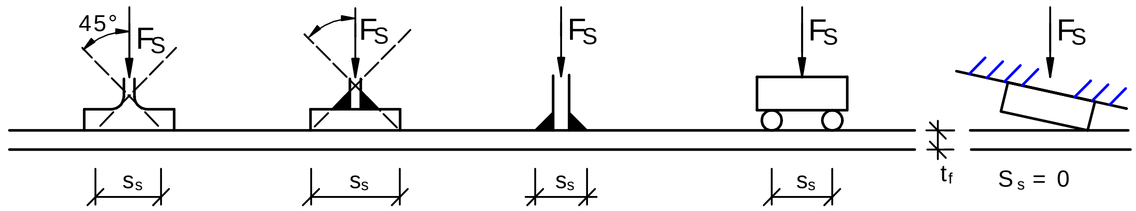


Figure 6.2: Length of stiff bearing

(3) If the bearing surface of the applied load rests at an angle to the flange surface, see Figure 6.2, s_s should be taken as zero.

6.4 Reduction factor χ_F for effective length for resistance

(1) The reduction factor χ_F should be obtained from:

$$\chi_F = \frac{0,5}{\overline{\lambda}_F} \leq 1,0 \tag{6.3}$$

where $$\overline{\lambda}_F = \sqrt{\frac{\ell_y t_w f_{yw}}{F_{cr}}} \tag{6.4}$$

$$F_{cr} = 0,9\, k_F E \frac{t_w^3}{h_w} \tag{6.5}$$

(2) For webs without longitudinal stiffeners k_F should be obtained from Figure 6.1.

NOTE For webs with longitudinal stiffeners information may be given in the National Annex. The following rules are recommended:

For webs with longitudinal stiffeners k_F may be taken as

$$k_F = 6 + 2\left[\frac{h_w}{a}\right]^2 + \left[5,44\frac{b_1}{a} - 0,21\right]\sqrt{\gamma_s} \tag{6.6}$$

where b_1 is the depth of the loaded subpanel taken as the clear distance between the loaded flange and the stiffener

$$\gamma_s = 10,9\frac{I_{s\ell,1}}{h_w t_w^3} \leq 13\left[\frac{a}{h_w}\right]^3 + 210\left[0,3 - \frac{b_1}{a}\right] \tag{6.7}$$

where $I_{s\ell,1}$ is the second moment~~s~~ of area of the stiffener closest to the loaded flange including contributing parts of the web according to Figure 9.1.

Equation (6.6) is valid for $0,05 \leq \frac{b_1}{h_w} \leq 0,3$ and $\frac{b_1}{a} \leq 0,3$ and loading according to type a) in Figure 6.1.

(3) ℓ_y should be obtained from 6.5.

6.5 Effective loaded length

(1) The effective loaded length ℓ_y should be calculated as follows:

$$m_1 = \frac{f_{yf} b_f}{f_{yw} t_w} \tag{6.8}$$

$$m_2 = 0,02\left(\frac{h_w}{t_f}\right)^2 \quad \text{if } \overline{\lambda}_F > 0,5$$
$$m_2 = 0 \quad \text{if } \overline{\lambda}_F \leq 0,5 \tag{6.9}$$

For box girders, b_f in equation (6.8) should be limited to $15\varepsilon t_f$ on each side of the web.

(2) For types ~~(~~a) and ~~(~~b) in Figure 6.1, ℓ_y should be obtained using:

$$\ell_y = s_s + 2\, t_f \left(1 + \sqrt{m_1 + m_2}\right) \text{ , but } \ell_y \leq \text{ distance between adjacent transverse stiffeners} \qquad (6.10)$$

(3) For type c) ℓ_y should be taken as the smallest value obtained from the equations (6.11), (6.12) and (6.13).

$$\ell_y = \ell_e + t_f \sqrt{\frac{m_1}{2} + \left(\frac{\ell_e}{t_f}\right)^2 + m_2} \qquad (6.11)$$

$$\ell_y = \ell_e + t_f \sqrt{m_1 + m_2} \qquad (6.12)$$

$$\ell_e = \frac{k_F E\, t_w^2}{2\, f_{yw} h_w} \leq s_s + c \qquad (6.13)$$

6.6 Verification

(1) The verification should be performed as follows:

$$\eta_2 = \frac{F_{Ed}}{\dfrac{f_{yw}\, L_{eff}\, t_w}{\gamma_{M1}}} \leq 1{,}0 \qquad (6.14)$$

where F_{Ed} is the design transverse force;

L_{eff} is the effective length for resistance to transverse forces, see 6.2(2);

t_w is the thickness of the plate.

7 Interaction

7.1 Interaction between shear force, bending moment and axial force

(1) Provided that $\bar{\eta}_3$ (see below) does not exceed 0,5 , the design resistance to bending moment and axial force need not be reduced to allow for the shear force. If $\bar{\eta}_3$ is more than 0,5 the combined effects of bending and shear in the web of an I or box girder should satisfy:

$$\bar{\eta}_1 + \left(1 - \frac{M_{f,Rd}}{M_{pl,Rd}}\right)\left(2\bar{\eta}_3 - 1\right)^2 \leq 1{,}0 \qquad \text{for } \bar{\eta}_1 \geq \frac{M_{f,Rd}}{M_{pl,Rd}} \qquad (7.1)$$

where $M_{f,Rd}$ is the design plastic moment of resistance of the section consisting of the effective area of the flanges;

$M_{pl,Rd}$ is the design plastic resistance of the cross section consisting of the effective area of the flanges and the fully effective web irrespective of its section class.

$$\bar{\eta}_1 = \frac{M_{Ed}}{M_{pl,Rd}}$$

$$\bar{\eta}_3 = \frac{V_{Ed}}{V_{bw,Rd}}$$

In addition the requirements in sections 4.6 and 5.5 should be met.

Action effects should include global second order effects of members where relevant.

(2) The criterion given in (1) should be verified at all sections other than those located at a distance less than $h_w/2$ from ~~the interior~~ a support with vertical stiffeners.

(3) The plastic moment of resistance $M_{f,Rd}$ ~~should~~ may be taken as the product of the ~~design~~ yield strength, the effective area of the flange with the smallest value of $A_f f_y / \gamma_{M0}$ and the distance between the centroids of the flanges.

(4) If an axial force N_{Ed} is present, $M_{pl,Rd}$ and $M_{f,Rd}$ should be reduced in accordance with 6.2.9 of EN 1993-1-1 and 5.4(2) respectively. When the axial force is so large that the whole web is in compression 7.1(5) should be applied.

(5) A flange in a box girder should be verified using 7.1(1) taking $M_{f,Rd} = 0$ and τ_{Ed} taken as the average shear stress in the flange which should not be less than half the maximum shear stress in the flange and $\bar{\eta}_1$ is taken as η_1 according to 4.6(1). In addition the subpanels should be checked using the average shear stress within the subpanel and χ_w determined for shear buckling of the subpanel according to 5.3, assuming the longitudinal stiffeners to be rigid.

7.2 Interaction between transverse force, bending moment and axial force

(1) If the girder is subjected to a concentrated transverse force acting on the compression flange in conjunction with bending and axial force, the resistance should be verified using 4.6, 6.6 and the following interaction expression:

$$\eta_2 + 0{,}8\,\eta_1 \le 1{,}4 \qquad (7.2)$$

(2) If the concentrated load is acting on the tension flange the resistance should be verified according to section 6. Additionally 6.2.1(5) of EN 1993-1-1 should be met.

8 Flange induced buckling

(1) To prevent the compression flange buckling in the plane of the web, the following criterion should be met:

$$\frac{h_w}{t_w} \le k \frac{E}{f_{yf}} \sqrt{\frac{A_w}{A_{fc}}} \qquad (8.1)$$

where A_w is the cross section area of the web;~~,~~

A_{fc} is the effective cross section area of the compression flange;~~,~~

h_w is the depth of the web;~~,~~

t_w is the thickness of the web.

The value of the factor k should be taken as follows:

- plastic rotation utilized k = 0,3
- plastic moment resistance utilized k = 0,4
- elastic moment resistance utilized k = 0,55

(2) When the girder is curved in elevation, with the compression flange on the concave face, the following criterion should be met:

$$\frac{h_w}{t_w} \le \frac{k \dfrac{E}{f_{yf}} \sqrt{\dfrac{A_w}{A_{fc}}}}{\sqrt{1 + \dfrac{h_w E}{3 r f_{yf}}}} \qquad (8.2)$$

r is the radius of curvature of the compression flange.

NOTE The National Annex may give further information on flange induced buckling.

9 Stiffeners and detailing

9.1 General

(1) This section gives design rules for stiffeners in plated structures which supplement the plate buckling rules specified in sections 4 to 7.

NOTE The National Annex may give further requirements on stiffeners for specific applications.

(2) When checking the buckling resistance, the section of a stiffener may be taken as the gross area comprising the stiffener plus a width of plate equal to 15εt but not more than the actual dimension available, on each side of the stiffener avoiding any overlap of contributing parts to adjacent stiffeners, see Figure 9.1.

(3) The axial force in a transverse stiffener should be taken as the sum of the force resulting from shear (see 9.3.3(3)) and any other point load.

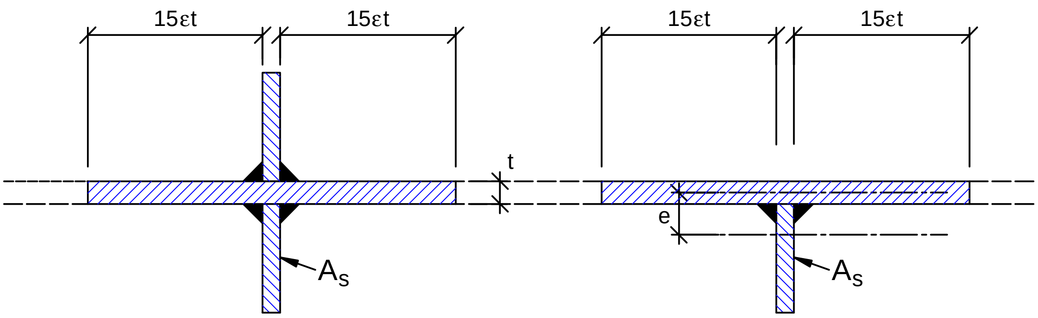


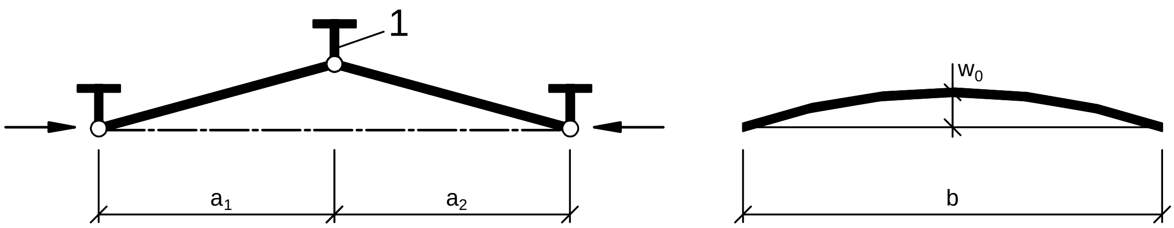
Figure 9.1: Effective cross-section of stiffener

9.2 Direct stresses

9.2.1 Minimum requirements for transverse stiffeners

(1) In order to provide a rigid support for a plate with or without longitudinal stiffeners, intermediate transverse stiffeners should satisfy the criteria given below.

(2) The transverse stiffener should be treated as a simply supported ~~beam~~ member with an initial sinusoidal imperfection w_0 equal to s/300, where s is the smallest of a_1, a_2 or b, see Figure 9.2 , where a_1 and a_2 are the lengths of the panels adjacent to the transverse stiffener under consideration and b is the depth or span of the transverse stiffener. Eccentricities should be accounted for.



1 Transverse stiffener

Figure 9.2: Transverse stiffener

(3) The transverse stiffener should carry the deviation forces from the adjacent compressed panels under the assumption that both adjacent transverse stiffeners are rigid and straight together with any external load and axial force according to the NOTE to 9.3.3(3). The compressed panels and the longitudinal stiffeners are considered to be simply supported at the transverse stiffeners.

(4) It should be verified that using a second order elastic method analysis both the following criteria are satisfied at the ultimate limit state:

– that the maximum stress in the stiffener should not exceed f_y/γ_{M1};

– that the additional deflection should not exceed b/300.

(5) In ~~the~~ absence of an axial force ~~or/and transverse loads~~ in the transverse stiffener both the criteria in (4) above may be assumed to be satisfied provided that the second moment of area I_{st} of the transverse stiffeners is not less than:

$$I_{st} = \frac{\sigma_m}{E}\left(\frac{b}{\pi}\right)^4 \left(1 + w_0 \frac{300}{b} u\right) \tag{9.1}$$

where $\sigma_m = \frac{\sigma_{cr,c}}{\sigma_{cr,p}} \frac{N_{Ed}}{b}\left(\frac{1}{a_1} + \frac{1}{a_2}\right)$

$$u = \frac{\pi^2 \, E \, e_{max}}{\frac{f_y \; 300 \, b}{\gamma_{M1}}} \geq 1{,}0$$

e_{max} is the maximum distance from the extreme fibre of the stiffener to the centroid of the stiffener;

N_{Ed} is the maximum compressive force of the adjacent panels but not less than the maximum compressive stress times half the effective[p] compression area of the panel including stiffeners;

$\sigma_{cr,c}$, $\sigma_{cr,p}$ are defined in 4.5.3 and Annex A.

NOTE Where out of plane loading is applied to the transverse stiffeners reference should be made to EN 1993-2 and EN 1993-1-7.

(6) If the stiffener carries axial compression this should be increased by $\Delta N_{st} = \sigma_m b^2 / \pi^2$ in order to account for deviation forces. The criteria in (4) apply but ΔN_{st} need not be considered when calculating the uniform stresses from axial load in the stiffener.

(7) As a simplification the requirement of (4) may in absence of axial forces be verified using a first order elastic analysis taking account of the following additional uniformly distributed lateral load q acting on the length b:

$$q = \frac{\pi}{4} \sigma_m (w_0 + w_{el}) \tag{9.2}$$

where σ_m is defined in (5) above;

w_0 is defined in Figure 9.2;

w_{el} is the elastic deformation, that may be either determined iteratively or be taken as the maximum additional deflection b/300.

(8~~7~~) Unless a more advanced method of analysis is carried out in order to prevent torsional buckling of stiffeners with open cross-sections, the following criterion should be satisfied:

$$\frac{I_T}{I_p} \geq 5{,}3 \frac{f_y}{E} \qquad (9.3)$$

where I_p is the polar second moment of area of the stiffener alone around the edge fixed to the plate;

I_T is the St. Venant torsional constant for the stiffener alone.

(9~~8~~) Where warping stiffness is considered stiffeners should either fulfil (8~~7~~) or the criterion

$$\sigma_{cr} \geq \theta\, f_y \qquad (9.4)$$

where σ_{cr} is the elastic critical stress for torsional buckling not considering rotational restraint from the plate;

θ is a parameter to ensure class 3 behaviour.

NOTE The parameter θ may be given in the National Annex. The value $\theta = 6$ is recommended.

9.2.2 Minimum requirements for longitudinal stiffeners

(1) The requirements concerning torsional buckling in 9.2.1(7) and (8) also apply to longitudinal stiffeners.

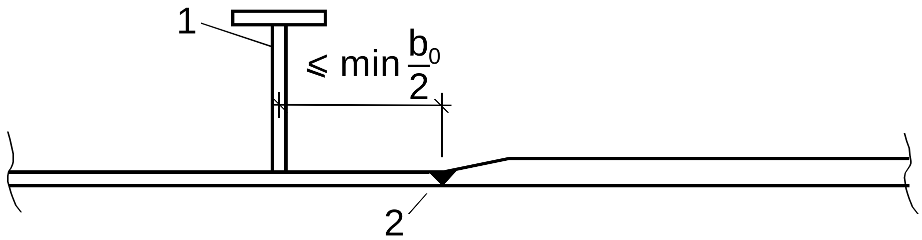
(2) Discontinuous longitudinal stiffeners that do not pass through openings made in the transverse stiffeners or are not connected to either side of the transverse stiffeners should be:

- used only for webs (i.e. not allowed in flanges);
- neglected in global analysis;
- neglected in the calculation of stresses;
- considered in the calculation of the effective[p] widths of web sub-panels;
- considered in the calculation of the critical stresses.

(3) Strength assessments for stiffeners should be performed according to 4.5.3 and 4.6.

9.2.3 Welded plates

(1) Plates with changes in plate thickness should be welded adjacent to the transverse stiffener, see Figure 9.3. The effects of eccentricity need not be taken into account unless the distance to the stiffener from the welded junction exceeds $b_0/2$ or 200 mm whichever is the smallest, where b_0 is the width of the plate between longitudinal stiffeners.



1 Transverse stiffener
2 Transverse weld

Figure 9.3: Welded plates

9.2.4 Cut outs in stiffeners

(1) The dimensions of ~~C~~cut outs in longitudinal stiffeners should be as shown in Figure 9.4.

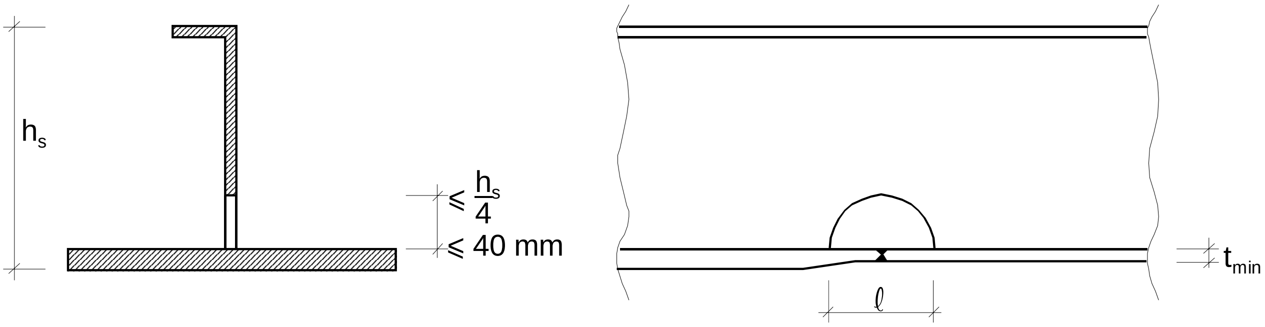


Figure 9.4: Cut outs in longitudinal stiffeners

(2) The length ℓ should not exceed:

$\ell \leq 6\,t_{min}$ for flat stiffeners in compression

$\ell \leq 8\,t_{min}$ for other stiffeners in compression

$\ell \leq 15\,t_{min}$ for stiffeners without compression

where t_{min} is the lesser of the plate thicknesses

(3) The values ℓ in (2) for stiffeners in compression may be increased by $\sqrt{\frac{\sigma_{x,Rd}}{\sigma_{x,Ed}}}$ when $\sigma_{x,Ed} \leq \sigma_{x,Rd}$ and $\ell \leq 15 t_{min}$.

(4) The dimensions of ~~C~~cut outs in transverse stiffeners should be as shown in Figure 9.5.

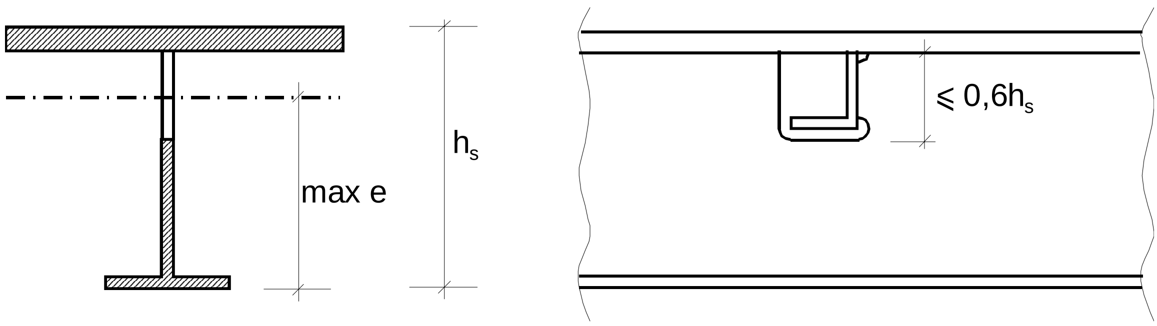


Figure 9.5: Cut outs in transverse stiffeners

(5) The gross web adjacent to the cut out should resist a shear force V, where

$$V = \frac{I_{net}}{\max e} \frac{f_{yk}}{\gamma_{M0}} \frac{\pi}{b_G} \qquad (9.5)$$

I_{net} is the second moment of area for the net section of the transverse stiffener;

e is the maximum distance from the underside of the flange plate to the neutral axis of net section, see Figure 9.5;

b_G is the length of the transverse stiffener between the flanges.

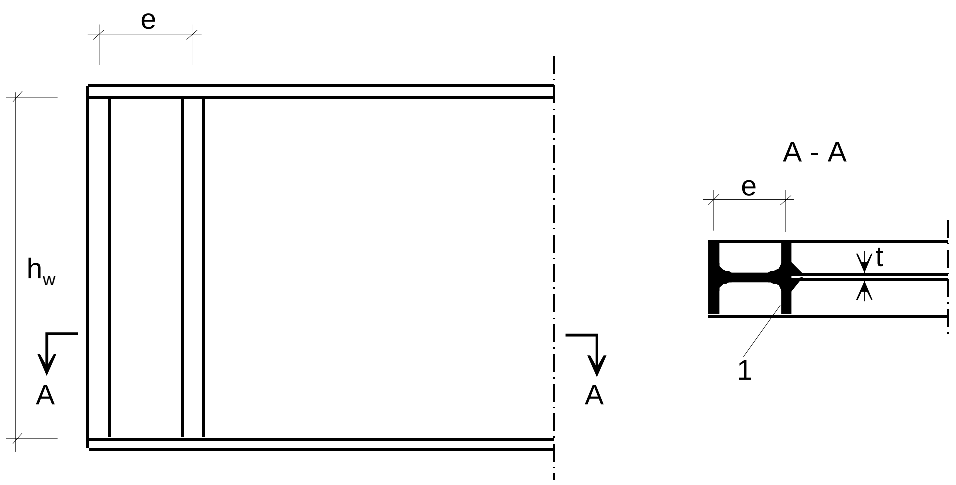
9.3 Shear

9.3.1 Rigid end post

(1) The rigid end post (see Figure 5.1) should act as a bearing stiffener resisting the reaction from the support (see 9.4), and should be designed as a short beam resisting the longitudinal membrane stresses in the plane of the web.

NOTE For the effects of eccentricity due to movements of bearings, see EN 1993-2.

(2) A rigid end post should comprise of two double-sided transverse stiffeners that form the flanges of a short beam of length h_w, see Figure 5.1 (b). The strip of web plate between the stiffeners forms the web of the short beam. Alternatively, a~~n~~ rigid end post may be in the form of a rolled section, connected to the end of the web plate as shown in Figure 9.6.



1 Inserted section

Figure 9.6: Rolled section forming an end-post

(3) Each double sided stiffener consisting of flats should have a cross sectional area of at least $4h_w t^2 / e$, where e is the centre to centre distance between the stiffeners and $e > 0{,}1\, h_w$, see Figure 5.1 (b). Where a rolled section other than flats is used for the end-post its section modulus should be not less than $4h_w t^2$ for bending around a horizontal axis perpendicular to the web.

(4) As an alternative the girder end may be provided with a single double-sided stiffener and a vertical stiffener adjacent to the support so that the subpanel resists the maximum shear when designed with a non-rigid end post.

9.3.2 Stiffeners acting as non-rigid end post

(1) A non-rigid end post may be a single double sided stiffener as shown in Figure 5.1 (c). It may act as a bearing stiffener resisting the reaction at the girder support (see 9.4).

9.3.3 Intermediate transverse stiffeners

(1) Intermediate stiffeners that act as rigid supports to interior panels of the web should be designed for strength and stiffness.

(2) When flexible intermediate transverse stiffeners are used, their stiffness should be considered in the calculation of k_τ in 5.3(5).

(3) The effective section of intermediate stiffeners acting as rigid supports for web panels should have a minimum second moment of area I_{st}:

$$\begin{aligned} &\text{if } a/h_w < \sqrt{2}: \quad I_{st} \geq 1{,}5\, h_w^3\, t^3 / a^2 \\ &\text{if } a/h_w \geq \sqrt{2}: \quad I_{st} \geq 0{,}75\, h_w\, t^3 \end{aligned} \tag{9.6}$$

NOTE Intermediate rigid stiffeners may be designed for an axial force equal to $\left(V_{Ed} - \frac{1}{\bar{\lambda}_w^2} f_{yw} h_w t / \left(\sqrt{3}\,\gamma_{M1}\right)\right)$ according to ~~9.4~~9.2.1(3). In the case of variable shear forces the check is performed for the shear force at the distance $0{,}5h_w$ from the edge of the panel with the largest shear force.

9.3.4 Longitudinal stiffeners

(1) If longitudinal stiffeners are taken into account in the stress analysis they should be checked for direct stresses for the cross sectional resistance.

9.3.5 Welds

(1) The web to flange welds may be designed for the nominal shear flow V_{Ed}/h_w if V_{Ed} does not exceed $\chi_w f_{yw} h_w t / \left(\sqrt{3}\,\gamma_{M1}\right)$. For larger values V_{Ed} the weld between flanges and webs should be designed for the shear flow $\eta\, f_{yw} t / \left(\sqrt{3}\,\gamma_{M1}\right)$.

(2) In all other cases welds should be designed to transfer forces along and across welds making up sections taking into account analysis method (elastic/plastic) and second order effects.

9.4 Transverse loads

(1) If the design resistance of an unstiffened web is insufficient, transverse stiffeners should be provided.

(2) The out-of-plane buckling resistance of the transverse stiffener under transverse loads and shear force (see 9.3.3(3)) should be determined from 6.3.3 or 6.3.4 of EN 1993-1-1, using buckling curve c. when both end are assumed to be fixed laterally a buckling length ℓ of not less than $0{,}75h_w$ should be used. A larger value of ℓ should be used for conditions that provide less end restraint. If the stiffeners have cut outs at the loaded end, the cross sectional resistance should be checked at ~~the~~ this end.

(3) Where single sided or other asymmetric stiffeners are used, the resulting eccentricity should be allowed for using 6.3.3 or 6.3.4 of EN 1993-1-1. If the stiffeners are assumed to provide lateral restraint to the compression flange they should comply with the stiffness and strength criteria in the design for lateral torsional buckling.

10 Reduced stress method

(1) The reduced stress method may be used to determine the stress limits for stiffened or unstiffened plates.

NOTE 1 This method is an alternative to the effective width method specified in section 4 to 7 in respect of the following:

- $\sigma_{x,Ed}$, $\sigma_{z,Ed}$ and τ_{Ed} are considered as acting together
- the stress limits of the weakest part of the cross section may govern the resistance of the full cross section.

NOTE 2 The stress limits may also be used to determine equivalent effective areas. The National Annex may give limits of application for the methods.

(2) For unstiffened or stiffened panels subjected to combined stresses $\sigma_{x,Ed}$, $\sigma_{z,Ed}$ and τ_{Ed} class 3 section properties may be assumed, where

$$\frac{\rho\,\alpha_{ult,k}}{\gamma_{M1}} \geq 1 \tag{10.1}$$

where $\alpha_{ult,k}$ is the minimum load amplifier for the design loads to reach the characteristic value of resistance of the most critical point of the plate, see (4);

ρ is the reduction factor depending on the plate slenderness $\bar{\lambda}_p$ to take account of plate buckling, see (5);

γ_{M1} is the partial factor applied to this method.

(3) The plate slenderness $\bar{\lambda}_p$ should be taken from

$$\bar{\lambda}_p = \sqrt{\frac{\alpha_{ult,k}}{\alpha_{cr}}} \tag{10.2}$$

where α_{cr} is the minimum load amplifier for the design loads to reach the elastic critical ~~resistance~~ load of the plate under the complete stress field, see (6)

NOTE 1 For calculating α_{cr} for the complete stress field of the stiffened plate may be modelled using the rules in section 4 and 5 without reduction of the second moment of area of longitudinal stiffeners as specified in 5.3(4).

NOTE 2 When α_{cr} cannot be determined for the panel and its subpanels as a whole, separate checks for the subpanel and the full panel may be applied.

(4) In determining $\alpha_{ult,k}$ the yield criterion may be used for resistance:

$$\frac{1}{\alpha_{ult,k}^2} = \left(\frac{\sigma_{x,Ed}}{f_y}\right)^2 + \left(\frac{\sigma_{z,Ed}}{f_y}\right)^2 - \left(\frac{\sigma_{x,Ed}}{f_y}\right)\left(\frac{\sigma_{z,Ed}}{f_y}\right) + 3\left(\frac{\tau_{Ed}}{f_y}\right)^2 \tag{10.3}$$

where $\sigma_{x,Ed}$, $\sigma_{z,Ed}$ and τ_{Ed} are the components of the stress field in the ultimate limit state.

NOTE By using the equation (10.3) it is assumed that the resistance is reached when yielding occurs without plate buckling.

(5) The reduction factor ρ may be determined using either of the following methods:

a) the minimum value of the following reduction factors:

ρ_x for longitudinal stresses from 4.5.4(1) taking into account columnlike behaviour where relevant;

ρ_z for transverse stresses from 4.5.4(1) taking into account columnlike behaviour where relevant;

$\chi_{w\cancel{v}}$ for shear stresses from 5.2(1);

each calculated for the slenderness $\overline{\lambda}_p$ according to equation (10.2).

NOTE This method leads to the verification formula:

$$\left(\frac{\sigma_{x,Ed}}{f_y/\gamma_{M1}}\right)^2+\left(\frac{\sigma_{z,Ed}}{f_y/\gamma_{M1}}\right)^2-\left(\frac{\sigma_{x,Ed}}{f_y/\gamma_{M1}}\right)\left(\frac{\sigma_{z,Ed}}{f_y/\gamma_{M1}}\right)+3\left(\frac{\tau_{Ed}}{f_y/\gamma_{M1}}\right)^2\leq\rho^2 \qquad (10.4)$$

NOTE For determining ρ_z for transverse stresses the rules in section 4 for direct stresses σ_x should be applied to σ_z in the z-direction. For consistency section 6 should not be applied.

b) a value interpolated between the values of ρ_x, ρ_z and $\chi_{w\cancel{v}}$ as determined in a) by using the formula for $\alpha_{ult,k}$ as interpolation function

NOTE This method leads to the verification format:

$$\left(\frac{\sigma_{x,Ed}}{\rho_x f_y/\gamma_{M1}}\right)^2+\left(\frac{\sigma_{z,Ed}}{\rho_z f_y/\gamma_{M1}}\right)^2-\left(\frac{\sigma_{x,Ed}}{\rho_x f_y/\gamma_{M1}}\right)\left(\frac{\sigma_{z,Ed}}{\rho_z f_y/\gamma_{M1}}\right)+3\left(\frac{\tau_{Ed}}{\chi_w f_y/\gamma_{M1}}\right)^2\leq 1 \qquad (10.5)$$

$$\cancel{\left(\frac{\sigma_{x,Ed}}{\rho_x f_y/\gamma_{M1}}\right)^2+\left(\frac{\sigma_{z,Ed}}{\rho_z f_y/\gamma_{M1}}\right)^2-\left(\frac{\sigma_{x,Ed}}{\rho_x f_y/\gamma_{M1}}\right)\left(\frac{\sigma_{z,Ed}}{\rho_z f_y/\gamma_{M1}}\right)+3\left(\frac{\tau_{Ed}}{\chi_v f_y/\gamma_{M1}}\right)^2\leq 1}$$

NOTE 1 Since verification formulae (10.3), (10.4) and (10.5) include an interaction between shear force, bending moment, axial force and transverse force, section 7 should not be applied.

NOTE 2 The National Annex may give further information on the use of equations (10.4) and (10.5). In case of panels with tension and compression it is recommended to apply equations (10.4) and (10.5) only for the compressive parts.

(6) Where α_{cr} values for the complete stress field are not available and only $\alpha_{cr,i}$ values for the various components of the stress field $\sigma_{x,Ed}$, $\sigma_{z,Ed}$ and τ_{Ed} can be used, the α_{cr} value may be determined from:

$$\frac{1}{\alpha_{cr}}=\frac{1+\psi_x}{4\,\alpha_{cr,x}}+\frac{1+\psi_z}{4\,\alpha_{cr,z}}+\left[\left(\frac{1+\psi_x}{4\,\alpha_{cr,x}}+\frac{1+\psi_z}{4\,\alpha_{cr,z}}\right)^2+\frac{1-\psi_x}{2\,\alpha_{cr,x}^2}+\frac{1-\psi_z}{2\,\alpha_{cr,z}^2}+\frac{1}{\alpha_{cr,\tau}^2}\right]^{1/2} \qquad (10.6)$$

where $\alpha_{cr,x}=\dfrac{\sigma_{cr,x}}{\sigma_{x,Ed}}$

$\alpha_{cr,z}=\dfrac{\sigma_{cr,z}}{\sigma_{z,Ed}}$

$\alpha_{cr,\tau}=\dfrac{\tau_{cr,\tau}}{\tau_{\tau,Ed}}$

and $\sigma_{cr,x}$, $\sigma_{cr,z}$, τ_{cr}, ψ_x and ψ_z are determined from sections 4 to 6.

(7) Stiffeners and detailing of plate panels should be designed according to section 9.

Annex A [informative] – Calculation of critical stresses for stiffened plates

A.1 Equivalent orthotropic plate

(1) Plates with more than two longitudinal stiffeners may be treated as equivalent orthotropic plates.

(2) The elastic critical plate buckling stress of the equivalent orthotropic plate may be taken as:

$$\sigma_{cr,p} = k_{\sigma,p}\ \sigma_E \tag{A.1}$$

where $\sigma_E = \dfrac{\pi^2\ E\ t^2}{12\left(1-\nu^2\right)b^2} = 190000\left(\dfrac{t}{b}\right)^2$ in $[MPa]$

$k_{\sigma,p}$ is the buckling coefficient according to orthotropic plate theory with the stiffeners smeared over the plate;

b is defined in Figure A.1;

t is the thickness of the plate.

NOTE 1 The buckling coefficient $k_{\sigma,p}$ is obtained either from appropriate charts for smeared stiffeners or relevant computer simulations; alternatively charts for discretely located stiffeners may be used provided local buckling in the subpanels can be ignored and treated separately.

NOTE 2 $\sigma_{cr,p}$ is the elastic critical plate buckling stress at the edge of the panel where the maximum compression stress occurs, see Figure A.1.

NOTE 3 Where a web is of concern, the width b in equation (A.1) ~~may~~ should be replaced by h_w.

NOTE 4 For stiffened plates with at least three equally spaced longitudinal stiffeners the plate buckling coefficient $k_{\sigma,p}$ (global buckling of the stiffened panel) may be approximated by

$$\begin{aligned} k_{\sigma,p} &= \frac{2\left(\left(1+\alpha^2\right)^2+\gamma-1\right)}{\alpha^2(\psi+1)(1+\delta)} \quad \text{if} \quad \alpha \le \sqrt[4]{\gamma} \\ k_{\sigma,p} &= \frac{4\left(1+\sqrt{\gamma}\right)}{(\psi+1)(1+\delta)} \quad \text{if} \quad \alpha > \sqrt[4]{\gamma} \end{aligned} \tag{A.2}$$

with: $\psi = \dfrac{\sigma_2}{\sigma_1} \ge 0{,}5$

$\gamma = \dfrac{I_{sl}}{I_p}$ $\cancel{\gamma = \dfrac{\sum I_{sl}}{I_p}}$

$\delta = \dfrac{A_{sl}}{A_p}$ $\cancel{\delta = \dfrac{\sum A_{sl}}{A_p}}$

$\alpha = \dfrac{a}{b} \ge 0{,}5$

where: I_{sl} $\cancel{\sum I_{sl}}$ is ~~the sum of~~ the second moment of area of the whole stiffened plate;

I_p is the second moment of area for bending of the plate $= \dfrac{bt^3}{12\left(1-\upsilon^2\right)} = \dfrac{bt^3}{10{,}92}$;

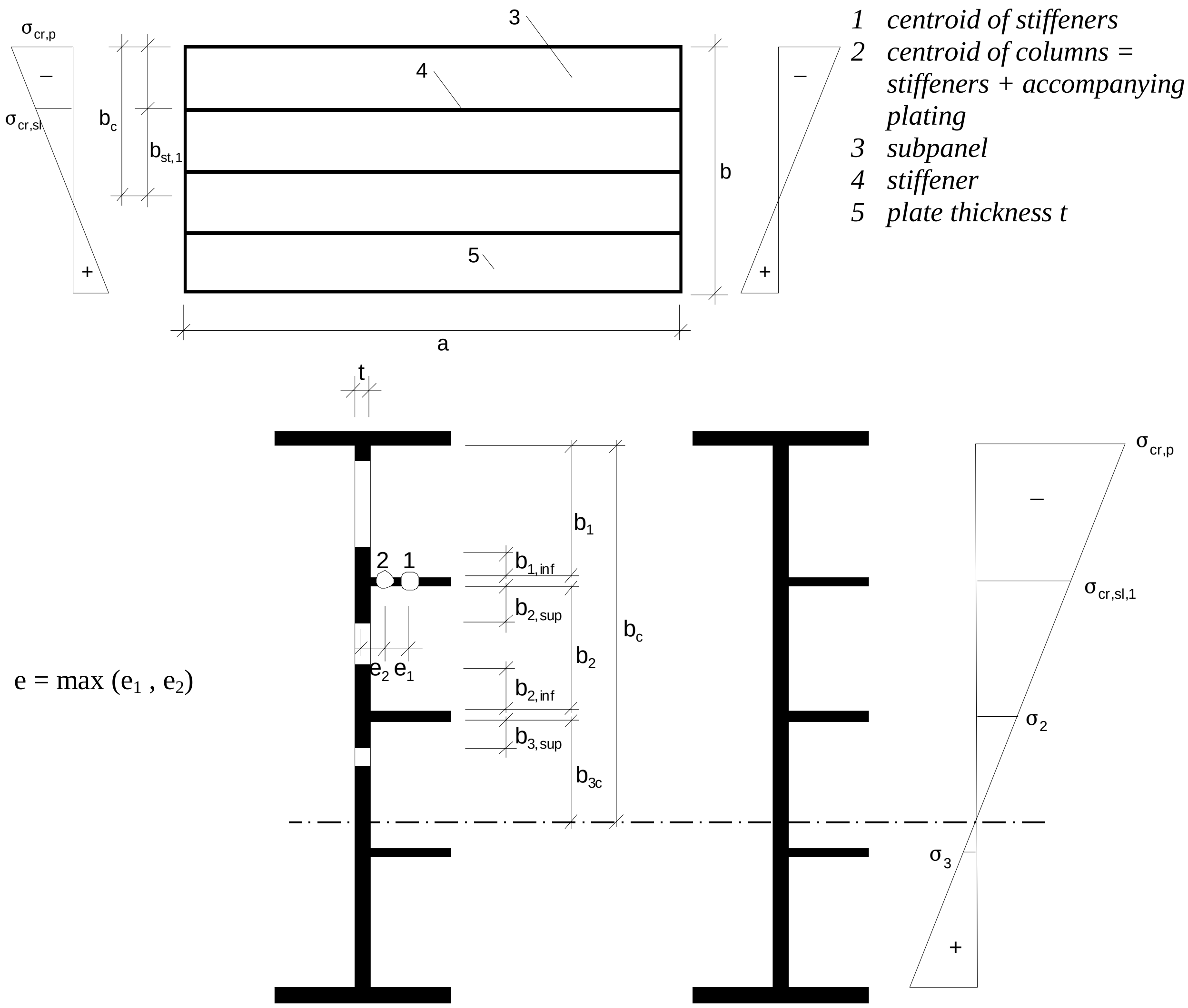
A_{sl} $\cancel{\sum A_{sl}}$ is ~~the sum of~~ the gross area of the individual longitudinal stiffeners;

A_p is the gross area of the plate $= bt$;

σ_1 is the larger edge stress;

σ_2 is the smaller edge stress;

a, b and t are as defined in Figure A.1.



	width for gross area	width for effective area according to Table 4.1	condition for ψ_i
$b_{1,inf}$	$\frac{3-\psi_1}{5-\psi_1} b_1$	$\frac{3-\psi_1}{5-\psi_1} b_{1,eff}$	$\psi_1 = \frac{\sigma_{cr,sl,1}}{\sigma_{cr,p}} > 0$
$b_{2,sup}$	$\frac{2}{5-\psi_2} b_2$	$\frac{2}{5-\psi_2} b_{2,eff}$	$\psi_2 = \frac{\sigma_2}{\sigma_{cr,sl,1}} > 0$ $\cancel{\psi_2 = \frac{\sigma_2}{\sigma_{cr,st,1}} > 0}$
$b_{2,inf}$	$\frac{3-\psi_2}{5-\psi_2} b_2$	$\frac{3-\psi_2}{5-\psi_2} b_{2,eff}$	$\psi_2 > 0$
$b_{3,sup}$	$0{,}4\ b_{c3}$	$0{,}4\ b_{c3,eff}$	$\psi_3 = \frac{\sigma_3}{\sigma_2} < 0$

Figure A.1: Notations for longitudinally stiffened plates

A.2 Critical plate buckling stress for plates with one or two stiffeners in the compression zone

A.2.1 General procedure

(1) If the stiffened plate has only one longitudinal stiffener in the compression zone the procedure in A.1 may be simplified by a fictitious isolated strut supported on an elastic foundation reflecting the plate effect in the direction perpendicular to this strut. The elastic critical stress of the strut may be obtained from A.2.2.

(2) For calculation of $A_{s\ell,1}$ and $I_{s\ell,1}$ the gross cross-section of the column should be taken as the gross area of the stiffener and adjacent parts of the plate described as follows. If the subpanel is fully in compression, a portion $(3-\psi)/(5-\psi)$ of its width b_1 should be taken at the edge of the panel and $2/(5-\psi)$ at the edge with the highest stress. If the stress changes from compression to tension within the subpanel, a portion 0,4 of the width b_c of the compressed part of this subpanel should be taken as part of the column, see Figure A.2 and also Table 4.1. ψ is the stress ratio relative to the subpanel in consideration.

(3) The effective^{p} cross-sectional area $A_{s\ell,eff}$ of the column should be taken as the effective^{p} cross-section of the stiffener and the adjacent effective^{p} parts of the plate, see Figure A.1. The slenderness of the plate elements in the column may be determined according to 4.4(4), with $\sigma_{com,Ed}$ calculated for the gross cross-section of the plate.

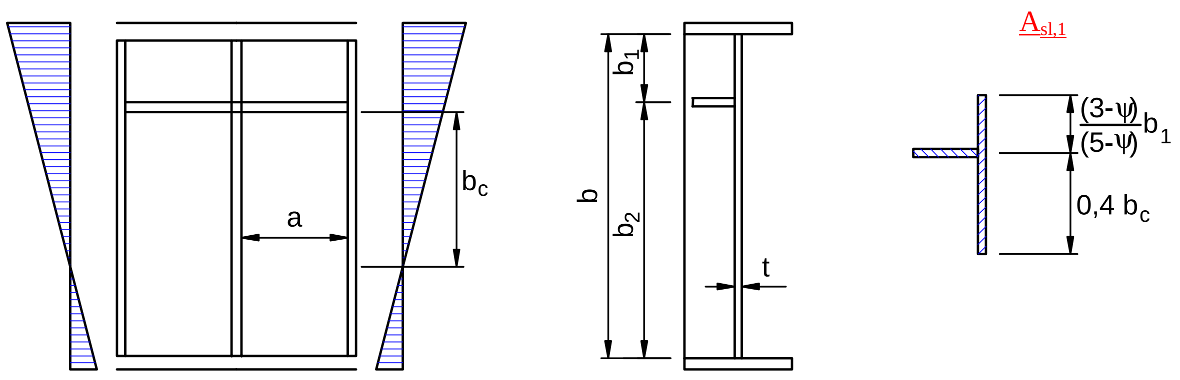
(4) If $\rho_c f_{yd}$,with ρ_c determined according to 4.5.4(1), is greater than the average stress in the column $\sigma_{com,Ed}$ no further reduction of the effective^{p} area of the column should be made. Otherwise the effective area in (4.6) should be modified as follows:

$$\underline{A_{c,eff,loc} = \frac{\rho_c\, f_y\, A_{sl,1}}{\sigma_{com,Ed}\, \gamma_{M1}}} \qquad \cancel{A_{c,eff} = \frac{\rho_c\, f_y\, A_{st}}{\sigma_{com,Ed}\, \gamma_{M1}}} \tag{A.3}$$

(5) The reduction mentioned in A.2.1(4) should be applied only to the area of the column. No reduction need be applied to other compressed parts of the plate, except for checking buckling of subpanels.

(6) As an alternative to using an effective^{p} area according to A.2.1(4), the resistance of the column may be determined from A.2.1(5) to (7) and checked to ensure that it exceeds the average stress $\sigma_{com,Ed}$.

> **NOTE** The method outlined in (6) may be used in the case of multiple stiffeners in which the restraining effect from the plate is neglected, that is the fictitious column is considered free to buckle out of the plane of the web.



a. b. c.

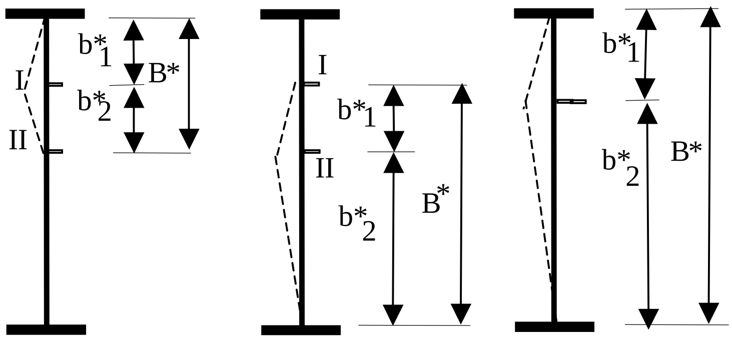
Figure A.2: Notations for a web plate with single stiffener in the compression zone

(7) If the stiffened plate has two longitudinal stiffeners in the compression zone, the one stiffener procedure described in A.2.1(1) may be applied, see Figure A.3. First, it is assumed that one of the stiffeners buckles while the other one acts as a rigid support. Buckling of both the stiffeners simultaneously is accounted for by considering a single lumped stiffener that is substituted for both individual ones such that:

a) its cross-sectional area and its second moment of area I_{st} are respectively the sum of that for the individual stiffeners

b) it is positioned at the location of the resultant of the respective forces in the individual stiffeners

For each of these situations illustrated in Figure A.3 a relevant value of $\sigma_{cr.p}$ is computed, see A.2.2(1), with $b_1 = b_1^*$ and $b_2 = b_2^*$ and $B^* = b_1^* + b_2^*$, see Figure A.3.



	Stiffener I	Stiffener II	Lumped stiffener
Cross-sectional area	$A_{s\ell,I}$ $\cancel{A_{s\ell,1}}$	$A_{s\ell,II}$ $\cancel{A_{s\ell,2}}$	$A_{s\ell,I} + A_{s\ell,II}$ $\cancel{A_{s\ell,1} + A_{s\ell,2}}$
Second moment of area	$I_{s\ell,I}$ $\cancel{I_{s\ell,1}}$	$I_{s\ell,II}$ $\cancel{I_{s\ell,2}}$	$I_{s\ell,I} + I_{s\ell,II}$ $\cancel{I_{s\ell,1} + I_{s\ell,2}}$

Figure A.3: Notations for plate with two stiffeners in the compression zone

A.2.2 Simplified model using a column restrained by the plate

(1) In the case of a stiffened plate with one longitudinal stiffener located in the compression zone, the elastic critical buckling stress of the stiffener can be calculated as follows ignoring stiffeners in the tension zone:

$$\sigma_{cr,s\ell} = \frac{1{,}05\,E}{A_{s\ell,1}} \frac{\sqrt{I_{s\ell,1}\,t^3\,b}}{b_1\,b_2} \qquad \text{if } a \geq a_c$$

$$\sigma_{cr,s\ell} = \frac{\pi^2\,E\,I_{s\ell,1}}{A_{s\ell,1}\,a^2} + \frac{E\,t^3\,b\,a^2}{4\,\pi^2\,(1-\nu^2)\,A_{s\ell,1}\,b_1^2\,b_2^2} \qquad \text{if } a \leq a_c \tag{A.4}$$

with $a_c = 4{,}33\sqrt[4]{\frac{I_{s\ell,1}\,b_1^2\,b_2^2}{t^3\,b}}$

where $A_{s\ell,1}$ is the gross area of the column obtained from A.2.1(2)

$I_{s\ell,1}$ is the second moment of area of the gross cross-section of the column defined in A.2.1(2) about an axis through its centroid and parallel to the plane of the plate;

b_1, b_2 are the distances from the longitudinal edges of the web to the stiffener ($b_1+b_2 = b$).

NOTE For determining $\sigma_{cr,c}$ see NOTE 2 to 4.5.3(3).

(2) In the case of a stiffened plate with two longitudinal stiffeners located in the compression zone the elastic critical plate buckling stress should be taken as the lowest of those computed for the three cases using equation (A.4) with $b_1 = b_1^*$, $b_2 = b_2^*$ and $b = B^*$. The stiffeners in the tension zone should be ignored in the calculation.

A.3 Shear buckling coefficients

(1) For plates with rigid transverse stiffeners and without longitudinal stiffeners or with more than two longitudinal stiffeners, the shear buckling coefficient k_τ can be obtained as follows:

$$\begin{aligned} k_\tau &= 5{,}34 + 4{,}00\,(h_w/a)^2 + k_{\tau st} \quad \text{when } a/h_w \geq 1 \\ k_\tau &= 4{,}00 + 5{,}34\,(h_w/a)^2 + k_{\tau st} \quad \text{when } a/h_w < 1 \end{aligned} \tag{A.5}$$

where $k_{\tau st} = 9\left(\frac{h_w}{a}\right)^2 \sqrt[4]{\left(\frac{I_{s\ell}}{t^3\,h_w}\right)^3}$ but not less than $\frac{2{,}1}{t}\sqrt[3]{\frac{I_{s\ell}}{h_w}}$

a is the distance between transverse stiffeners (see Figure 5.3);

$I_{s\ell}$ is the second moment of area of the longitudinal stiffener about the z-axis, see Figure 5.3 (b). For webs with two or more longitudinal stiffeners, not necessarily equally spaced, $I_{s\ell}$ is the sum of the stiffness of the individual stiffeners.

NOTE No intermediate non-rigid transverse stiffeners are allowed for in equation (A.5).

(2) The equation (A.5) also applies to plates with one or two longitudinal stiffeners, if the aspect ratio $\alpha = \frac{a}{h_w}$ satisfies $\alpha \geq 3$. For plates with one or two longitudinal stiffeners and an aspect ratio $\alpha < 3$ the shear buckling coefficient should be taken from:

$$k_\tau = 4{,}1 + \frac{6{,}3 + 0{,}18\,\frac{I_{s\ell}}{t^3 h_w}}{\alpha^2} + 2{,}2\,\sqrt[3]{\frac{I_{s\ell}}{t^3 h_w}} \tag{A.6}$$

Annex B [informative] – Non-uniform members

B.1 General

(1) The rules in section 10 are applicable to webs of members with non parallel flanges as in haunched beams and to webs with regular or irregular openings and non orthogonal stiffeners.

(2) α_{ult} and α_{crit} may be obtained from FE-methods, see Annex C.

(3) The reduction factors ρ_x , ρ_z and χ_w for $\bar{\lambda}_p$ may be obtained from the appropriate plate buckling curves, see sections 4 and 5.

NOTE The reduction factor ρ may be obtained as follows:

$$\rho = \frac{1}{\varphi_p + \sqrt{\varphi_p^2 - \bar{\lambda}_p}} \tag{B.1}$$

where $\varphi_p = \frac{1}{2}\left(1 + \alpha_p\left(\bar{\lambda}_p - \bar{\lambda}_{p0}\right) + \bar{\lambda}_p\right)$

and $\bar{\lambda}_p = \sqrt{\frac{\alpha_{ult,k}}{\alpha_{cr}}}$

This procedure applies to ρ_x, ρ_z and χ_w. The values of $\bar{\lambda}_{p0}$ and α_p are given in Table B.1. These values have been calibrated against the plate buckling curves in sections 4 and 5 and give a direct correlation to the equivalent geometric imperfection, by :

$$e_0 = \alpha_p\left(\bar{\lambda}_p - \bar{\lambda}_{p0}\right)\frac{t}{6}\,\frac{1 - \frac{\rho\bar{\lambda}_p}{\gamma_{M1}}}{1 - \rho\bar{\lambda}_p} \tag{B.2}$$

Table B.1: Values for $\bar{\lambda}_{p0}$ and α_p

Product	predominant buckling mode	α_p	$\bar{\lambda}_{p0}$
hot rolled	direct stress for $\psi \geq 0$	0,13	0,70
	direct stress for $\psi < 0$ shear transverse stress		0,80
welded ~~and~~ or cold formed	direct stress for $\psi \geq 0$	0,34	0,70
	direct stress for $\psi < 0$ shear transverse stress		0,80

B.2 Interaction of plate buckling and lateral torsional buckling

(1) The method given in B.1 may be extended to the verification of combined plate buckling and lateral torsional buckling of ~~beams~~ members by calculating α_{ult} and α_{crit} as follows:

α_{ult} is the minimum load amplifier for the design loads to reach the characteristic value of resistance of the most critical cross section, neglecting any plate buckling ~~or~~ and lateral torsional buckling;

α_{cr} is the minimum load amplifier for the design loads to reach the elastic critical resistance of the ~~beam~~ member including plate buckling and lateral torsional buckling modes.

(2) In case α_{cr} contains lateral torsional buckling modes, the reduction factor ρ used should be the minimum of the reduction factor according to B.1(4) and the χ_{LT} – value for lateral torsional buckling according to 6.3.3 of EN 1993-1-1.

Annex C [informative] – Finite Element Methods of analysis (FEM)

C.1 General

(1) Annex C gives guidance on the use of FE-methods for ultimate limit state, serviceability limit state or fatigue verifications of plated structures.

NOTE 1 For FE-calculation of shell structures see EN 1993-1-6.

NOTE 2 This guidance is intended for engineers who are experienced in the use of Finite Element methods.

(2) The choice of the FE-method depends on the problem to be analysed and based on the following assumptions:

Table C.1: Assumptions for FE-methods

No	Material behaviour	Geometric behaviour	Imperfections, see section C.5	Example of use
1	linear	linear	no	elastic shear lag effect, elastic resistance
2	non linear	linear	no	plastic resistance in ULS
3	linear	non linear	no	critical plate buckling load
4	linear	non linear	yes	elastic plate buckling resistance
5	non linear	non linear	yes	elastic-plastic resistance in ULS

C.2 Use

(1) In using FEM for design special care should be taken to

- the modelling of the structural component and its boundary conditions;
- the choice of software and documentation;
- the use of imperfections;
- the modelling of material properties;
- the modelling of loads;
- the modelling of limit state criteria;
- the partial factors to be applied.

NOTE The National Annex may define the conditions for the use of FEM analysis in design.

C.3 Modelling

(1) The choice of FE-models (shell models or volume models) and the size of mesh determine the accuracy of results. For validation sensivity checks with successive refinement may be carried out.

(2) The FE-modelling may be carried out either for:

- the component as a whole or
- a substructure as a part of the whole ~~component,~~ structure.

NOTE An example for a component could be the web and/or the bottom plate of continuous box girders in the region of an intermediate support where the bottom plate is in compression. An example for a substructure could be a subpanel of a bottom plate subject to biaxial stresses.

(3) The boundary conditions for supports, interfaces and applied loads should be chosen such that results obtained are conservative.

(4) Geometric properties should be taken as nominal.

(5) All imperfections should be based on the shapes and amplitudes as given in section C.5.

(6) Material properties should conform to C.6(2).

C.4 Choice of software and documentation

(1) The software should be suitable for the task and be proven reliable.

NOTE Reliability can be proven by appropriate bench mark tests.

(2) The mesh size, loading, boundary conditions and other input data as well as the output should be documented in a way that they can be reproduced by third parties.

C.5 Use of imperfections

(1) Where imperfections need to be included in the FE-model these imperfections should include both geometric and structural imperfections.

(2) Unless a more refined analysis of the geometric imperfections and the structural imperfections is carried out, equivalent geometric imperfections may be used.

NOTE 1 Geometric imperfections may be based on the shape of the critical plate buckling modes with amplitudes given in the National Annex. 80 % of the geometric fabrication tolerances is recommended.

NOTE 2 Structural imperfections in terms of residual stresses may be represented by a stress pattern from the fabrication process with amplitudes equivalent to the mean (expected) values.

(3) The direction of the applied imperfection should be such that the lowest resistance is obtained.

(4) For applying equivalent geometric imperfections Table C.2 and Figure C.1 may be used.

Table C.2: Equivalent geometric imperfections

~~t~~Type of imperfection	~~c~~Component	~~s~~Shape	~~m~~Magnitude
global	member with length ℓ	bow	see EN 1993-1-1, Table 5.1
global	longitudinal stiffener with length a	bow	min (a/400, b/400)
local	panel or subpanel with short span a or b	buckling shape	min (a/200, b/200)
local	stiffener or flange subject to twist	bow twist	1 / 50

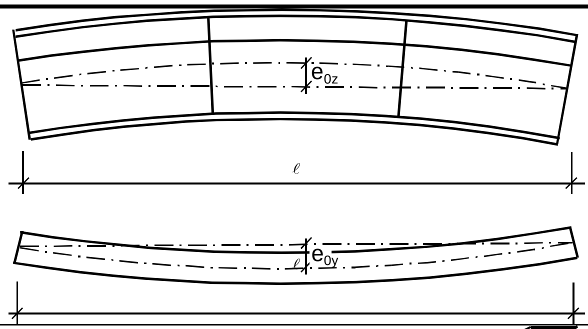
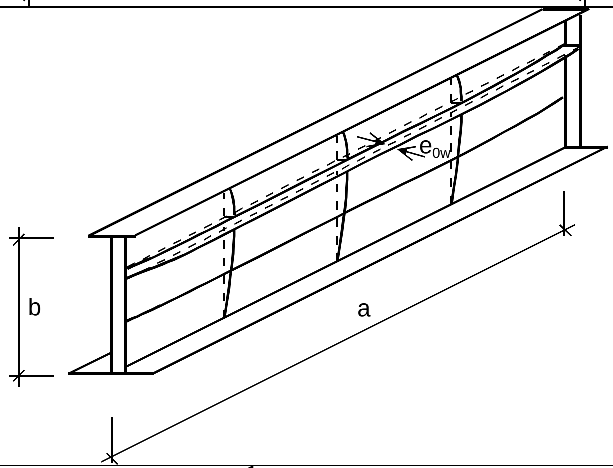
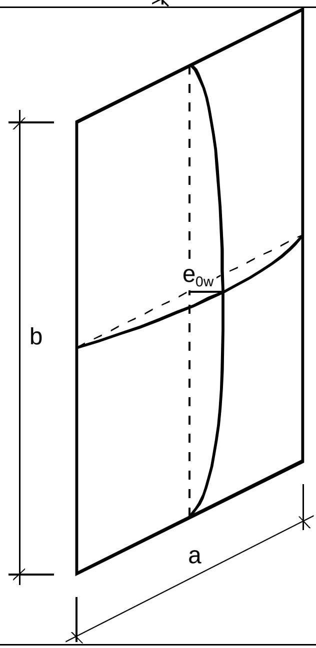
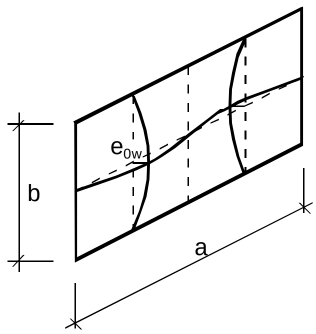
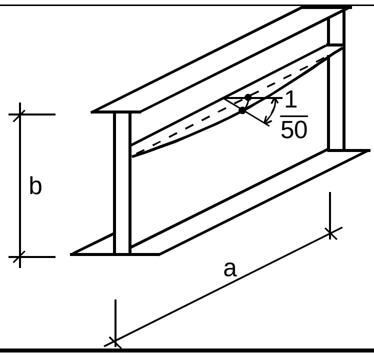
Type of imperfection	Component
global member with length ℓ	
global longitudinal stiffener with length a	
local panel or subpanel	 
local stiffener or flange subject to twist	

Figure C.1: Modelling of equivalent geometric imperfections

(5) In combining imperfections a leading imperfection should be chosen and the accompanying imperfections may have their values reduced to 70%.

NOTE 1 Any type of imperfection should be taken as the leading imperfection and the others may be taken as the accompanying imperfections.

NOTE 2 Equivalent geometric imperfections may be substituted by the appropriate ~~disturbing~~ fictitious forces acting on the member.

C.6 Material properties

(1) Material properties should be taken as characteristic values.

(2) Depending on the accuracy and the allowable strain required for the analysis the following assumptions for the material behaviour may be used, see Figure C.2:

a) elastic-plastic without strain hardening;

b) elastic-plastic with a nominal plateau slope;

c) elastic-plastic with linear strain hardening;

d) true stress-strain curve modified from the test results as follows:

$$\sigma_{true} = \sigma(1+\varepsilon)$$
$$\varepsilon_{true} = \ell n(1+\varepsilon) \qquad \text{(C.1)}$$

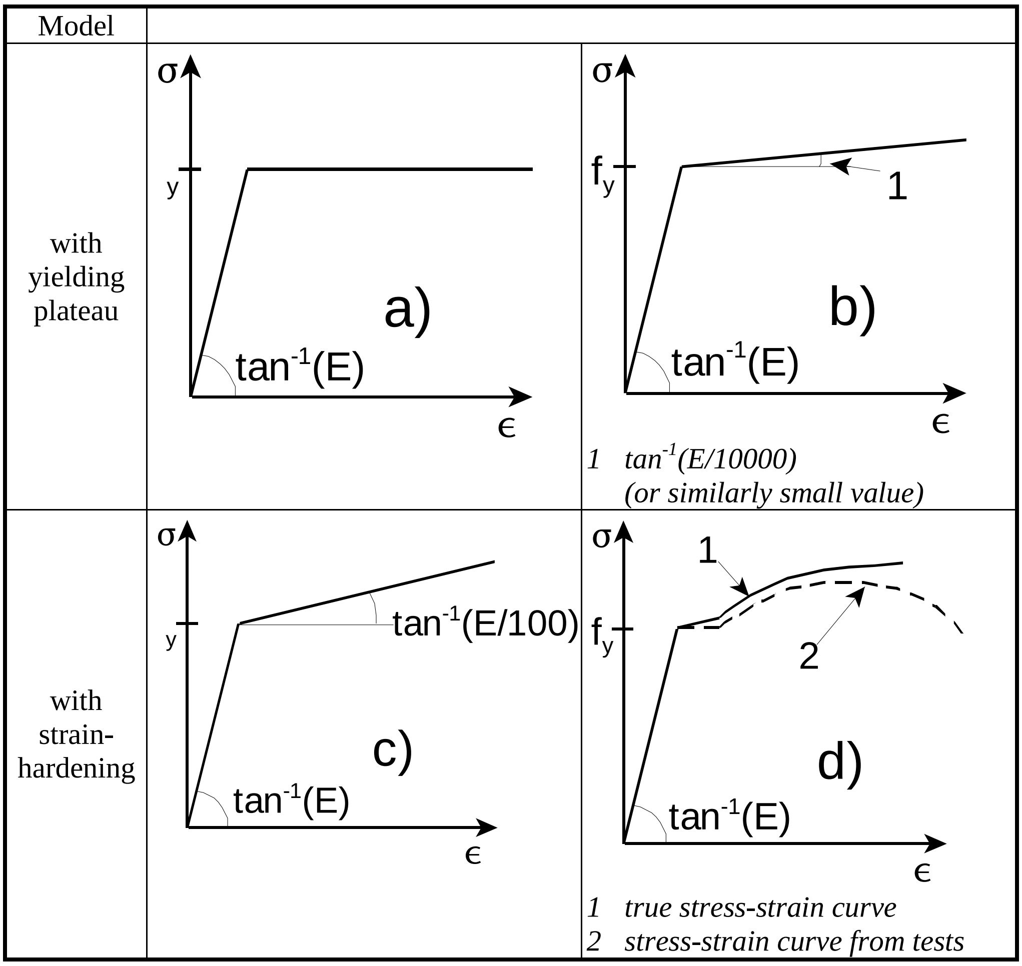


Figure C.2: Modelling of material behaviour

NOTE For the elastic modulus E the nominal value is relevant.

C.7 Loads

(1) The loads applied to the structures should include relevant load factors and load combination factors. For simplicity a single load multiplier α may be used.

C.8 Limit state criteria

(1) The ultimate limit state criteria should be used as follows:

1. for structures susceptible to buckling:

   attainment of the maximum load.

2. for regions subjected to tensile stresses:

   attainment of a limiting value of the principal membrane strain.

NOTE 1 The National Annex may specify the limiting of principal strain. A value of 5% is recommended.

NOTE 2 Other criteria may be used, e.g. attainment of the yielding criterion or limitation of the yielding zone.

C.9 Partial factors

(1) The load magnification factor α_u to the ultimate limit state should be sufficient to achieve the required reliability.

(2) The magnification factor α_u should consist of two factors as follows:

1. α_1 to cover the model uncertainty of the FE-modelling used. It should be obtained from evaluations of test calibrations, see Annex D to EN 1990;
2. α_2 to cover the scatter of the loading and resistance models. It may be taken as γ_{M1} if instability governs and γ_{M2} if fracture governs.

(3) It should be verified that:

$$\alpha_u > \alpha_1 \, \alpha_2 \tag{C.2}$$

NOTE The National Annex may give information on γ_{M1} and γ_{M2}. The use of γ_{M1} and γ_{M2} as specified in EN 1993-1-1 is recommended.

Annex D [informative] – Plate girders with corrugated webs

D.1 General

(1) Annex D covers design rules for I-girders with trapezoidally or sinusoidally corrugated webs, see Figure D.1.

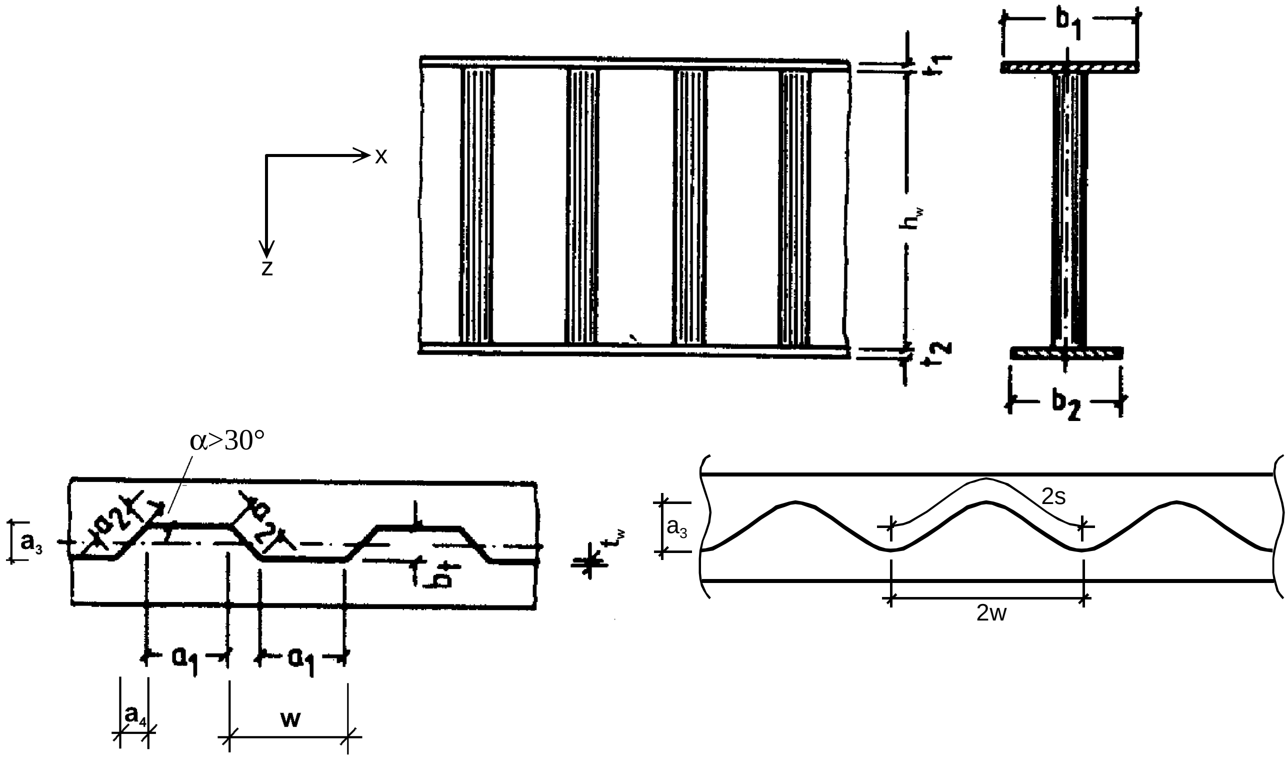


Figure D.1: Geometric notations

D.2 Ultimate limit state

D.2.1 Moment of resistance

(1) The moment of resistance M_{Rd} due to bending should be taken as the minimum of the following:

$$M_{Rd} = \min\left\{ \underbrace{\frac{b_2 t_2 f_{yw,r}}{\gamma_{M0}}\left(h_w + \frac{t_1 + t_2}{2}\right)}_{\text{tension flange}} ; \underbrace{\frac{b_1 t_1 f_{yw,r}}{\gamma_{M0}}\left(h_w + \frac{t_1 + t_2}{2}\right)}_{\text{compression flange}} ; \underbrace{\frac{b_1 t_1 \chi f_{yw}}{\gamma_{M1}}\left(h_w + \frac{t_1 + t_2}{2}\right)}_{\text{compression flange}} \right\} \quad \text{(D.1)}$$

where $f_{yw,,r}$ is the value of yield stress reduced due to transverse moments in the flanges

$f_{y,w,r} = f_{yw} f_T$

$$f_T = 1 - 0{,}4\sqrt{\frac{\sigma_x(M_z)}{\dfrac{f_{yf}}{\gamma_{M0}}}}$$

$\sigma_x(M_z)$ is the stress due to the transverse moment in the flange

χ is the reduction factor for ~~lateral~~ out of plane buckling according to 6.3 of EN 1993-1-1

NOTE 1 The transverse moment M_z results from the shear flow in flanges as indicated in Figure D.2.

NOTE 2 For sinusoidally corrugated webs f_T is 1,0.

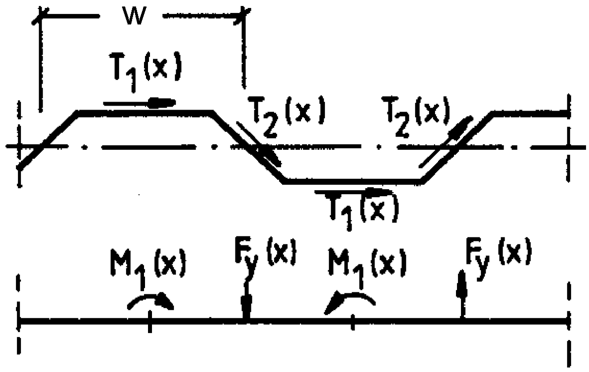


Figure D.2: Transverse moments M_z due to shear flow introduction into the flange

(2) The effective area of the compression flange should be determined from 4.4(1) and (2) using the larger value of the slenderness parameter $\bar{\lambda}_p$ defined in 4.4(2) and the buckling factor k_σ taken as:

a) $$k_\sigma = 0{,}43 + \left(\frac{b}{a}\right)^2 \tag{D.2}$$

where b is the maximum width of the outstand from the toe of the weld to the free edge

$$a = a_1 + 2a_4$$

b) $$k_\sigma = 0{,}60 \tag{D.3}$$

where $b = \dfrac{b_1}{2}$

D.2.2 Shear resistance

(1) The shear resistance V_{Rd} should be taken as:

$$V_{Rd} = \chi_c \frac{f_{yw}}{\gamma_{M1}\sqrt{3}} h_w t_w \tag{D.4}$$

where χ_c is the lesser of the values of reduction factors for local buckling $\chi_{c,\ell}$ and global buckling $\chi_{c,g}$ obtained from (2) and (3)

(2) The reduction factor $\chi_{c,\ell}$ for local buckling should be calculated from:

$$\chi_{c,\ell} = \frac{1{,}15}{0{,}9 + \bar{\lambda}_{c,\ell}} \leq 1{,}0 \tag{D.5}$$

where $$\bar{\lambda}_{c,\ell} = \sqrt{\frac{f_y}{\tau_{cr,\ell}\sqrt{3}}} \tag{D.6}$$

$$\tau_{cr,\ell} = 4{,}83\, E \left[\frac{t_w}{a_{max}}\right]^2 \tag{D.7}$$

a_{max} should be taken as the greater of a_1 and a_2.

For sinusoidally corrugated webs $\tau_{cr,\ell}$ may be obtained from

$$\tau_{cr,l} = \left(5{,}34 + \frac{a_3\, s}{h_w t_w}\right)\frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t_w}{s}\right)^2 \qquad \text{(D.8)}$$

where w length of one half wave, see Figure D.1,

s unfolded length of one half wave, see Figure D.1.

(3) The reduction factor $\chi_{c,g}$ for global buckling should be taken as

$$\chi_{c,g} = \frac{1{,}5}{0{,}5 + \bar{\lambda}_{c,g}^2} \le 1{,}0 \qquad \text{(D.9)}$$

where $\bar{\lambda}_{c,g} = \sqrt{\frac{f_y}{\tau_{cr,g}\sqrt{3}}}$ (D.10)

$$\tau_{cr,g} = \frac{32{,}4}{t_w\, h_w^2}\sqrt[4]{D_x\, D_z^3} \qquad \text{(D.11)}$$

$$D_x = \frac{E\, t^3}{12(1-\nu^2)}\,\frac{w}{s}$$

$$D_z = \frac{E\, I_z}{w}$$

I_z second moment of area of one corrugation of length w, see Figure D.1

NOTE 1 s and I_z are related to the actual shape of the corrugation.

NOTE 2 Equation (D.11) is valid for plates that are assumed to be hinged at the edges.

D.2.3 Requirements for end stiffeners

(1) Bearing stiffeners should be designed according to section 9.

Annex E [normative] – Refined methods for determining effective cross sections

E.1 Effective areas for stress levels below the yield strength

(1) Alternatively to the method given in 4.4(2) the following formulae may be applied to determine effective areas at stress levels lower than the yield strength:

a) for doubly supported compression elements:

$$\rho = \frac{1-0{,}055(3+\psi)/\bar{\lambda}_{p,red}}{\bar{\lambda}_{p,red}} + 0{,}18\frac{(\bar{\lambda}_p - \bar{\lambda}_{p,red})}{(\bar{\lambda}_p - 0{,}6)} \quad \text{but } \rho \leq 1{,}0 \tag{E.1}$$

b) for outstand compression elements:

$$\rho = \frac{1-0{,}188/\bar{\lambda}_{p,red}}{\bar{\lambda}_{p,red}} + 0{,}18\frac{(\bar{\lambda}_p - \bar{\lambda}_{p,red})}{(\bar{\lambda}_p - 0{,}6)} \quad \text{but } \rho \leq 1{,}0 \tag{E.2}$$

For notations see 4.4(2) and 4.4(4). For calculation of resistance to global buckling 4.4(5) applies.

E.2 Effective areas for stiffness

(1) For the calculation of effective areas for stiffness the serviceability limit state slenderness $\bar{\lambda}_{p,ser}$ may be calculated from:

$$\bar{\lambda}_{p,ser} = \bar{\lambda}_p \sqrt{\frac{\sigma_{com,Ed,ser}}{f_y}} \tag{E.3}$$

where $\sigma_{com,Ed,ser}$ is defined as the largest compressive stress (calculated on the basis of the effective cross section) in the relevant element under service loads.

(2) The second moment of area may be calculated by an interpolation of the gross cross section and the effective cross section for the relevant load combination using the expression:

$$I_{eff} = I_{gr} - \frac{\sigma_{gr}}{\sigma_{com,Ed,ser}}\left(I_{gr} - I_{eff}\left(\sigma_{con,Ed,ser}\right)\right) \tag{E.4}$$

where I_{gr} is the second moment of area of the gross cross section

σ_{gr} is the maximum bending stress at serviceability limit states based on the gross cross section

$I_{eff}(\sigma_{com,Ed,ser})$ is the second moment of area of the effective cross section with allowance for local buckling according to E.1 calculated for the maximum stress $\sigma_{com,Ed,ser} \geq \sigma_{gr}$ within the calculation length considered.

(3) The effective second moment of area I_{eff} may be taken as variable along the span according to the most severe locations. Alternatively a uniform value may be used based on the maximum absolute sagging moment under serviceability loading.

(4) The calculations require iterations, but as a conservative approximation they may be carried out as a single calculation at a stress level equal to or higher than $\sigma_{com,Ed,ser}$.

CEN/TC250/SC3/**N1360 E**

EUROPEAN STANDARD
NORME EUROPÉENNE
EUROPÄISCHE NORM

prEN 1993-1-6 : 2004

Oct 2004

UDC

Descriptors:

English version

Eurocode 3: Design of steel structures

Part 1-6 : Strength and Stability of Shell Structures

Calcul des structures en acier

Partie 1.6 :

Resistance et Stabilité des Coques

Bemessung und Konstruktion von Stahlbauten

Teil 1.6 :

Aus Schalen

Stage 49 ? draft

5 October 2004

CEN

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

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

Ref. No. EN 1993-1.6 : 20xx. E

Contents Page

National annex for EN 1993-1-6

This standard gives alternative procedures, values and recommendations with notes indicating where national choices may have to be made. Therefore the National Standard implementing EN 1993-1-6 should have a National Annex containing all Nationally Determined Parameters to be used for the design of steel structures to be constructed in the relevant country.

National choice is allowed in EN 1993-1-6 through:

- 4.1.4 (3)
- 5.2.4 (1)
- 6.3 (5)
- 7.3.1 (5)
- 7.3.2 (1)
- 8.4.2 Table 8.1
- 8.4.3 Tables 8.2 and 8.3
- 8.4.4 Table 8.4
- 8.4.5 (1)
- 8.5.2 (2)
- 8.7.2 Table 8.5
- 8.7.2 (7), (16) and (18)
- 9.2.1 (2)

1. Introduction

1.1 Scope

(1) EN 1993-1-6 gives design requirements for plated steel structures that have the form of a shell of revolution.

(2) This Standard is intended for use in conjunction with EN1993-1-1, EN1993-1-3, EN1993-1-4, EN1993-1-9 and the relevant application parts of EN1993, which include:

- Part 3.1 for towers and masts;
- Part 3.2 for chimneys;
- Part 4.1 for silos;
- Part 4.2 for tanks;
- Part 4.3 for pipelines.

(3) This Standard defines the characteristic and design values of the resistance of the structure.

(4) This Standard is concerned with the requirements for design against the ultimate limit states of:

- plastic limit;
- cyclic plasticity;
- buckling;
- fatigue.

(5) Overall equilibrium of the structure (sliding, uplifting, overturning) is not included in this Standard, but is treated in EN1993-1-1. Special considerations for specific applications are included in the relevant applications parts of EN1993.

(6) The provisions in this Standard apply to axisymmetric shells and associated circular or annular plates and to beam section rings and stringer stiffeners where they form part of the complete structure. The following shell forms are covered: cylinders, cones and spherical caps.

(7) Cylindrical, conical and spherical panels are not explicitly covered by this Standard. However, the provisions can be applicable if the appropriate boundary conditions are duly taken into account.

(8) This Standard is intended for application to structural engineering steel shell structures. However, its provisions can be applied to other metallic shells provided that the appropriate material properties are duly taken into account.

(9) The provisions of this Standard are intended to be applied within the temperature range defined in the relevant EN1993 application parts. The maximum temperature is restricted so that the influence of creep can be neglected if high temperature creep effects are not covered by the relevant application part.

(10) The provisions in this Standard apply to structures that satisfy the brittle fracture provisions given in EN1993-1-10.

(11) The provisions of this Standard apply to structural design under actions that can be treated as quasi-static in nature.

(12) In this Standard, it is assumed that both wind loading and bulk solids flow can, in general, be treated as quasi-static actions.

(13) Dynamic effects should be taken into account according to the relevant application part of EN 1993, including the consequences for fatigue. However, the stress resultants arising from dynamic behaviour are treated in this part as quasi-static.

(14) The provisions in this Standard apply to structures that are constructed in accordance with EN1090.

(15) This Standard does not cover the aspects of leakage of contents.

(16) This Standard is not intended for application to structures outside the following limits:

- design metal temperatures outside the range −50°C to +300°C;
- radius to thickness ratios outside the range 20 to 5000.

NOTE: It should be noted that the hand calculation rules of this standard may be rather conservative when applied to some geometries and loading conditions for relatively thick-walled shells.

1.2 Normative references

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

EN 1090 *Execution of steel structures:*

EN 1990 *Basis of design;*

EN 1991 *Eurocode 1: Actions on structures;*

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

Part 1.1: *General rules and rules for buildings;*

Part 1.3: *Cold formed members and sheeting;*

Part 1.4: *Stainless steels;*

Part 1.5: *Plated structural elements;*

Part 1.9: *Fatigue;*

Part 1.10: *Material toughness and through-thickness properties;*

Part 2: *Steel bridges;*

Part 3.1: *Towers and masts;*

Part 3.2: *Chimneys;*

Part 4.1: *Silos;*

Part 4.2: *Tanks;*

Part 4.3: *Pipelines.*

EN 13084 *Free standing chimneys:*

Part 7: *Product specification of cylindrical steel fabrications for use in single wall steel chimneys and steel liners.*

1.3 Definitions

The terms that are defined in EN 1990 for common use in the Structural Eurocodes apply to this Standard. Unless otherwise stated, the definitions given in ISO 8930 also apply in this Standard. Supplementary to EN 1993-1-1, for the purposes of this Standard, the following definitions apply:

1.3.1 Structural forms and geometry

1.3.1.1
shell
A structure or a structural component formed from a curved thin plate.

1.3.1.2
shell of revolution
A shell whose form is defined by a meridional generator line rotated around a single axis through 2π radians. The shell can be of any length.

1.3.1.3
complete axisymmetric shell
A shell composed of a number of parts, each of which is a shell of revolution.

1.3.1.4
shell segment
A shell of revolution in the form of a defined shell geometry with a constant wall thickness: a cylinder, conical frustum, spherical frustum, annular plate, toroidal knuckle or other form.

1.3.1.5
shell panel
An incomplete shell of revolution: the shell form is defined by a rotation of the generator about the axis through less than 2π radians.

1.3.1.6
middle surface
The surface that lies midway between the inside and outside surfaces of the shell at every point. Where the shell is stiffened on only one surface, the reference middle surface is still taken as the middle surface of the curved shell plate. The middle surface is the reference surface for analysis, and can be discontinuous at changes of thickness or shell junctions, leading to eccentricities that may be important to the shell structural behaviour.

1.3.1.7
junction
The point at which two or more shell segments meet: it can include a stiffener or not: the point of attachment of a ring stiffener to the shell may be treated as a junction.

1.3.1.8
stringer stiffener
A local stiffening member that follows the meridian of the shell, representing a generator of the shell of revolution. It is provided to increase the stability, or to assist with the introduction of local loads. It is not intended to provide a primary resistance to bending effects caused by transverse loads.

1.3.1.9
rib
A local member that provides a primary load carrying path for bending down the meridian of the shell, representing a generator of the shell of revolution. It is used to transfer or distribute transverse loads by bending.

1.3.1.10
ring stiffener
A local stiffening member that passes around the circumference of the shell of revolution at a given point on the meridian. It is assumed to have no stiffness in the meridional plane of the shell. It is provided to increase the stability or to introduce axisymmetric local loads acting in the plane of the ring by a state of axisymmetric normal forces. It is not intended to provide primary resistance for bending.

1.3.1.11
base ring
A structural member that passes around the circumference of the shell of revolution at the base and provides means of attachment of the shell to a foundation or other structural member. It is needed to ensure that the assumed boundary conditions are achieved in practice.

1.3.1.12
ring beam or ring girder
A circumferential stiffener that has bending stiffness and strength both in the plane of the shell circular section and normal to that plane. It is a primary load carrying structural member, provided for the distribution of local loads into the shell.

1.3.2 Limit states

1.3.2.1
plastic limit
The ultimate limit state where the structure develops zones of yielding in a pattern such that its ability to resist increased loading is deemed to be exhausted. It can be related to a small deflection theory limit load or plastic collapse mechanism.

1.3.2.2
tensile rupture
The ultimate limit state where the shell plate experiences gross section failure due to tension.

1.3.2.3
cyclic plasticity
The ultimate limit state where repeated yielding is caused by cycles of loading and unloading, leading to a low cycle fatigue failure where the energy absorption capacity of the material is exhausted.

1.3.2.4
buckling
The ultimate limit state where the structure suddenly loses its stability under membrane compression and/or shear. It leads either to large displacements or to the structure being unable to support the applied loads.

1.3.2.5
fatigue
The ultimate limit state where many cycles of loading cause cracks to develop of the shell plate.

1.3.3 Actions

1.3.3.1
axial load
Externally applied loading acting in the axial direction.

1.3.3.2
radial load
Externally applied loading acting normal to the surface of a cylindrical shell.

1.3.3.3
internal pressure
Component of the surface loading acting axisymmetrically, normal to the shell in the outward direction. Its magnitude can vary in both the meridional and circumferential directions (e.g. under solids loading in a silo).

1.3.3.4
external pressure
Component of the surface loading acting axisymmetrically, normal to the shell in the inward direction. It magnitude can vary in both the meridional and circumferential directions (e.g. under wind).

1.3.3.5
hydrostatic pressure
Pressure varying linearly with the axial coordinate of the shell of revolution.

1.3.3.6
wall friction load
Meridional component of the surface loading acting along the wall due to friction connected with internal pressure (when solids are contained within the shell).

1.3.3.7
local load
Point applied force or distributed load acting on a limited part of the circumference of the shell and over a limited height.

1.3.3.8
patch load
Local distributed load acting normal to the shell.

1.3.3.9
suction
Constant external pressure due to the sucking effect of the wind action on a shell with openings or vents.

1.3.3.10
partial vacuum
Constant external pressure due to the removal of stored liquids or solids from within a container with inadequate venting.

1.3.3.11
thermal action
Temperature variation either along or around the shell or through the shell thickness.

1.3.4 Types of analysis

1.3.4.1
global analysis
An analysis that includes the complete structure, rather than individual structural parts treated separately.

1.3.4.2
membrane theory analysis
An analysis that predicts the behaviour of a thin-walled shell structure under distributed loads by adopting a set of membrane forces that satisfy equilibrium with the external loads.

1.3.4.3
linear elastic shell analysis (LA)
An analysis that predicts the behaviour of a thin-walled shell structure on the basis of the small deflection linear elastic shell bending theory, related to the perfect geometry of the middle surface of the shell.

1.3.4.4
linear elastic bifurcation (eigenvalue) analysis (LBA)
An analysis that evaluates the linear bifurcation eigenvalue for a thin-walled shell structure on the basis of the small deflection linear elastic shell bending theory, related to the perfect geometry of the middle surface of the shell. It should be noted that, where an eigenvalue is mentioned, this does not relate to vibration modes.

1.3.4.5
geometrically nonlinear elastic analysis (GNA)
An analysis based on the principles of shell bending theory applied to the perfect structure, using a linear elastic material law but including nonlinear, large deflection theory for the displacements. A bifurcation eigenvalue check is included at each load level.

1.3.4.6
materially nonlinear analysis (MNA)
An analysis based on shell bending theory applied to the perfect structure, using the assumption of small deflections, as in 1.3.4.3, but adopting a nonlinear elasto-plastic material law.

1.3.4.7
geometrically and materially nonlinear analysis (GMNA)
An analysis based on shell bending theory applied to the perfect structure, using the assumptions of nonlinear, large deflection theory for the displacements and a nonlinear, elasto-plastic material law. A bifurcation eigenvalue check is included at each load level.

1.3.4.8
geometrically nonlinear elastic analysis with imperfections included (GNIA)
An analysis with imperfections included, similar to a GNA analysis as defined in 1.3.4.5, but adopting a model for the geometry of the structure that includes the imperfect shape (i.e. the geometry of the middle surface includes unintended deviations from the ideal shape). A bifurcation eigenvalue check is included at each load level.

1.3.4.9
geometrically and materially nonlinear analysis with imperfections included (GMNIA)
An analysis with imperfections included, similar to the GMNA analysis as defined in 1.3.4.7, but adopting a model for the geometry of the structure that includes the imperfect shape (i.e. the geometry of the middle surface includes unintended deviations from the ideal shape). A bifurcation eigenvalue check is included at each load level.

1.3.5 Special definitions for buckling calculations

1.3.5.1
critical buckling resistance
The smallest bifurcation or limit load determined assuming the idealised conditions of elastic material behaviour, perfect geometry, perfect load application, perfect support, material isotropy and absence of residual stresses (LBA analysis).

1.3.5.2
critical buckling stress
The nominal membrane stress (based on membrane theory) associated with the elastic critical buckling resistance.

1.3.5.3
characteristic buckling stress
The nominal membrane stress associated with buckling in the presence of inelastic material behaviour, the geometrical and structural imperfections that are inevitable in practical construction, and follower load effects.

1.3.5.4
design buckling stress
The design value of the buckling stress, obtained by dividing the characteristic buckling stress by the partial factor for resistance.

1.3.5.5
key value of the stress
The value of stress in a non-uniform stress field that is used to characterise the stress magnitudes in an LS3 assessment.

1.3.5.6
fabrication tolerance quality class
The category of fabrication tolerance requirements that is assumed in design.

1.4 Symbols

(1) In addition to those given in EN 1990 and EN 1993-1-1, the following symbols are used:

(2) Coordinate system (see figure 1.1):

- r radial coordinate, normal to the axis of revolution;
- x meridional coordinate;
- z axial coordinate;
- θ circumferential coordinate;
- ϕ meridional slope: angle between axis of revolution and normal to the meridian of the shell;

(3) Pressures:

- p_{n} normal to the shell;
- p_{x} meridional surface loading parallel to the shell;
- p_{θ} circumferential surface loading parallel to the shell;

(4) Line forces:

- P_{n} load per unit circumference normal to the shell;
- P_{x} load per unit circumference acting in the meridional direction;
- P_{θ} load per unit circumference acting circumferentially on the shell;

(5) Membrane stress resultants:

n_x meridional membrane stress resultant;
n_θ circumferential membrane stress resultant;
$n_{x\theta}$ membrane shear stress resultant;

(6) Bending stress resultants:

m_x meridional bending moment per unit width;
m_θ circumferential bending moment per unit width;
$m_{x\theta}$ twisting shear moment per unit width;
q_{xn} transverse shear force associated with meridional bending;
$q_{\theta n}$ transverse shear force associated with circumferential bending;

(7) Stresses:

σ_x meridional stress;
σ_θ circumferential stress;
σ_{eq} von Mises equivalent stress (can be negative in cyclic loading conditions);
τ, $\tau_{x\theta}$ in-plane shear stress;
τ_{xn}, $\tau_{\theta n}$ meridional, circumferential transverse shear stresses associated with bending;

(8) Displacements:

u meridional displacement;
v circumferential displacement;
w displacement normal to the shell surface;
β_ϕ meridional rotation (see 5.2.2);

(9) Shell dimensions:

d internal diameter of shell;
L total length of the shell;
ℓ length of shell segment;
ℓ_g gauge length for measurement of imperfections;
$\ell_{g\theta}$ gauge length for measurement of imperfections in circumferential direction;
ℓ_{gw} gauge length for measurement of imperfections across welds;
ℓ_R limited length of shell for buckling strength assessment;
r radius of the middle surface, normal to the axis of revolution;
t thickness of shell wall;
t_{max} maximum thickness of shell wall at a joint;
t_{min} minimum thickness of shell wall at a joint;
t_{ave} average thickness of shell wall at a joint;
β apex half angle of cone;

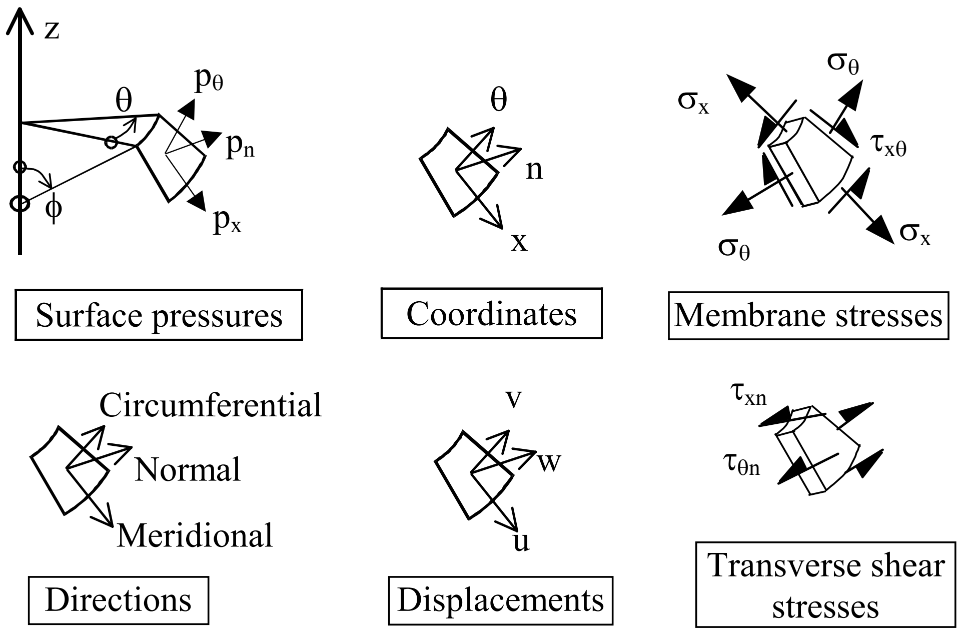


Figure 1.1: Symbols in shells of revolution

(10) Tolerances (see 8.4):

e	eccentricity between the middle surfaces of joined plates;
U_e	accidental eccentricity tolerance parameter;
U_r	out-of-roundness tolerance parameter;
U_n	initial dimple imperfection amplitude parameter for numerical calculations;
U_0	initial dimple tolerance parameter;
Δw_0	tolerance normal to the shell surface;

(11) Properties of materials:

E	Young's modulus of elasticity;
f_{eq}	von Mises equivalent strength;
f_y	yield strength;
f_u	ultimate strength;
ν	Poisson's ratio;

(12) Parameters in strength assessment:

C	coefficient in buckling strength assessment;
D	cumulative damage in fatigue assessment;
F	generalised action;
R	calculated resistance (used with subscripts to identify the basis);
R_{pl}	plastic reference resistance (defined as a load factor on design loads);
R_{cr}	elastic critical buckling resistance (defined as a load factor on design loads);
k	calibration factor for nonlinear analyses;
k	power of interaction expressions in buckling strength interaction expressions;
n	number of cycles of loading;
α	elastic imperfection reduction factor in buckling strength assessment;
β	plastic range factor in buckling interaction;
γ	partial factor;
Δ	range of parameter when alternating or cyclic actions are involved;
ε_p	plastic strain;
η	interaction exponent for buckling;
$\overline{\lambda}$	relative slenderness of shell;
$\overline{\lambda}_{ov}$	overall relative slenderness for the complete shell (multiple segments);

$\overline{\lambda}_0$	squash limit relative slenderness (value of $\overline{\lambda}$ at which stability reductions commence);
$\overline{\lambda}_p$	plastic limit relative slenderness (value of $\overline{\lambda}$ below which plasticity affects the stability);
ω	relative length parameter for shell;
χ	buckling reduction factor for elastic-plastic effects in buckling strength assessment;
χ_{ov}	overall buckling resistance reduction factor for complete shell;

(13) Subscripts:

E	value of stress or displacement (arising from design actions);
F	actions;
M	material;
R	resistance;
S	value of stress resultant (arising from design actions);
cr	critical buckling value;
d	design value;
int	internal;
k	characteristic value;
max	maximum value;
min	minimum value;
nom	nominal value;
pl	plastic value;
u	ultimate;
y	yield.

(14) Further symbols are defined where they first occur.

1.5 Sign conventions

(1) Outward direction positive: internal pressure positive, outward displacement positive, except as noted in (4).

(2) Tensile stresses positive, except as noted in (4).

NOTE: Compression is treated as positive in EN 1993-1-1.

(3) Shear stresses positive as shown in figures 1.1 and D.1.

(4) For simplicity, in section 8 and Annex D, compressive stresses are treated as positive. For these cases, both external pressures and internal pressures are treated as positive where they occur.

2 Basis of design and modelling

2.1 General

(1) The basis of design shall be in accordance with EN1990, as supplemented by the following.

(2) In particular, the shell shall be designed in such a way that it will sustain all actions and satisfy the following requirements:

- overall equilibrium;
- equilibrium between actions and internal forces and moments (see sections 6 and 8);
- limitation of cracks due to cyclic plastification (see section 7);
- limitation of cracks due to fatigue (see section 9).

(3) The design of the shell shall satisfy the serviceability requirements set out in the appropriate application standard (EN1993 Parts 3.1, 3.2, 4.1, 4.2, 4.3).

(4) The shell may be proportioned using design assisted by testing. Where appropriate, the requirements are set out in the appropriate application standard (EN1993 Parts 3.1, 3.2, 4.1, 4.2, 4.3).

(5) All actions should be introduced using their design values according to EN 1991 and EN 1993 Parts 3.1, 3.2, 4.1, 4.2, 4.3 as appropriate.

2.2 Types of analysis

2.2.1 General

(1) One or more of the following types of analysis should be used as detailed in section 4, depending on the limit state and other considerations:

- Global analysis (see 2.2.2);
- Membrane theory analysis (see 2.2.3);
- Linear elastic shell analysis (see 2.2.4);
- Linear elastic bifurcation analysis (see 2.2.5);
- Geometrically nonlinear elastic analysis (see 2.2.6);
- Materially nonlinear analysis (see 2.2.7);
- Geometrically and materially nonlinear analysis (see 2.2.8);
- Geometrically nonlinear elastic analysis with imperfections included (see 2.2.9);
- Geometrically and materially nonlinear analysis with imperfections included (see 2.2.10).

2.2.2 Global analysis

(1) A global analysis may involve approximate treatments of certain parts of the structure.

2.2.3 Membrane theory analysis

(1) A membrane theory analysis should not be used unless the following conditions are met:

- the boundary conditions are appropriate for transfer of the stresses in the shell into support reactions without causing bending effects;
- the shell geometry varies smoothly in shape (without discontinuities);
- the loads have a smooth distribution (without locally concentrated or point loads).

(2) A membrane theory analysis does not necessarily fulfil the compatibility of deformations at boundaries or between shell segments of different shape or between shell segments subjected to different loading. However, the resulting field of membrane forces satisfies the requirements of primary stresses (LS1).

2.2.4 Linear elastic shell analysis (LA)

(1) The linearity of the theory results from the assumptions of a linear elastic material law and the linear small deflection theory. Small deflection theory implies that the assumed geometry remains that of the undeformed structure.

(2) An LA analysis satisfies compatibility in the deformations as well as equilibrium. The resulting field of membrane and bending stress matches the requirements of primary plus secondary stresses (LS2).

2.2.5 Linear elastic bifurcation analysis (LBA)

(1) The conditions of 2.2.4 concerning the material and geometric assumptions are met. However, this linear bifurcation analysis obtains the lowest eigenvalue at which the shell may buckle into a different deformation mode, assuming no change of geometry, no change in the direction of action of the loads, and no material degradation. Imperfections of all kinds are ignored. This analysis provides the basis of the critical buckling resistance evaluation (LS3).

2.2.6 Geometrically nonlinear elastic analysis (GNA)

(1) A GNA analysis satisfies both equilibrium and compatibility of the deflections under conditions in which the change in the geometry of the structure caused by loading is included. The resulting field of stresses matches the definition of primary plus secondary stresses (LS2).

(2) Where compression or shear stresses are predominant in some part of the shell, a GNA analysis delivers the elastic buckling load of the perfect structure, including changes in geometry, that may be of assistance in checking the limit state LS3 (see 8.7).

(3) Where this analysis is used for a buckling load evaluation, the eigenvalues of the system must be checked to ensure that the numerical process does not fail to detect a bifurcation in the load path.

2.2.7 Materially nonlinear analysis (MNA)

(1) The result of an MNA analysis gives the plastic limit load, which can be interpreted as a load amplification factor R on the design value of the loads F_{Ed}. This may be used to verify limit state LS1. An MNA analysis can also be used to give the plastic strain increment $\Delta\varepsilon$ during one cycle of cyclic loading. This may be used to verify limit state LS2.

2.2.8 Geometrically and materially nonlinear analysis (GMNA)

(1) The result of a GMNA analysis, analogously to 2.2.5, gives the geometrically nonlinear plastic limit load of the perfect structure and the plastic strain increment, that may be used for checking the limit states LS1 and LS2.

(2) Where compression or shear stresses are predominant in some part of the shell, a GMNA analysis gives the elasto-plastic buckling load of the perfect structure, that may be of assistance in checking the limit state LS3 (see 8.7).

(3) Where this analysis is used for a buckling load evaluation, the eigenvalues of the system must be checked to ensure that the numerical process does not fail to detect a bifurcation in the load path.

2.2.9 Geometrically nonlinear elastic analysis with imperfections included (GNIA)

(1) A GNIA analysis is used in cases where compression or shear stresses dominate in the shell. It delivers elastic buckling loads of the "real" imperfect structure, that may be of assistance in checking the limit state LS3 (see 8.7).

(2) Where this analysis is used for a buckling load evaluation, the eigenvalues of the system must be checked to ensure that the numerical process does not fail to detect a bifurcation in the load path.

2.2.10 Geometrically and materially nonlinear analysis with imperfections included (GMNIA)

(1) A GMNIA analysis is used in cases where compression or shear stresses are dominant in the shell. It delivers elasto-plastic buckling loads for the "real" imperfect structure, that may be used for checking the limit state LS3.

(2) Where this analysis is used for a buckling load evaluation, the eigenvalues of the system must be checked to ensure that the numerical process does not fail to detect a bifurcation in the load path.

(3) Where this analysis is used for a buckling load evaluation, an additional GMNA analysis of the perfect shell should always be conducted to ensure that the degree of imperfection sensitivity of the structural system is identified.

2.3 Shell boundary conditions

(1) The boundary conditions assumed in the design calculation shall be chosen in such a way as to ensure that they achieve a realistic or conservative model of the real construction. Special attention shall be given not only to the constraint of displacements normal to the shell wall (deflections), but also to the constraint of the displacements in the plane of the shell wall (meridional and circumferential) because of the significant effect these have on shell strength and buckling resistance.

(2) In shell buckling (eigenvalue) calculations (limit state LS3), the definition of the boundary conditions shall refer to the incremental displacements during the buckling process, and not to total displacements induced by the applied actions before buckling.

(3) The boundary conditions at a continuously supported lower edge of a shell shall take into account whether local uplifting of the shell is prevented or not.

(4) The shell edge rotation β_ϕ should be particularly considered in short shells and in the calculation of secondary stresses in longer shells (according to the limit states LS2 and LS4).

(5) The boundary conditions set out in 5.2.2 should be used in computer analyses and in selecting expressions from Annexes A to D.

(6) The structural connections between shell segments at a junction should be such as to ensure that the boundary condition assumptions used in the design of the individual shell segments are satisfied.

3 Materials and geometry

3.1 Material properties

(1) The material properties of steels should be obtained from the relevant applications standards.

(2) Where materials with nonlinear stress-strain curves are involved and a buckling analysis is carried out under stress design (see 8.5), the initial tangent value of Young´s modulus E should be replaced by a reduced value. If no better method is available, the secant modulus at the 0,2% proof stress should be used when assessing the critical load or critical stress.

(3) Where the temperature exceeds 100°C, the material properties should be obtained from EN 13084-7.

(4) In a global numerical analysis using material nonlinearity, the stress-strain curve should be obtained from EN1993-1-5 Annex C for carbon steels and EN1993-1-4 Annex C for stainless steels.

3.2 Design values of geometrical data

(1) The thickness t of the shell shall be taken as defined in the relevant application standard. If no application standard is relevant, the nominal thickness of the wall, reduced by the prescribed value of the corrosion loss, shall be used.

(2) The thickness ranges within which the rules of this Standard may be applied are defined in the relevant EN1993 application parts.

(3) The middle surface of the shell shall be taken as the reference surface for loads.

(4) The radius r of the shell shall be taken as the nominal radius of the middle surface of the shell, measured normal to the axis of revolution.

(5) The buckling design rules of this Standard should not be applied outside the ranges of the r/t ratio set out in section 8 or Annex D or in the relevant EN1993 application parts.

3.3 Geometrical tolerances and geometrical imperfections

(1) Tolerance values for the deviations of the geometry of the shell surface from the nominal values are defined in the execution standards due to the requirements of serviceability. Relevant items are:

- out-of-roundness (deviation from circularity),
- eccentricities (deviations from a continuous middle surface in the direction normal to the shell along junctions of plates),
- local dimples (local normal deviations from the nominal middle surface).

NOTE: Until there is a European standard for execution, the tolerances can be obtained from this standard or the relevant application standards.

(2) If the limit state of buckling (LS3, as described in 4.1.3) is one of the ultimate limit states to be considered, additional buckling-relevant geometrical tolerances have to be observed in order to keep the geometrical imperfections within specified limits. These buckling-relevant geometrical tolerances are quantified in section 8 or in the relevant EN1993 application parts.

(3) Calculation values for the deviations of the shell surface geometry from the nominal geometry, as required for geometrical imperfection assumptions (overall imperfections or local imperfections) for the buckling design by global GMNIA analysis (see 8.7), shall be derived from the specified geometrical tolerances. Relevant rules are given in 8.7 or in relevant EN1993 application parts.

4 Ultimate limit states in steel shells

4.1 Ultimate limit states to be considered

4.1.1 LS1: Plastic limit

(1) The limit state of the plastic limit shall be taken as the condition in which the capacity of the structure to resist the actions on it is exhausted by yielding of the material. The resistance offered by the structure at the plastic limit state may be derived as the plastic collapse load obtained from a mechanism based on small displacement theory.

(2) The limit state of tensile rupture shall be taken as the condition in which the shell wall experiences gross section tensile failure, leading to separation of the two parts of the shell.

(3) In the absence of fastener holes, verification at the limit state of tensile rupture may be assumed to be covered by the check for the plastic limit state. However, where holes for fasteners occur, a supplementary check in accordance with 6.2 of EN 1993-1-1 should be carried out.

(4) In verifying the plastic limit state, plastic or partially plastic behaviour of the structure may be assumed (i.e. elastic compatibility considerations may be neglected).

NOTE: The basic characteristic of this limit state is that the load or actions sustained (resistance) cannot be increased without exploiting a significant change in the geometry of the structure or strain-hardening of the material.

(5) All relevant combinations of extreme loads shall be accounted for when checking LS1.

(6) The following methods of analysis (see 2.2) should be used for the calculation of the design stresses and stress resultants when checking LS1:
- membrane theory;
- expressions in Annexes A and B;
- linear elastic analysis (LA);
- materially nonlinear analysis (MNA);
- geometrically and materially nonlinear analysis (GMNA).

4.1.2 LS2: Cyclic plasticity

(1) The limit state of cyclic plasticity shall be taken as the condition in which repeated cycles of loading and unloading produce yielding in tension and in compression at the same point, thus causing plastic work to be repeatedly done on the structure, eventually leading to local cracking by exhaustion of the energy absorption capacity of the material.

NOTE: The stresses that are associated with this limit state develop under a combination of all actions and the compatibility conditions for the structure.

(2) All variable actions (such as imposed loads and temperature variations) that can lead to yielding, and which might be applied with more than three cycles in the life of the structure, shall be accounted for when checking LS2.

(3) In the verification of this limit state, compatibility of the deformations under elastic or elastic-plastic conditions should be considered.

(4) The following methods of analysis (see 2.2) should be used for the calculation of the design stresses and stress resultants when checking LS2:
- expressions in Annex C;
- elastic analysis (LA or GNA);
- MNA or GMNA and find plastic strains.

(5) Low cycle fatigue failure may be assumed to be prevented if the procedures set out in this standard are adopted.

4.1.3 LS3: Buckling

(1) The limit state of buckling shall be taken as the condition in which all or part of the structure suddenly develops large displacements normal to the shell surface, caused by loss of stability under compressive membrane or shear membrane stresses in the shell wall, leading to inability to sustain any increase in the stress resultants, possibly causing catastrophic failure.

(2) The following methods of analysis (see 2.2), as appropriate, should be used for the calculation of the design stresses and stress resultants when checking LS3:

- membrane theory for axisymmetric conditions only (for exceptions, see relevant application parts of EN 1993)
- expressions in Annex A;
- linear elastic analysis (LA), which is a minimum requirement for stress analysis under general loading conditions (unless the load case is given in Annex A);
- linear elastic bifurcation analysis (LBA), which is required for shells under general loading conditions if the critical buckling resistance is to be used;
- materially nonlinear analysis (MNA), which is required for shells under general loading conditions if the reference plastic resistance is to be used;
- GMNIA, coupled with MNA, LBA and GMNA, using appropriate imperfections and calculated calibration factors.

(3) All relevant combinations of extreme loads causing compressive membrane or shear membrane stresses in the shell shall be accounted for when checking LS3.

(4) Because the strength under limit state LS3 depends strongly on the quality of construction, the strength assessment shall take account of the associated requirements for fabrication tolerances.

NOTE: For this purpose, three fabrication quality classes are set out in section 8.

4.1.4 LS4: Fatigue

(1) The limit state of fatigue shall be taken as the condition in which repeated cycles of increasing and decreasing stress lead to the development of a fatigue crack.

(2) The following methods of analysis (see 2.2) should be used for the calculation of the design stresses and stress resultants when checking LS4:

- expressions in Annex C, using stress concentration factors;
- elastic analysis (LA or GNA), using stress concentration factors.

(3) All variable actions that will be applied with more than N_f cycles in the life of the structure according to the relevant action spectrum in EN 1991 in accordance with the appropriate application part of EN 1993-3 or EN 1993-4, should be accounted for when checking LS4.

NOTE: The National Annex may choose the value of N_f. The value N_f = 10 000 is recommended.

4.2 Design concepts for the limit states design of shells

4.2.1 General

(1) The limit state verification should be carried out using one of the following:

- stress design;
- direct design by application of standard expressions;
- design by global numerical analysis (for example, by means of computer programs such as those based on the finite element method).

(2) Account should be taken of the fact that elasto-plastic material responses induced by different stress components in the shell have different effects on the failure modes and the ultimate limit states. The stress components should therefore be placed in stress categories with different limits. Stresses that develop to meet equilibrium requirements should be treated as more significant than stresses that are induced by the compatibility of deformations normal to the shell. Local stresses caused by notch effects in construction details may be assumed to have a negligibly small influence on the resistance to static loading.

(3) The categories distinguished in the stress design should be primary, secondary and local stresses. Primary and secondary stress states may be replaced by stress resultants where appropriate.

(4) In a global analysis, the primary and secondary stress states should be replaced by the limit load and the strain range for cyclic loading.

(5) In general, it may be assumed that primary stress states control LS1, whereas secondary stress states affect LS2 and LS3 and local stresses govern LS4.

4.2.2 Stress design

4.2.2.1 General

(1) Where the stress design approach is used, the limit states should be assessed in terms of three categories of stress: primary, secondary and local. The categorisation is performed, in general, on the von Mises equivalent stress at a point, but buckling stresses cannot be assessed using this value.

4.2.2.2 Primary stresses

(1) The primary stresses should be taken as the stress system required for equilibrium with the imposed loading. They may be calculated from any realistic statically admissible determinate system. The limit state should be deemed to be reached when the primary stress reaches the yield strength throughout the full thickness of the wall at a sufficient number of points, such that only the strain hardening reserve or a change of geometry would lead to an increase in the resistance of the structure.

(2) The calculation of primary stresses should be based on any system of stress resultants, consistent with the requirements of equilibrium of the structure. It may also take into account the benefits of plasticity theory. Alternatively, since linear elastic analysis satisfies equilibrium requirements, its predictions may also be used as a representation of the limit state. Any of the methods given in 5.3 may be applied.

(3) Because limit state design allows for full plastification of the cross-section, the primary stresses due to bending moments may be calculated on the basis of the plastic section modulus (see 6.2.1). Where there is interaction between stress resultants in the cross-section, interaction rules based on the von Mises yield criterion may be applied.

(4) The primary stresses should be limited by the design value of the yield strength (see section 6).

4.2.2.3 Secondary stresses

(1) In statically indeterminate structures, account should be taken of the secondary stresses, induced by internal compatibility and compatibility with the boundary conditions, that are caused by imposed loading or imposed displacements (temperature, prestressing, settlement, shrinkage).

NOTE: As the von Mises yield condition is approached, the displacements of the structure increase without further increase in the stress state.

(2) Where cyclic loading causes plasticity, and several loading cycles occur, consideration should be given to the possible reduction of resistance caused by the secondary stresses. Where the cyclic loading is of such a magnitude that yielding occurs at both the maximum load and again on unloading, account should be taken of a possible failure by cyclic plasticity associated with the secondary stresses.

(3) If the stress calculation is carried out using a linear elastic analysis that allows for all relevant compatibility conditions (effects at boundaries, junctions, variations in wall thickness etc.), the stresses that vary linearly through the thickness may be taken as the sum of the primary and secondary stresses and used in an assessment involving the von Mises yield criterion (see 6.2).

NOTE: The secondary stresses are never needed separately from the primary stresses.

(4) The secondary stresses should be limited as follows:

- The sum of the primary and secondary stresses (including bending stresses) should be limited to $2f_y$ for the condition of cyclic plasticity (LS2: see section 7);
- The membrane component of the sum of the primary and secondary stresses should be limited by the design buckling resistance (LS3: see section 8).

- The sum of the primary and secondary stresses (including bending stresses) should be limited to the fatigue resistance (LS4: see section 9).

4.2.2.4 Local stresses

(1) The highly localised stresses associated with stress raisers in the shell wall due to notch effects (holes, welds, stepped walls, attachments, and joints) should be taken into account in a fatigue assessment (LS4).

(2) For construction details given in EN 1993-1-9, the fatigue design may be based on the nominal linear elastic stresses (sum of the primary and secondary stresses) at the relevant point. For all other details, the local stresses may be calculated by applying stress concentration factors (notch factors) to the stresses calculated using a linear elastic stress analysis.

(3) The local stresses should be limited according to the requirements for fatigue (LS4) set out in section 9.

4.2.3 Direct design

(1) Where direct design is used, the limit states may be represented by standard expressions that have been derived from either membrane theory, plastic mechanism theory or linear elastic analysis.

(2) The membrane theory expressions given in Annex A may be used to determine the primary stresses needed for assessing LS1 and LS3.

(3) The expressions for plastic design given in Annex B may be used to determine the plastic limit loads needed for assessing LS1.

(4) The expressions for linear elastic analysis given in Annex C may be used to determine stresses of the primary plus secondary stress type needed for assessing LS2 and LS4. An LS3 assessment may be based on the membrane part of these expressions.

4.2.4 Design by global numerical analysis

(1) Where a global numerical analysis is used, the assessment of the limit states shall be carried out using one of the alternative types of analysis specified in 2.2 (but not membrane theory analysis) applied to the complete structure.

(2) Linear elastic analysis (LA) may be used to determine stresses or stress resultants, for use in assessing LS2 and LS4. The membrane parts of the stresses may be used in assessing LS3. LS1 may be assessed using LA, but LA only gives an approximate estimate and its results should be interpreted as set out in section 5.

(3) Linear elastic bifurcation analysis (LBA) may be used to determine the elastic critical buckling resistance of the structure, for use in assessing LS3.

(4) A materially nonlinear analysis (MNA) may be used to determine plastic limit loads, that may be used for assessing LS1. Under a cyclic loading history, an MNA analysis may be used to determine plastic strain incremental changes, for use in assessing LS2. An MNA analysis may be used to determine the reference plastic load required as part of the assessment of LS3.

(5) Geometrically nonlinear elastic analyses (GNA and GNIA) include consideration of the deformations of the structure, but none of the design methodologies of section 8 permit these to be used without a GMNIA analysis. A GNA analysis may be used to determine the elastic buckling load of the perfect structure. A GNIA analysis may be used to determine the elastic buckling load of the imperfect structure.

(6) Geometrically and materially nonlinear analysis may be used to determine collapse loads for the imperfect structure (GMNIA). These collapse loads may be used for assessing LS3. For design purposes the analysis should be interpreted as detailed in 6.3 and 8.7 respectively. Under a cyclic loading history, the plastic strain incremental changes for the perfect structure may be used for assessing LS2.

5 Stress resultants and stresses in shells

5.1 Stress resultants in the shell

(1) In principle, the eight stress resultants in the shell wall at any point should be calculated and the assessment of the shell with respect to each limit state should take all of them into account. However, the shear stresses τ_{xn}, $\tau_{\theta n}$ due to the transverse shear forces q_{xn}, $q_{\theta n}$ are insignificant compared with the other components of stress in almost all practical cases, so they may usually be neglected in design.

(2) Accordingly, for most design purposes, the evaluation of the limit states may be made using only the six stress resultants in the shell wall n_x, n_θ, $n_{x\theta}$, m_x, m_θ, $m_{x\theta}$. Where the structure is axisymmetric and subject only to axisymmetric loading and support, only n_x, n_θ, m_x and m_θ need be used.

(3) If any uncertainty arises concerning the stress to be used in any of the limit state verifications, the von Mises equivalent stress on the shell surface should be used.

5.2 Modelling of the shell for analysis

5.2.1 Geometry

(1) The shell shall be represented by its middle surface.

(2) The radius of curvature shall be taken as the nominal radius of curvature. Imperfections shall be neglected, except as set out in section 8 (LS3 buckling limit state).

(3) An assembly of shell segments shall not be subdivided into separate segments for analysis unless the boundary conditions for each segment are chosen in such as way as to represent interactions between them in a conservative manner.

(4) A base ring intended to transfer local support forces into the shell shall not be separated from the shell it supports in an assessment of limit state LS3.

(5) Eccentricities and steps in the shell middle surface shall be included in the analysis model if they induce significant bending effects as a result of the membrane stress resultants following an eccentric path.

(6) At junctions between shell segments, any eccentricity between the middle surfaces of the shell segments shall be considered in the modelling.

(7) A ring stiffener should be treated as a separate structural component of the shell, except where the spacing of the rings is closer than $1{,}5\sqrt{rt}$.

(8) A shell that has discrete stringer stiffeners attached to it may be treated as an orthotropic uniform shell, provided that the stringer stiffeners are no further apart than $5\sqrt{rt}$.

(9) A shell that is corrugated (vertically or horizontally) may be treated as an orthotropic uniform shell provided that the corrugation wavelength is less than $0{,}5\sqrt{rt}$.

(10) A hole in the shell may be neglected in the modelling provided its largest dimension is smaller than $0{,}5\sqrt{rt}$.

(11) The overall stability of the complete structure should be verified as detailed in EN1993 Parts 3.1, 3.2, 4.1, 4.2 or 4.3 as appropriate.

5.2.2 Boundary conditions

(1) The appropriate boundary conditions should be used in analyses for the assessment of limit states according to the conditions shown in table 5.1. For the special conditions needed for buckling calculations, reference should be made to 8.4.

(2) Rotational restraints at shell boundaries may be neglected in modelling for limit state LS1, but should be included in modelling for limit states LS2 and LS4. For short shells (see Annex D), the rotational restraint should be included for limit state LS3.

(3) Support boundary conditions should be checked to ensure that they do not cause excessive non-uniformity of transmitted forces or introduced forces that are eccentric to the shell middle surface. Reference should be made to the relevant EN 1993 application parts for the detailed application of this rule to silos and tanks.

(4) When a global numerical analysis is used, the boundary condition for the normal displacement w should also be used for the circumferential displacement v, except where special circumstances make this inappropriate.

Table 5.1: Boundary conditions for shells

Boundary condition code	Simple term	Description	Normal displacements	Vertical displacements	Meridional rotation
BC1r	Clamped	radially restrained meridionally restrained rotation restrained	$w = 0$	$u = 0$	$\beta_\phi = 0$
BC1f		radially restrained meridionally restrained rotation free	$w = 0$	$u = 0$	$\beta_\phi \neq 0$
BC2r		radially restrained meridionally free rotation restrained	$w = 0$	$u \neq 0$	$\beta_\phi = 0$
BC2f	Pinned	radially restrained meridionally free rotation free	$w = 0$	$u \neq 0$	$\beta_\phi \neq 0$
BC3	Free edge	radially free meridionally free rotation free	$w \neq 0$	$u \neq 0$	$\beta_\phi \neq 0$
NOTE: The circumferential displacement v is closely linked to the displacement w normal to the surface so separate boundary conditions are not identified in paragraph (3) for these two parameters.					

5.2.3 Actions and environmental influences

(1) Actions shall all be assumed to act at the shell middle surface. Eccentricities of load shall be represented by static equivalent forces and moments at the shell middle surface.

(2) Local actions and local patches of action shall not be represented by equivalent uniform loads except as detailed in section 8 (LS3 for buckling).

(3) The modelling should account for whichever of the following are relevant:

- local settlement under shell walls;
- local settlement under discrete supports;
- uniformity of support of structure;
- thermal differentials from one side of the structure to the other;
- thermal differentials from inside to outside the structure;
- wind effects on openings and penetrations;
- interaction of wind effects on groups of structures;
- connections to other structures;
- conditions during erection.

5.2.4 Stress resultants and stresses

(1) Provided that the radius to thickness ratio is greater than $(r/t)_{min}$, the curvature of the shell may be ignored when calculating the stress resultants from the stresses in the shell wall.

NOTE: The National Annex may choose the value of $(r/t)_{min}$. The value $(r/t)_{min} = 25$ is recommended.

5.3 Types of analysis

(1) The design should be based on one or more of the types of analysis given in table 5.2. Reference should be made to 2.2 for the conditions governing the use of each type of analysis.

Table 5.2: Types of shell analysis

Type of analysis	Shell theory	Material law	Shell geometry
Membrane theory of shells	membrane equilibrium	not applicable	perfect
Linear elastic shell analysis (LA)	linear bending and stretching	linear	perfect
Linear elastic bifurcation analysis (LBA)	linear bending and stretching	linear	perfect
Geometrically non-linear elastic analysis (GNA)	non-linear	linear	perfect
Materially non-linear analysis (MNA)	linear	non-linear	perfect
Geometrically and materially non-linear analysis (GMNA)	non-linear	non-linear	perfect
Geometrically non-linear elastic analysis with imperfections (GNIA)	non-linear	linear	imperfect
Geometrically and materially non-linear analysis with imperfections (GMNIA)	non-linear	non-linear	imperfect

6 Plastic limit state (LS1)

6.1 Design values of actions

(1) The design values of the actions shall be based on the most adverse relevant load combination (including the relevant γ_F and ψ factors).

(2) Only those actions that represent loads affecting the equilibrium of the structure need be included.

6.2 Stress design

6.2.1 Design values of stresses

(1) Although stress design is based on an elastic analysis and therefore cannot accurately predict the plastic limit state, it may be used, on the basis of the lower bound theorem, to provide a conservative assessment of the plastic collapse resistance which is used to represent the plastic limit state (see 4.1.1).

(2) The Ilyushin yield criterion may be used, as detailed in (6), that comes closer to the true plastic collapse state than a simple elastic stress evaluation.

(3) At each point in the structure the design value of the stress $\sigma_{eq,Ed}$ should be taken as the highest primary stress determined in a structural analysis that considers the laws of equilibrium between imposed design load and internal forces and moments.

(4) The primary stress may be taken as the maximum value of the stresses required for equilibrium with the applied loads at a point or along a line in the shell structure.

(5) Where a membrane theory analysis is used, the resulting two dimensional field of stress resultants $n_{x,Ed}$, $n_{\theta,Ed}$ and $n_{x\theta,Ed}$ may be represented by the equivalent design stress $\sigma_{eq,Ed}$ obtained from:

$$\sigma_{eq,Ed} = \frac{1}{t}\sqrt{n_{x,Ed}^2 + n_{\theta,Ed}^2 - n_{x,Ed}\,n_{\theta,Ed} + 3n_{x\theta,Ed}^2}. \quad \text{... (6.1)}$$

(6) Where an LA or GNA analysis is used, the resulting two dimensional field of primary stresses may be represented by the von Mises equivalent design stress:

$$\sigma_{eq,Ed} = \sqrt{\sigma_{x,Ed}^2 + \sigma_{\theta,Ed}^2 - \sigma_{x,Ed}\,\sigma_{\theta,Ed} + 3(\tau_{x\theta,Ed}^2 + \tau_{xn,Ed}^2 + \tau_{\theta n,Ed}^2)} \quad \text{... (6.2)}$$

in which:

$$\sigma_{x,Ed} = \frac{n_{x,Ed}}{t} \pm \frac{m_{x,Ed}}{(t^2/4)}, \qquad \sigma_{\theta,Ed} = \frac{n_{\theta,Ed}}{t} \pm \frac{m_{\theta,Ed}}{(t^2/4)}, \quad \text{... (6.3)}$$

$$\tau_{x\theta,Ed} = \frac{n_{x\theta,Ed}}{t} \pm \frac{m_{x\theta,Ed}}{(t^2/4)}, \qquad \tau_{xn,Ed} = \frac{q_{xn,Ed}}{t}, \qquad \tau_{\theta n,Ed} = \frac{q_{\theta n,Ed}}{t} \quad \text{... (6.4)}$$

NOTE1: The above expressions give a simplified conservative equivalent stress for design purposes.

NOTE2: The values of $\tau_{xn,Ed}$ and $\tau_{\theta n,Ed}$ are usually very small and do not affect the plastic resistance, so they may generally be ignored.

6.2.2 Design values of resistance

(1) The von Mises design strength should be taken from:

$$f_{eq,Rd} = f_y / \gamma_{M0} \quad \text{... (6.5)}$$

(2) The partial factor for resistance γ_{M0} should be taken from the relevant application standard.

(3) Where no application standard exists for the form of construction involved, or the application standard does not define the relevant values of γ_{M0}, the value of γ_{M0} should be taken from EN1993-1-1.

(4) The effect of fastener holes should be taken into account in accordance with 6.2.3 of EN 1993-1-1 for tension and 6.2.4 of EN 1993-1-1 for compression. For the tension check, the resistance should be based on the design value of the ultimate strength f_{ud}.

6.2.3 Stress limitation

(1) In every verification of this limit state, the design stresses should satisfy the condition:

$$\sigma_{eq,Ed} \leq f_{eq,Rd} \quad \text{... (6.6)}$$

6.3 Design by global numerical MNA or GMNA analysis

(1) The design plastic limit resistance shall be determined as a load factor R applied to the design values of the combination of actions for the relevant load case.

(2) The design values of the actions F_{Ed} should be determined as detailed in 6.1. The relevant load cases should be formed according to the required load combinations.

(3) In an MNA or GMNA analysis based on the design yield strength f_{yd}, the shell should be subject to the design values of the load cases detailed in (2), progressively increased by the load ratio R until the plastic limit condition is reached.

(4) Where an MNA analysis is used, the load ratio R_{MNA} may be taken as the largest value attained in the analysis, ignoring the effect of strain hardening.

(5) Where a GMNA analysis is used, if the analysis predicts a maximum load followed by a descending path, the maximum value should be used to determine the load ratio R_{GMNA}. Where a GMNA analysis does not predict a maximum load, but produces a progressively rising action-displacement relationship without strain hardening of the material, the load ratio R_{GMNA} should be taken as no larger than the value at which the maximum von Mises equivalent plastic strain in the structure attains the value $\varepsilon_{mps} = n_{mps} (f_{yd} / E)$.

NOTE: The National Annex may choose the value of n_{mps}. The value $n_{mps} = 50$ is recommended.

(6) The result of the analysis should produce a load ratio R greater than 1,0: that is, it should satisfy the condition:

$$F_{Ed} \leq F_{Rd} \quad \text{... (6.7)}$$

in which:

$$F_{Rd} = R\, F_{Ed}. \quad \text{... (6.8)}$$

6.4 Direct design

(1) For each shell segment in the structure represented by a basic loading case as given by Annex A, the highest von Mises membrane stress $\sigma_{eq,Ed}$ determined under the design values of the actions F_{Ed} should be limited to the stress resistance according to 6.2.2.

(2) For each shell or plate segment in the structure represented by a basic load case as given in Annex B, the design value of the actions F_{Ed} should not exceed the load resistance F_{Rd} based on the design yield strength f_{yd}.

(3) Where net section failure at a bolted joint is a design criterion, the design value of the actions F_{Ed} should be determined for each joint. Where the stress can be represented by a basic load case as given in Annex A, and where the resulting stress state involves only membrane stresses, F_{Ed} should not exceed the load resistance F_{Rd} based on the design ultimate strength f_{ud} (see 6.2.2(4)).

7 Cyclic plasticity limit state (LS2)

7.1 Design values of actions

(1) Unless an improved definition is used, the design values of the actions for each load case should be chosen as the characteristic values of those parts of the total actions that are expected to be applied and removed more than three times in the design life of the structure.

(2) Where an elastic analysis or the expressions from Annex C are used, only the varying part of the actions between the extreme upper and lower values should be taken into account.

(3) Where a materially nonlinear computer analysis is used, the varying part of the actions between the extreme upper and lower values should be considered to act in the presence of coexistent fixed parts of the load.

7.2 Stress design

7.2.1 Design values of stress range

(1) The shell should be analysed using an LA or GNA analysis of the structure subject to the two extreme design values of the actions F_{Ed}. For each extreme load condition in the cyclic process, the stress components should be evaluated. From adjacent extremes in the cyclic process, the design values of the change in each stress component $\Delta\sigma_{x,Ed}$, $\Delta\sigma_{\theta,Ed}$, $\Delta\tau_{x\theta,Ed}$ on each shell surface and at any point in the structure should be determined. From these changes in stress, the design value of the von Mises equivalent stress change on each surface $\Delta\sigma_{eq,Ed,i}$ should be found from:

$$\Delta\sigma_{eq,Ed,i} = \sqrt{\Delta\sigma_{x,Ed}^2 - \Delta\sigma_{x,Ed}\,\Delta\sigma_{\theta,Ed} + \Delta\sigma_{\theta,Ed}^2 + 3\Delta\tau_{x\theta,Ed}^2} \quad \text{... (7.1)}$$

(2) The design value of the stress range $\Delta\sigma_{eq,Ed}$ should be taken as the largest change in the von Mises equivalent stress changes $\Delta\sigma_{eq,Ed,i}$, considering each shell surface in turn.

(3) At a junction between shell segments, where the analysis models the intersection of the middle surfaces and ignores the finite size of the junction, the stress range may be taken at the first physical point in the shell segment (as opposed to the value calculated at the intersection of the two middle surfaces).

NOTE: This allowance is relevant where the stress changes very rapidly close to the junction.

7.2.2 Design values of resistance

(1) The von Mises equivalent stress range resistance $\Delta f_{eq,Rd}$ should be determined from:

$$\Delta f_{eq,Rd} = 2\, f_{yd} \quad \text{... (7.2)}$$

7.2.3 Stress range limitation

(1) In every verification of this limit state, the design stress range should satisfy:

$$\Delta\sigma_{eq,Ed} \leq \Delta f_{eq,Rd} \quad \text{... (7.3)}$$

7.3 Design by global numerical MNA or GMNA analysis

7.3.1 Design values of total accumulated plastic strain

(1) Where a materially nonlinear global numerical analysis (MNA or GMNA) is used, the shell should be subject to the design values of the varying and fixed actions detailed in 7.1.

(2) The total accumulated von Mises equivalent plastic strain $\varepsilon_{p,eq,Ed}$ at the end of the design life of the structure should be assessed.

(3) The total accumulated von Mises equivalent plastic strain may be determined using an analysis that models all cycles of loading during the design life.

(4) Unless a more refined analysis is carried out, the total accumulated von Mises equivalent plastic strain $\varepsilon_{p,eq,Ed}$ may be determined from:

$$\varepsilon_{p,eq,Ed} = n\,\Delta\varepsilon_{p,eq,Ed} \qquad \text{... (7.4)}$$

where:

n is the number of cycles of loading in the design life of the structure;

$\Delta\varepsilon_{p,eq,Ed}$ is the largest increment in the von Mises equivalent plastic strain during one complete load cycle at any point in the structure, occurring after the third cycle.

(5) It may be assumed that "at any point in the structure" means at any point not closer to a notch or local discontinuity than the thickest adjacent plate thickness.

NOTE 1: It is usual to use an MNA analysis for this purpose.

NOTE 2: The National Annex may give recommendations for a more refined analysis.

7.3.2 Total accumulated plastic strain limitation

(1) Unless a more sophisticated low cycle fatigue assessment is undertaken, the design value of the total accumulated von Mises equivalent plastic strain $\varepsilon_{p,eq,Ed}$ should satisfy the condition:

$$\varepsilon_{p,eq,Ed} \leq n_{p,eq}\ (f_{yd} / E) \qquad \text{... (7.5)}$$

NOTE: The National Annex may choose the value of $n_{p,eq}$. The value $n_{p,eq} = 5$ is recommended.

7.4 Direct design

(1) For each shell segment in the structure, represented by a basic loading case as given by Annex C, the highest von Mises equivalent stress range $\Delta\sigma_{eq,Ed}$ considering both shell surfaces under the design values of the actions F_{Ed} should be determined using the relevant expressions given in Annex C. The further assessment procedure should be as detailed in 7.2.

8 Buckling limit state (LS3)

8.1 Design values of actions

(1) All relevant combinations of actions causing compressive membrane stresses or shear membrane stresses in the shell wall shall be taken into account.

8.2 Special definitions and symbols

(1) Reference should be made to the special definitions of terms concerning buckling in 1.3.5.

(2) In addition to the symbols defined in 1.4, additional symbols should be used in this section 8 as set out in (3) and (4).

(3) The stress resultant and stress quantities should be taken as follows:

$n_{x,Ed}$, $\sigma_{x,Ed}$ are the design values of the acting buckling-relevant meridional membrane stress resultant and stress (positive when compression);

$n_{\theta,Ed}$, $\sigma_{\theta,Ed}$ are the design values of the acting buckling-relevant circumferential membrane (hoop) stress resultant and stress (positive when compression);

$n_{x\theta,Ed}$, $\tau_{x\theta,Ed}$ are the design values of the acting buckling-relevant shear membrane stress resultant and stress.

(4) Buckling resistance parameters for use in stress design:

$\sigma_{x,Rcr}$ is the meridional critical buckling stress;
$\sigma_{\theta,Rcr}$ is the circumferential critical buckling stress;
$\tau_{x\theta,Rcr}$ is the shear critical buckling stress;
$\sigma_{x,Rk}$ is the meridional characteristic buckling stress;
$\sigma_{\theta,Rk}$ is the circumferential characteristic buckling stress;
$\tau_{x\theta,Rk}$ is the shear characteristic buckling stress;
$\sigma_{x,Rd}$ is the meridional design buckling stress;
$\sigma_{\theta,Rd}$ is the circumferential design buckling stress;
$\tau_{x\theta,Rd}$ is the shear design buckling stress.

NOTE: This is a special convention for shell design that differs from that detailed in EN1993-1-1.

(5) The sign convention for use with LS3 should be taken as compression positive for meridional and circumferential stresses and stress resultants.

8.3 Buckling-relevant boundary conditions

(1) For the buckling limit state, special attention should be paid to the boundary conditions which are relevant to the incremental displacements of buckling (as opposed to pre-buckling displacements). Examples of relevant boundary conditions are shown in figure 8.1, in which the codes of Table 5.1 are used.

8.4 Buckling-relevant geometrical tolerances

8.4.1 General

(1) Unless specific buckling-relevant geometrical tolerances are given in the relevant EN 1993 application parts, the following tolerance limits should be observed if LS3 is one of the ultimate limit states to be considered.

NOTE 1: The characteristic buckling stresses determined hereafter include imperfections that are based on the amplitudes and forms of geometric tolerances that are expected to be met during execution.

NOTE 2: The geometric tolerances given here are those that are known to have a large impact on the safety of the structure.

(2) The fabrication tolerance quality class should be chosen as Class A, Class B or Class C according to the tolerance definitions in 8.4.2, 8.4.3, 8.4.4 and 8.4.5. The description of each class relates only to the strength evaluation.

NOTE: Until there is an execution standard that classifies geometrical tolerances according to safety considerations, these tolerances are intended to be adopted in execution.

(3) Each of the imperfection types should be classified separately: the lowest class should then govern the entire design.

(4) The different tolerance types may each be treated independently, and no interactions need normally be considered.

(5) It should be established by representative sample checks that the measurements of the geometrical imperfections stay within the geometrical tolerances stipulated in 8.4.2 to 8.4.4.

(6) Sample imperfection measurements should be undertaken on the unloaded structure (except for self weight) and, where possible, with the operational boundary conditions.

(7) If the measurements of geometrical imperfections do not satisfy the geometrical tolerances stated in 8.4.2 to 8.4.4, any correction steps, such as by straightening, should be investigated and decided individually.

NOTE: Before a decision is made in favour of straightening to reduce geometric imperfections, it should be noted that this can cause additional residual stresses. The degree to which the design buckling resistances are utilised in the design should also be considered.

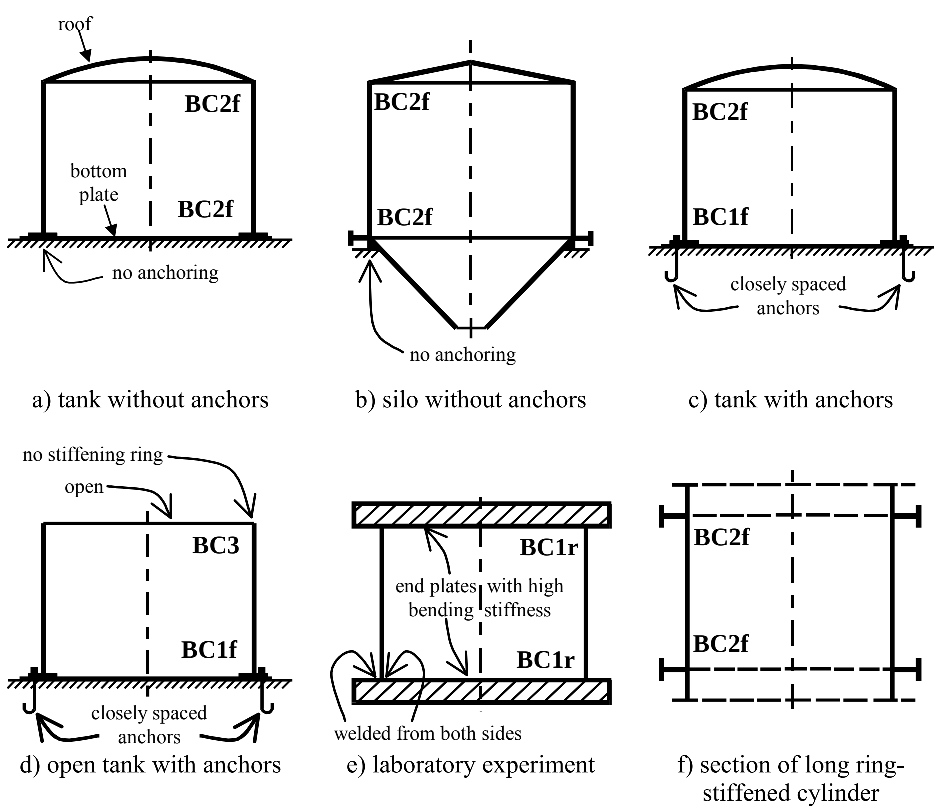


Figure 8.1: Schematic examples of boundary conditions for limit state LS3

8.4.2 Out-of-roundness tolerance

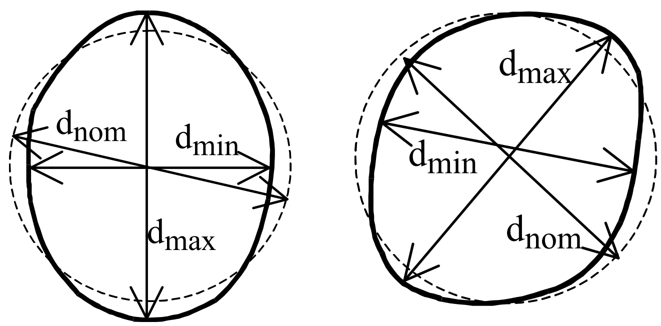
(1) The out-of-roundness should be assessed in terms of the parameter U_r (see figure 8.2) given by:

$$U_r = \frac{d_{max} - d_{min}}{d_{nom}} \qquad \text{... (8.1)}$$

where:

d_{max} is the maximum measured internal diameter,
d_{min} is the minimum measured internal diameter,
d_{nom} is the nominal internal diameter.

(2) The measured internal diameter from a given point should be taken as the largest distance across the shell from the point to any other internal point at the same axial coordinate. An appropriate number of diameters should be measured to identify the maximum and minimum values.



a) flattening b) unsymmetrical

Figure 8.2: Measurement of diameters for assessment of out-of-roundness

(3) The out-of-roundness parameter U_r should satisfy the condition:

$$U_r \leq U_{r,max} \qquad \text{... (8.2)}$$

where:

$U_{r,max}$ is the out-of-roundness tolerance parameter for the relevant fabrication tolerance quality class.

NOTE: Values for the out-of-roundness tolerance parameter $U_{r,max}$ may be obtained from the National Annex. The recommended values are given in Table 8.1.

Table 8.1: Values for out-of-roundness tolerance parameter $U_{r,max}$

	Diameter range	$d \leq 0{,}50$m	$0{,}50\text{m} < d < 1{,}25$m	$1{,}25\text{m} \leq d$
Fabrication tolerance quality class	Description	Value of $U_{r,max}$		
Class A	Excellent	0,014	0,007 + 0,0093(1,25−d)	0,007
Class B	High	0,020	0,010 + 0,0133(1,25−d)	0,010
Class C	Normal	0,030	0,015 + 0,0200(1,25−d)	0,015

8.4.3 Accidental eccentricity tolerance

(1) At joints in shell walls perpendicular to membrane compressive forces, the accidental eccentricity should be evaluated from the measurable total eccentricity e_{tot} and the intended offset e_{int} from:

$$e_a = e_{tot} - e_{int} \qquad \text{... (8.3)}$$

where:

e_{tot} is the eccentricity between the middle surfaces of the joined plates (see figure 8.3c);
e_{int} is the intended offset between the middle surfaces of the joined plates (see figure 8.3b);

e_a is the accidental eccentricity between the middle surfaces of the joined plates.

(2) The accidental eccentricity e_a should satisfy the maximum permitted accidental eccentricity for the relevant fabrication tolerance quality class.

NOTE: Values for the maximum permitted accidental eccentricity may be obtained from the National Annex. The recommended values are given in Table 8.2.

Table 8.2: Values for maximum permitted accidental eccentricities

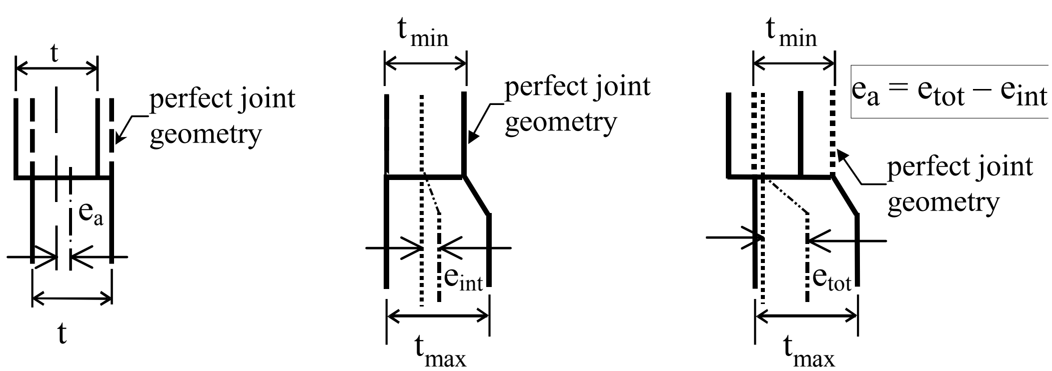
Fabrication tolerance quality class	Description	Maximum permitted accidental eccentricity
Class A	Excellent	$e_a \leq 2$ mm
Class B	High	$e_a \leq 3$ mm
Class C	Normal	$e_a \leq 4$ mm

(3) The accidental eccentricity e_a should also be assessed in terms of the accidental eccentricity parameter U_e given by:

$$U_e = \frac{e_a}{t_{ave}} \quad \text{or} \quad U_e = \frac{e_a}{t} \qquad \text{... (8.4)}$$

where:

t_{ave} is the mean thickness of the thinner and thicker plates at the joint.



a) accidental eccentricity when there is no change of plate thickness

b) intended offset at a change of plate thickness without accidental eccentricity

c) total eccentricity (accidental plus intended) at change of plate thickness

Figure 8.3: Accidental eccentricity and intended offset at a joint

(4) The accidental eccentricity parameter U_e should satisfy the condition:

$$U_e \leq U_{e,max} \qquad \text{... (8.5)}$$

where:

$U_{e,max}$ is the accidental eccentricity tolerance parameter for the relevant fabrication tolerance quality class.

NOTE 1: Values for the out-of-roundness tolerance parameter $U_{e,max}$ may be obtained from the National Annex. The recommended values are given in Table 8.3.

Table 8.3: Values for accidental eccentricity tolerances

Fabrication tolerance quality class	Description	Value of $U_{e,max}$
Class A	Excellent	0,14
Class B	High	0,20
Class C	Normal	0,30

NOTE 2: Intended offsets are treated within D.2.1.2 and lapped joints are treated within D.3. These two cases are not treated as imperfections within this standard.

8.4.4 Dimple tolerances

(1) A dimple measurement gauge should be used in every position (see figure 8.4) in both the meridional and circumferential directions. The meridional gauge should be straight, but the gauge for measurements in the circumferential direction should have a curvature equal to the intended radius of curvature r of the middle surface of the shell.

(2) The depth Δw_0 of initial dimples in the shell wall should be measured using gauges of length ℓ_g which should be taken as follows:

a) Wherever axial compressive stresses are present, including across welds, in both the meridional and circumferential directions, measurements should be made using the gauge of length ℓ_{gx} given by:

$$\ell_{gx} = 4\sqrt{rt} \qquad \text{... (8.6)}$$

b) Where circumferential compressive stresses or shear stresses occur, circumferential direction measurements should be made using the gauge of length $\ell_{g\theta}$ given by:

$$\ell_{g\theta} = 2{,}3\,(\ell^2\, \mathrm{rt})^{0,25}, \qquad \text{but} \qquad \ell_{g\theta} \leq r \qquad \text{... (8.7)}$$

where:

ℓ is the axial length of the shell segment.

c) Additionally, across welds, in both the circumferential and meridional directions, the gauge length ℓ_{gw} should be used:

$$\ell_{gw} = 25\,t \quad \text{or} \quad \ell_{gw} = 25\,t_{min}, \quad \text{but with } \ell_{gw} \leq 500\text{mm} \qquad \text{... (8.8)}$$

where:

t_{min} is the thickness of the thinnest plate at the weld.

(3) The depth of initial dimples should be assessed in terms of the dimple parameters U_{0x}, $U_{0\theta}$, U_{0w} given by:

$$U_{0x} = \Delta w_{0x}/\ell_{gx} \qquad U_{0\theta} = \Delta w_{0\theta}/\ell_{g\theta} \qquad U_{0w} = \Delta w_{0w}/\ell_{gw} \qquad \text{... (8.9)}$$

(4) The value of the dimple parameters U_{0x}, $U_{0\theta}$, U_{0w} should satisfy the conditions:

$$U_{0x} \leq U_{0,max} \qquad U_{0\theta} \leq U_{0,max} \qquad U_{0w} \leq U_{0,max} \qquad \text{... (8.10)}$$

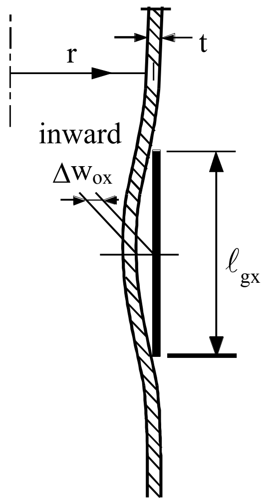
where:

$U_{0,max}$ is the dimple tolerance parameter for the relevant fabrication tolerance quality class.

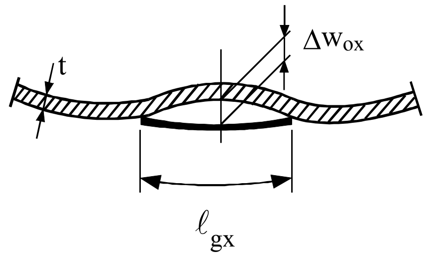
NOTE 1: Values for the dimple tolerance parameter $U_{0,max}$ may be obtained from the National Annex. The recommended values are given in Table 8.4.

Table 8.4: Values for dimple tolerance parameter $U_{0,max}$

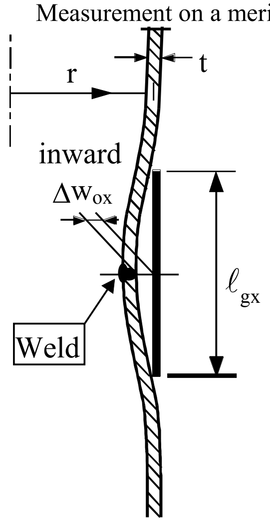
Fabrication tolerance quality class	Description	Value of $U_{0,max}$
Class A	Excellent	0,006
Class B	High	0,010
Class C	Normal	0,016



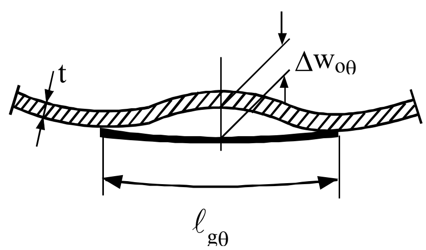
a) Measurement on a meridian



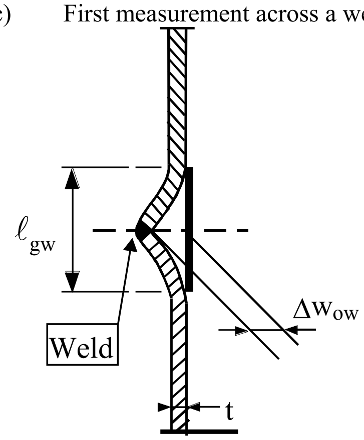
b) First measurement on a circumferential circle



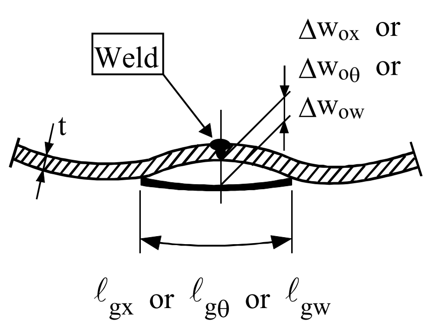
c) First measurement across a weld



d) Second measurement on circumferential circle



e) Second measurement across a weld with special gauge



f) Measurements on circumferential circle across weld

Figure 8.4: Measurement of depths Δw_0 of initial dimples

8.4.5 Interface flatness tolerance

(1) Where another structure continuously supports a shell (such as a foundation), its deviation from flatness at the interface should not include a local slope in the circumferential direction greater than β_θ.

NOTE: The National Annex may choose the value of β_θ. The value $\beta_\theta = 0{,}1\% = 0{,}001$ radians is recommended.

8.5 Stress design

8.5.1 Design values of stresses

(1) The design values of stresses $\sigma_{x,Ed}$, $\sigma_{\theta,Ed}$ and $\tau_{x\theta,Ed}$ should be taken as the key values of compressive and shear membrane stresses obtained from linear shell analysis (LA). Under purely axisymmetric conditions of loading and support, and in other simple load cases, membrane theory may generally be used.

(2) The key values of membrane stresses should be taken as the maximum value of each stress at that axial coordinate in the structure, unless specific provisions are given in Annex D of this Standard or the relevant application part of EN 1993.

NOTE: In some cases (e.g. stepped walls under circumferential compression, see Annex D.2.3), the key values of membrane stresses are fictitious and larger than the real maximum values.

(3) For basic loading cases the membrane stresses may be taken from Annex A or Annex C.

8.5.2 Design resistance (buckling strength)

(1) The buckling resistance should be represented by the buckling stresses as defined in 1.3.5. The design buckling stresses should be obtained from:

$$\sigma_{x,Rd} = \sigma_{x,Rk}/\gamma_{M1}, \quad \sigma_{\theta,Rd} = \sigma_{\theta,Rk}/\gamma_{M1}, \quad \tau_{x\theta,Rd} = \tau_{x\theta,Rk}/\gamma_{M1} \qquad \text{... (8.11)}$$

(2) The partial factor for resistance to buckling γ_{M1} should be taken from the relevant application standard.

NOTE: The value of the partial factor γ_{M1} may be defined in the National Annex. Where no application standard exists for the form of construction involved, or the application standard does not define the relevant values of γ_{M1}, it is recommended that the value of γ_{M1} should not be taken as smaller than $\gamma_{M1} = 1{,}1$.

(3) The characteristic buckling stresses should be obtained by multiplying the characteristic yield strength by the buckling reduction factors:

$$\sigma_{x,Rk} = \chi_x f_{y,k}, \quad \sigma_{\theta,Rk} = \chi_\theta f_{y,k}, \quad \tau_{x\theta,Rk} = \chi_t f_{y,k} / \sqrt{3} \qquad \text{... (8.12)}$$

(4) The buckling reduction factors χ_x, χ_θ and χ_t should be determined as a function of the relative slenderness of the shell $\bar{\lambda}$ from:

$$\chi = 1 \qquad \text{when} \qquad \bar{\lambda} \le \bar{\lambda}_0 \qquad \text{... (8.13)}$$

$$\chi = 1 - \beta \left(\frac{\bar{\lambda} - \bar{\lambda}_0}{\bar{\lambda}_p - \bar{\lambda}_0} \right)^{\eta} \qquad \text{when} \qquad \bar{\lambda}_0 < \bar{\lambda} < \bar{\lambda}_p \qquad \text{... (8.14)}$$

$$\chi = \frac{\alpha}{\bar{\lambda}^2} \qquad \text{when} \qquad \bar{\lambda}_p \le \bar{\lambda} \qquad \text{... (8.15)}$$

where:

α is the elastic imperfection reduction factor
β is the plastic range factor
η is the interaction exponent

$\overline{\lambda}_0$ is the squash limit relative slenderness

The values of these parameters should be taken from Annex D.

NOTE: Expression (8.15) describes the elastic buckling stress, accounting for geometric imperfections. In this case, where the behaviour is entirely elastic, the characteristic buckling stresses may alternatively be determined directly from $\sigma_{x,Rk} = \alpha_x \sigma_{x,Rcr}$, $\sigma_{\theta,Rk} = \alpha_\theta \sigma_{x,Rcr}$, and $\tau_{x\theta,Rk} = \alpha_\tau \tau_{x\theta,Rcr}$.

(5) The value of the plastic limit relative slenderness $\overline{\lambda}_p$ should be determined from:

$$\overline{\lambda}_p = \sqrt{\frac{\alpha}{1-\beta}} \qquad \text{... (8.16)}$$

(6) The relative shell slenderness parameters for different stress components should be determined from:

$$\overline{\lambda}_x = \sqrt{f_{y,k} / \sigma_{x,Rcr}}, \qquad \overline{\lambda}_\theta = \sqrt{f_{y,k} / \sigma_{\theta,Rcr}}, \qquad \overline{\lambda}_\tau = \sqrt{(f_{y,k} / \sqrt{3}) / \tau_{x\theta,Rcr}} \qquad \text{... (8.17)}$$

(7) The critical buckling stresses $\sigma_{x,Rcr}$, $\sigma_{\theta,Rcr}$ and $\tau_{x\theta,Rcr}$ should be obtained by means of the relevant expressions in Annex D.

(8) Where no appropriate expressions are given in Annex D, the critical buckling stresses may be extracted from a numerical LBA analysis of the shell under the buckling-relevant combinations of actions defined in 8.1. For the conditions that this analysis must satisfy, see 8.6.2 (5) and (6).

8.5.3 Stress limitation (buckling strength verification)

(1) Although buckling is not a purely stress-initiated failure phenomenon, the buckling limit state, within this section, should be represented by limiting the design values of membrane stresses. The influence of bending stresses on the buckling strength may be neglected provided they arise as a result of boundary compatibility effects. In the case of bending stresses from local loads or from thermal gradients, special consideration should be given.

(2) Depending on the loading and stressing situation, one or more of the following checks for the key values of single membrane stress components should be carried out:

$$\sigma_{x,Ed} \le \sigma_{x,Rd}, \qquad \sigma_{\theta,Ed} \le \sigma_{\theta,Rd}, \qquad \tau_{x\theta,Ed} \le \tau_{x\theta,Rd} \qquad \text{... (8.18)}$$

(3) If more than one of the three buckling-relevant membrane stress components are present under the actions under consideration, the following interaction check for the combined membrane stress state should be carried out:

$$\left(\frac{\sigma_{x,Ed}}{\sigma_{x,Rd}}\right)^{k_x} + \left(\frac{\sigma_{\theta,Ed}}{\sigma_{\theta,Rd}}\right)^{k_\theta} - k_i \left(\frac{\sigma_{x,Ed}}{\sigma_{x,Rd}}\right)\left(\frac{\sigma_{\theta,Ed}}{\sigma_{\theta,Rd}}\right) + \left(\frac{\tau_{x\theta,Ed}}{\tau_{x\theta,Rd}}\right)^{k_\tau} \le 1 \qquad \text{... (8.19)}$$

where $\sigma_{x,Ed}$, $\sigma_{\theta,Ed}$ and $\tau_{x\theta,Ed}$ are the interaction-relevant groups of the significant values of compressive and shear membrane stresses in the shell and the values of the buckling interaction parameters k_x, k_θ, k_τ and k_i are given in Annex D.

(4) Where $\sigma_{x,Ed}$ or $\sigma_{\theta,Ed}$ is tensile, its value should be taken as zero in expression (8.19).

NOTE: For axially compressed cylinders with internal pressure (leading to circumferential tension) special provisions are made in Annex D. The resulting value of $\sigma_{x,Rd}$ accounts for both the strengthening effect of internal pressure on the elastic buckling resistance and the weakening effect of the elastic-plastic elephant's foot phenomenon (expression D.43). If the tensile $\sigma_{\theta,Ed}$ is then taken as zero in expression (8.19), the buckling strength is accurately represented.

(5) The locations and values of each of the buckling-relevant membrane stresses to be used together in combination in expression (8.19) are defined in Annex D.

(6) Where the shell buckling condition is not included in Annex D, the buckling interaction parameters may be conservatively estimated using:

$$k_x = 1{,}0 + \chi_x^2 \quad \text{...(8.20)}$$

$$k_\theta = 1{,}0 + \chi_\theta^2 \quad \text{...(8.21)}$$

$$k_\tau = 1{,}5 + 0{,}5\,\chi_\tau^2 \quad \text{...(8.22)}$$

$$k_i = (\chi_x\,\chi_\theta)^2 \quad \text{...(8.23)}$$

NOTE: These rules may sometimes be very conservative, but they include the two limiting cases which are well established as safe for a wide range of cases:

a) in very thin shells, the interaction between σ_x and σ_θ is approximately linear; and

b) in very thick shells, the interaction becomes that of von Mises.

8.6 Design by global numerical analysis using MNA and LBA analyses

8.6.1 Design value of actions

(1) The design values of actions shall be taken as in 8.1 (1).

8.6.2 Design value of resistance

(1) The design buckling resistance shall be determined as a load factor R applied to the design values of the combination of actions for the relevant load case.

(2) The design buckling resistance R_d should be obtained from the plastic reference resistance R_{pl} and the elastic critical buckling resistance R_{cr}, combining these to find the characteristic buckling resistance R_k. The partial factor γ_{M1} should then be used to obtain the design resistance.

(3) The plastic reference resistance R_{pl} (see figure 8.5) should be obtained by materially non-linear analysis (MNA) as the plastic limit load under the applied combination of actions.

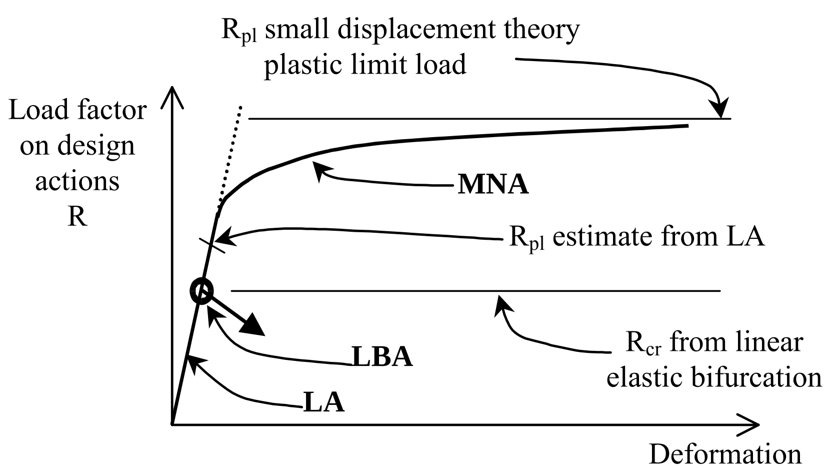


Figure 8.5: Definition of plastic reference resistance R_{pl} and critical buckling resistance R_{cr} derived from global MNA and LBA analyses

(4) Where it is not possible to undertake a materially non-linear analysis (MNA), the plastic reference resistance R_{pl} may be conservatively estimated from linear shell analysis (LA) conducted using the design values of the applied combination of actions using the following procedure. The evaluated membrane stress

resultants $n_{x,Ed}$, $n_{\theta,Ed}$ and $n_{x\theta,Ed}$ at any point in the shell should be used to find the plastic reference resistance from:

$$R_{pl} = \frac{t\ f_{y,k}}{\sqrt{n_{x,Ed}^2 - n_{x,Ed}n_{\theta,Ed} + n_{\theta,Ed}^2 + 3n_{x\theta,Ed}^2}} \qquad \text{... (8.24)}$$

The lowest value of plastic resistance so calculated should be taken as the estimate of the plastic reference resistance R_{pl}.

NOTE: A safe estimate of R_{pl} can usually be obtained by applying expression (8.24) in turn at the three points in the shell where each of the three buckling-relevant membrane stress resultants attains its highest value, and using the lowest of these three estimates as the relevant value of R_{pl}.

(5) The critical buckling resistance R_{cr} should be determined from an eigenvalue analysis (LBA) applied to the linear elastic calculated stress state in the geometrically perfect shell (LA) under the design values of the load combination. The lowest eigenvalue (bifurcation load factor) should be taken as the critical buckling resistance R_{cr}, see figure 8.5.

(6) It should be verified that the eigenvalue algorithm that is used is reliable at finding the eigenmode that leads to the lowest eigenvalue. In cases of doubt, neighbouring eigenvalues and their eigenmodes should be calculated to obtain a fuller insight into the bifurcation behaviour of the shell. The analysis should be carried out using software that has been authenticated against benchmark cases with physically similar buckling characteristics.

(7) The overall relative slenderness $\overline{\lambda}_{ov}$ for the complete shell should be determined from:

$$\overline{\lambda}_{ov} = \sqrt{R_{pl} / R_{cr}} \qquad \text{... (8.25)}$$

(8) The overall buckling reduction factor χ_{ov} should be determined as $\chi_{ov} = f(\overline{\lambda}_{ov}, \overline{\lambda}_{ov,0}, \alpha_{ov}, \beta_{ov}, \eta_{ov})$ using 8.5.2 (4), in which α_{ov} is the overall elastic imperfection factor, β_{ov} is the plastic range factor, η_{ov} is the interaction exponent and $\overline{\lambda}_{ov,0}$ is the squash limit relative slenderness.

(9) The evaluation of the factors $\overline{\lambda}_{ov,0}$, α_{ov}, β_{ov} and η_{ov} should take account of the imperfection sensitivity, geometric nonlinearity and other aspects of the particular shell buckling case. Conservative values for these parameters should be determined by comparison with known shell buckling cases (see Annex D) that have similar buckling modes, similar imperfection sensitivity, similar geometric nonlinearity, similar yielding sensitivity and similar postbuckling behaviour. The value of α_{ov} should also take account of the appropriate fabrication tolerance quality class.

NOTE: Care should be taken in choosing an appropriate value of α_{ov} when this approach is used on shell geometries and loading cases where snap-through buckling may occur. Such cases include shallow conical and spherical caps and domes under external pressure or on supports that can displace radially, and assemblies of cylindrical and conical shell segments without ring stiffeners at the meridional junctions and which are loaded meridionally.

The commonly reported elastic shell buckling loads for these special cases are normally based on geometrically nonlinear analysis applied to a perfect or imperfect geometry. By contrast, the methodology used here adopts the linear bifurcation load as the reference critical buckling resistance, and this is often much higher than the snap-through load. The design calculation must account for these two sources of reduced resistance by an appropriate choice of the overall imperfection reduction factor α_{ov}. This choice must include the effect of both the geometric nonlinearity that leads to snap-through and the additional strength reduction caused by geometric imperfections.

(10) If the provisions of (9) cannot be achieved beyond reasonable doubt, appropriate tests should be carried out (see EN1990, Annex D).

(11) If specific values of α_{ov}, β_{ov}, η_{ov} and $\overline{\lambda}_{ov,0}$ are not available according to (9) or (10), the values for an axially compressed unstiffened cylinder may be adopted (see D.1.2.2).

(12) The characteristic buckling resistance should be obtained from:

$$R_k = \chi_{ov} R_{pl} \qquad \text{... (8.26)}$$

where:

R_{pl} is the plastic reference resistance.

(13) The design buckling resistance R_d should be obtained from:

$$R_d = R_k/\gamma_{M1} \qquad \text{... (8.27)}$$

where:

γ_{M1} is the partial factor for resistance to buckling according to 8.5.2 (2).

8.6.3 Buckling strength verification

(1) It should be verified that:

$$F_d \leq R_d\ F_d \quad \text{or} \quad R_d \geq 1 \qquad \text{... (8.28)}$$

8.7 Design by global numerical GMNIA analysis

8.7.1 Design values of actions

(1) The design values of actions shall be taken as in 8.1 (1).

8.7.2 Design value of resistance

(1) The design buckling resistance shall be determined as a load factor R applied to the design values F_d of the combination of actions for the relevant load case.

(2) The characteristic buckling resistance R_k should be found from the imperfect elastic-plastic critical buckling resistance R_{GMNIA}, adjusted by the calibration factor k_{GMNIA}. The design buckling resistance R_d should then be found using the partial factor γ_{M1}.

(3) To determine the imperfect elastic-plastic critical buckling resistance R_{GMNIA}, a GMNIA analysis of the geometrically imperfect shell under the applied combination of actions should be carried out, accompanied by an eigenvalue analysis to detect possible bifurcations in the load path.

NOTE: If possible, the eigenvalue analysis should use the deformation theory of plasticity, since the flow theory of plasticity can give a considerable overestimate of the elastic-plastic buckling resistance for certain problems.

(4) An LBA analysis should first be performed on the perfect structure to determine the perfect elastic critical buckling resistance R_{LBA}. An MNA should next be performed on the perfect structure to determine the perfect plastic collapse resistance R_{MNA}. These two resistances should then be used to establish the overall relative slenderness $\overline{\lambda}_{ov}$ for the complete shell according to expression 8.25.

(5) A GMNA analysis should then be performed on the perfect structure to determine the perfect elastic-plastic critical buckling resistance R_{GMNA}. This resistance should be used later to verify that the effect of the chosen geometric imperfections has a sufficiently deleterious effect to give confidence that the lowest resistance has been obtained. The GMNA analysis should be carried out under the applied combination of actions, accompanied by an eigenvalue analysis to detect possible bifurcations in the load path.

(6) The imperfect elastic-plastic critical buckling resistance R_{GMNIA} should be found as the lowest load factor R obtained from the three following criteria C1, C2 and C3, see figure 8.6:

– Criterion C1: The maximum load factor on the load-deformation-curve (limit load);

– Criterion C2: The bifurcation load factor, where this occurs during the loading path before reaching the limit point of the load-deformation-curve;

– Criterion C3: The largest tolerable deformation, where this occurs during the loading path before reaching the bifurcation load or the limit load.

(7) The largest tolerable deformation should be assessed relative to the conditions of the individual structure. If no other value is available, the largest tolerable deformation may be deemed to have been reached when the greatest local rotation of the shell (slope of the surface relative to its original geometry) attains the value β.

NOTE: The National Annex may choose the value of β. The value $\beta = 0{,}1$ radians is recommended.

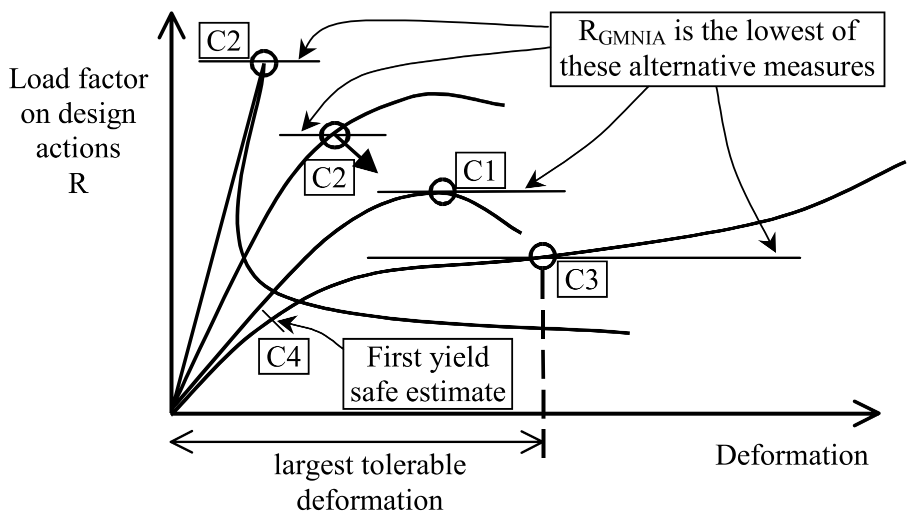


Figure 8.6: Definition of buckling resistance from global GMNIA analysis

(8) A conservative assessment of the resistance R_{GMNIA} may be obtained using a GNIA analysis of the geometrically imperfect shell under the applied combination of actions. In this case, the following criterion should be used to determine the lowest load factor R:

– Criterion C4: The load factor at which the equivalent stress at the most highly stressed point on the shell surface reaches the design value of the yield stress f_y/γ_{M0} (figure 8.6).

NOTE: It should be noted that GMNA, GMNIA and GNIA analyses must always be undertaken with regular eigenvalue checks to ensure that any possible bifurcation on the load path is detected.

(9) In formulating the GMNIA (or GNIA) analysis, appropriate allowances should be incorporated to cover the effects of imperfections that cannot be avoided in practice, including:

a) geometric imperfections, such as:

– deviations from the nominal geometric shape of the middle surface (pre-deformations, out-of-roundness);
– irregularities at and near welds (minor eccentricities, shrinkage depressions, rolling curvature errors);
– deviations from nominal thickness;
– lack of evenness of supports.

b) material imperfections, such as:

– residual stresses caused by rolling, pressing, welding, straightening etc.;
– inhomogeneities and anisotropies.

NOTE: Further possible negative influences on the critical buckling resistance R_{GMNIA}, such as ground settlements or flexibilities of connections or supports, are not classed as imperfections in the sense of these provisions.

(10) Imperfections should be allowed for in the GMNIA analysis by including appropriate additional quantities in the analytical model for the numerical computation.

(11) The imperfections should generally be introduced by means of equivalent geometric imperfections in the form of initial shape deviations perpendicular to the middle surface of the perfect shell, unless a better technique is used. The middle surface of the geometrically imperfect shell should be obtained by superposition of the equivalent geometric imperfections on the perfect shell geometry.

(12) The pattern of the equivalent geometric imperfections should be chosen in such a form that it has the most unfavourable effect on the buckling resistance R_{GMNIA} of the shell. If the most unfavourable pattern cannot be readily identified beyond reasonable doubt, the analysis should be carried out for a sufficient number of different imperfection patterns, and the worst case (lowest value of R_{GMNIA}) should be identified.

(13) The eigenmode-affine pattern should be used unless a different unfavourable pattern can be justified.

NOTE: The eigenmode affine pattern is the critical buckling mode associated with the elastic critical buckling resistance R_{cr} based on an LBA analysis of the perfect shell.

(14) The pattern of the equivalent geometric imperfections should, if practicable, reflect the constructional detailing and the boundary conditions in an unfavourable manner.

(15) Notwithstanding (13) and (14), patterns may be excluded from the investigation if they can be eliminated as unrealistic because of the method of fabrication, manufacture or erection.

(16) Modification of the adopted mode of geometric imperfections to include realistic structural details (such as axisymmetric weld depressions) should be explored.

NOTE: The National Annex may define additional requirements for the assessment of appropriate patterns of imperfections.

(17) The sign of the equivalent geometric imperfections should be chosen in such a manner that the maximum initial shape deviations are unfavourably oriented towards the centre of the shell curvature.

(18) The amplitude of the adopted equivalent geometric imperfection form should be taken as dependent on the fabrication tolerance quality class. The maximum deviation of the geometry of the equivalent imperfection from the perfect shape $\Delta w_{0,eff}$ should be the larger of $\Delta w_{0,eff,1}$ and $\Delta w_{0,eff,2}$, where:

$$\Delta w_{0,eff,1} = \ell_g \; U_{n1} \qquad \text{... (8.29)}$$

$$\Delta w_{0,eff,2} = n_i \; t \, U_{n2} \qquad \text{... (8.30)}$$

where:

ℓ_g	is	all relevant gauge lengths according to 8.4.4 (2);
t	is	the local shell wall thickness;
n_i	is	a multiplier to achieve an appropriate tolerance level;
U_{n1} and U_{n2}	are	the dimple imperfection amplitudes for the relevant fabrication tolerance quality class.

NOTE 1: The National Annex may choose the value of n_i. The value $n_i = 25$ is recommended.

NOTE 2: Values for the dimple tolerance parameter U_{n1} and U_{n2} may be obtained from the National Annex. The recommended values are given in Table 8.5.

Table 8.5: Values for initial dimple imperfection amplitudes U_{n1} and U_{n2}

Fabrication tolerance quality class	Description	Value of U_{n1}	Value of U_{n2}
Class A	Excellent	0,010	0,010
Class B	High	0,016	0,016
Class C	Normal	0,025	0,025

(19) The amplitude of the geometric imperfection in the adopted pattern of the equivalent geometric imperfection should be interpreted in a manner which is consistent with the gauge length method, set out in 8.4.4 (2), by which it is defined.

(20) Additionally, it should be verified that an analysis that adopts an imperfection whose amplitude is 10% smaller than the value $\Delta w_{0,eff}$ found in (18) does not yield a lower value for R_{GMNIA}. If a lower value is obtained, the procedure should be iterated to find the lowest value of R_{GMNIA} as the amplitude is varied.

(21) If follower load effects are possible, either they should be incorporated in the analysis, or it should be verified that their influence is negligible.

(22) For each calculated value of the buckling resistance R_{GMNIA}, the ratio of the imperfect to perfect resistance (R_{GMNIA}/R_{GMNA}) should be determined and compared with values of α found using the procedures of 8.5 and Annex D, to verify that the chosen geometric imperfection has a deleterious effect that is comparable with that obtained from a lower bound to test results.

NOTE: Where the resistance is dominated by plasticity effects, the ratio (R_{GMNIA}/R_{GMNA}) will be much larger than α, and no close comparison can be expected. However, where the resistance is controlled by buckling phenomena that are substantially elastic, the ratio (R_{GMNIA}/R_{GMNA}) should be only a little higher than the value determined by hand calculation, and the factors leading to the higher value should be considered.

(23) The reliability of the numerically determined critical buckling resistance R_{GMNIA} should be checked by one of the following alternative methods:

a) by using the same program to calculate values $R_{GMNIA,check}$ for other shell buckling cases for which characteristic buckling resistance values $R_{k,known,check}$ are known. The check cases should use basically similar imperfection assumptions and be similar in their buckling controlling parameters (such as relative shell slenderness, postbuckling behaviour, imperfection-sensitivity, geometric nonlinearity and material behaviour);

b) by comparison of calculated values ($R_{GMNIA,check}$) against test results ($R_{test,known,check}$). The check cases should satisfy the same similarity conditions given in (a).

NOTE: Other shell buckling cases for which the characteristic buckling resistance values $R_{k,known,check}$ are known may be found from the scientific literature on shell buckling. It should be noted that the hand calculations of 8.5 and Annex D are derived as general lower bounds on test results, and these sometimes lead to such low assessed values for the characteristic buckling resistance that they cannot be easily obtained numerically.

(24) Depending on the results of the reliability checks, the calibration factor k_{GMNIA} should be evaluated, as appropriate, from:

$$k_{GMNIA} = \frac{R_{k,known,check}}{R_{GMNIA,check}} \quad \text{or} \quad k_{GMNIA} = \frac{R_{test,known,check}}{R_{GMNIA,check}} \qquad \text{... (8.31)}$$

where:

$R_{k,known,check}$ is the known characteristic value;
$R_{test,known,check}$ is the known test result;
$R_{GMNIA,check}$ is the calculation outcome for the check buckling case or the test buckling case, as appropriate.

(25) Where test results are used to determine k_{GMNIA}, and the calculated value of k_{GMNIA} exceeds 1,0, the adopted value should be $k_{GMNIA} = 1,0$.

(26) Where a known characteristic value based on existing established theory is used to determine k_{GMNIA}, and the calculated value of k_{GMNIA} lies outside the range $0{,}8 < k_{GMNIA}$ 1,2, this procedure should not be used. The GMNIA result should be deemed invalid, and further calculations undertaken to establish the causes of the discrepancy.

(27) The characteristic buckling resistance should be obtained from:

$$R_k = k_{GMNIA} \; R_{GMNIA} \qquad \text{... (8.32)}$$

where:

R_{GMNIA} is the calculated imperfect elastic-plastic critical buckling resistance;
k_{GMNIA} is the calibration factor.

8.7.3 Buckling strength verification

(1) The design buckling resistance R_d should be obtained from:

$$R_d = R_k / \gamma_{M1} \qquad \text{... (8.33)}$$

where:

γ_{M1} is the partial factor for resistance to buckling according to 8.5.2 (2).

(2) It should be verified that:

$$F_d \leq R_d \; F_d \qquad \text{or} \qquad R_d \geq 1 \qquad \text{... (8.34)}$$

9 Fatigue limit state (LS4)

9.1 Design values of actions

(1) The design values of the actions for each load case shall be taken as the varying parts of the total action representing the anticipated action spectrum throughout the design life of the structure.

(2) The relevant action spectra should be obtained from EN 1991 in accordance with the definitions given in the appropriate application parts of EN 1993.

9.2 Stress design

9.2.1 General

(1) The fatigue assessment presented in EN 1993-1-9 should be used, except as provided here.

(2) The partial factor for resistance to fatigue γ_{Mf} should be taken from the relevant application standard.

NOTE: The value of the partial factor γ_{Mf} may be defined in the National Annex. Where no application standard exists for the form of construction involved, or the application standard does not define the relevant values of γ_{Mf}, the value of γ_{Mf} should be taken from EN 1993-1-9. It is recommended that the value of γ_{Mf} should not be taken as smaller than $\gamma_{Mf} = 1,1$.

9.2.2 Design values of stress range

(1) Stresses shall be determined by a linear elastic analysis of the structure subject to the design values of the fatigue actions.

(2) In each verification of the limit state, the design value of the fatigue stress should be taken as the larger stress range $\Delta\sigma$ of the values on the two surfaces of the shell, and based on the sum of the primary and the secondary stresses.

(3) Depending upon the fatigue assessment carried out according to EN 1993-1-9, either nominal stress ranges or geometric stress ranges should be evaluated.

(4) Nominal stress ranges may be used if 9.2.3 (2) is adopted.

(5) Geometric stress ranges should be used for construction details that differ from those of 9.2.3 (2).

(6) The geometric stress range takes into account only the overall geometry of the joint, excluding local stresses due to the weld geometry and internal weld effects. It may be determined by use of geometrical stress concentration factors given by expressions.

(7) Stresses used for the fatigue design of construction details with linear geometric orientation should be resolved into components transverse to and parallel to the axis of the detail.

9.2.3 Design values of resistance (fatigue strength)

(1) The design values of resistance obtained from the following may be applied to structural steels in the temperature range up to 150° C.

(2) The fatigue resistance of construction details commonly found in shell structures shall be obtained from EN 1993-3-2 in classes in terms of the stress range $\Delta\sigma_R$, in which the values are additionally classified according to the quality of the welds.

(3) The fatigue resistance of the detail classes should be obtained from EN 1993-1-9.

9.2.4 Stress range limitation

(1) In every verification of this limit state, the design stress range should satisfy the condition:

$$\gamma_{Ff}\ \Delta\sigma_E \ \leq\ \Delta\sigma_R\ /\ \gamma_{Mf} \qquad \text{... (9.1)}$$

where:

γ_{Ff} is the partial factor for the fatigue loading

γ_{Mf} is the partial factor for the fatigue resistance

$\Delta\sigma_E$ is the equivalent constant amplitude stress range of the design stress spectrum

$\Delta\sigma_R$ is the fatigue strength stress range for the relevant detail category and the number of cycles of the stress spectrum

(2) As an alternative to (1), a cumulative damage assessment may be made using the Palmgren-Miner rule:

$$D_d \leq 1 \qquad \text{... (9.2)}$$

in which:

$$D_d = \Sigma\ n_i\ /\ N_i \qquad \text{... (9.3)}$$

where:

n_i is the number of cycles of the stress range $\Delta\sigma_i$

N_i is the number of cycles of the stress range $\gamma_{Ff}\ \gamma_{Mf}\ \Delta\sigma_i$ to cause failure for the relevant detail category

(3) In the case of combination of normal and shear stress ranges the combined effects should be considered in accordance with EN1993-1-9.

9.3 Design by global numerical LA or GNA analysis

(1) The fatigue design on the basis of an elastic analysis (LA or GNA analysis) should follow the provisions given in 9.2 for stress design. However, the stress ranges due to the fatigue loading should be determined by means of a shell bending analysis, including the geometric discontinuities of joints in constructional details.

(2) If a three dimensional finite element analysis is used, the notch effects due to the local weld geometry should be eliminated.

ANNEX A (normative)
Membrane theory stresses in shells

A.1 General

A.1.1 Action effects and resistances

The action effects or resistances calculated using the expressions in this annex may be assumed to provide characteristic values of the action effect or resistance when characteristic values of the actions, geometric parameters and material properties are adopted.

A.1.2 Notation

The notation used in this annex for the geometrical dimensions, stresses and loads follows 1.4, and in addition, the following notation is used.

Roman upper case letters

- F_x axial load applied to the cylinder
- M global bending moment applied to the complete cylinder (not to be confused with the moment per unit width in the shell wall m)
- M_t global torque applied to the complete cylinder
- V global transverse shear applied to the complete cylinder

Roman lower case letters

- g unit weight of the material of the shell
- p_n distributed normal pressure
- p_x distributed axial traction on cylinder wall

Greek lower case letters

- ϕ meridional slope angle
- σ_x axial or meridional membrane stress (= n_x/t)
- σ_θ circumferential membrane stress (= n_θ/t)
- τ membrane shear stress (= $n_{x\theta}/t$)

A.1.3 Boundary conditions

(1) The boundary condition notations should be taken as detailed in 2.3.

(2) For these expressions to be strictly valid, the boundary conditions for cylinders should be taken as radially free at both ends, axially supported at one end, and rotationally restrained at both ends.

(3) For truncated cones, the boundary conditions should be understood to include components of loading transverse to the shell wall, so that the combined stress resultant introduced into the shell is solely in the direction of the shell meridian.

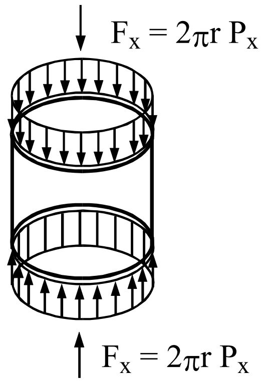
(4) The boundary conditions for these expressions to be strictly valid for cones should be taken as radially free at both ends, meridionally supported at one end, and rotationally restrained at both ends.

A.1.4 Sign convention

(1) The sign convention for stresses σ should be taken everywhere as tension positive, though some of the figures illustrate cases in which the external load is applied in the opposite sense.

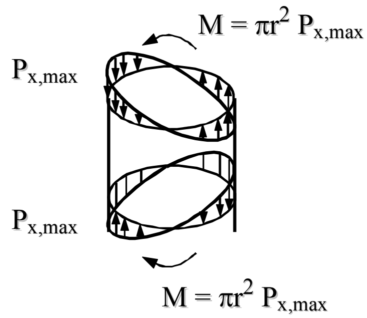
A.2 Unstiffened Cylindrical Shells

A.2.1 Uniform axial load



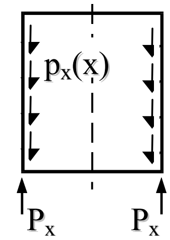
$$\sigma_x = -\frac{F_x}{2\pi r t}$$

A.2.2 Axial load from bending



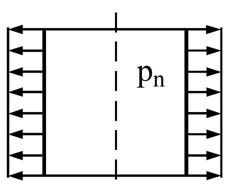
$$\sigma_x = \pm\frac{M}{\pi r^2 t}$$

A.2.3 Friction load



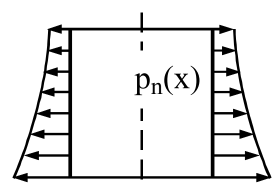
$$\sigma_x = -\frac{1}{t}\int_0^{\ell} p_x \, dx$$

A.2.4 Uniform internal pressure



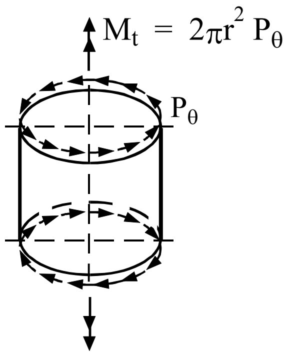
$$\sigma_\theta = p_n \frac{r}{t}$$

A.2.5 Variable internal pressure



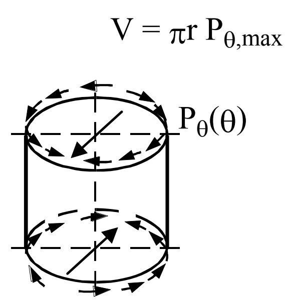
$$\sigma_\theta(x) = p_n(x) \frac{r}{t}$$

A.2.6 Uniform shear from torsion



$$\tau = \frac{M_t}{2\pi r^2 t}$$

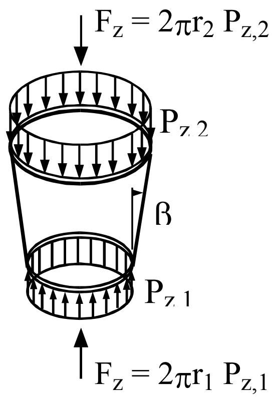
A.2.7 Sinusoidal shear from transverse force



$$\tau_{max} = \pm\frac{V}{\pi r t}$$

A.3 Unstiffened Conical Shells

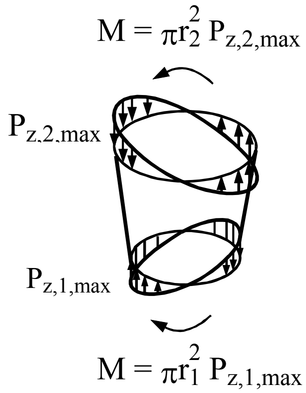
A.3.1 Uniform axial load



$$\sigma_x = -\frac{F_z}{2\pi r\, t \cos\beta}$$

$$\sigma_\theta = 0$$

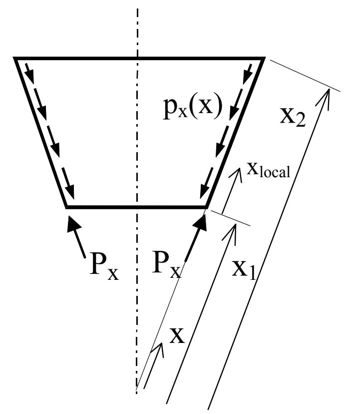
A.3.2 Axial load from bending



$$\sigma_{x,max} = \pm \frac{M}{\pi r^2\, t \cos\beta}$$

$$\sigma_\theta = 0$$

A.3.3 Friction load

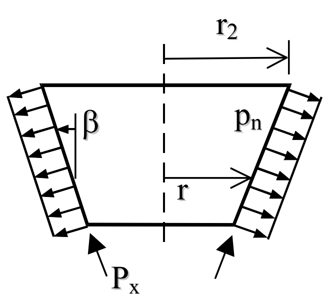


$$x_1 = \frac{r_1}{\sin\beta} \qquad x_2 = \frac{r_2}{\sin\beta}$$

$$\sigma_{x1} = -\frac{1}{x_1 t} \int_{x_1}^{x_2} p_x\, x \;dx$$

$$\sigma_\theta = 0$$

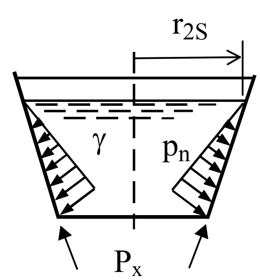
A.3.4 Uniform internal pressure



$$\sigma_x = -p_n \frac{r}{2t\cos\beta}\left[\left(\frac{r_2}{r}\right)^2 - 1\right]$$

$$\sigma_\theta = p_n \frac{r}{t\cos\beta}$$

A.3.5 Linearly varying internal pressure

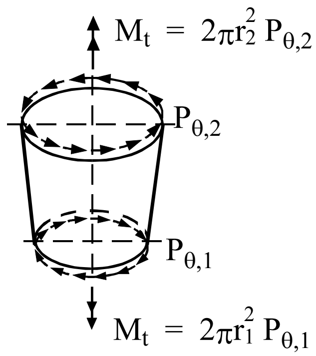


r_{2S} is the radius at the fluid surface

$$\sigma_x = -\frac{\gamma\, r}{t \sin\beta}\left\{\frac{r_{2S}}{6}\left[\left(\frac{r_{2S}}{r}\right)^2 - 3\right] + \frac{r}{3}\right\}$$

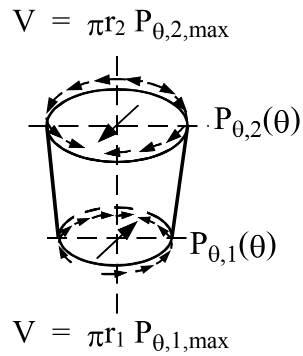
$$\sigma_\theta = +\frac{\gamma\, r}{t \sin\beta}\,(r_{2S} - r)$$

A.3.6 Uniform shear from torsion



$$\tau = \frac{M_t}{2\pi r^2 t}$$

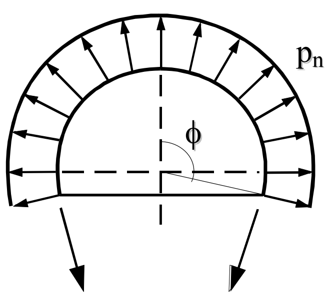
A.3.7 Sinusoidal shear from transverse force



$$\tau_{max} = \pm \frac{V}{\pi r t}$$

A.4 Unstiffened Spherical Shells

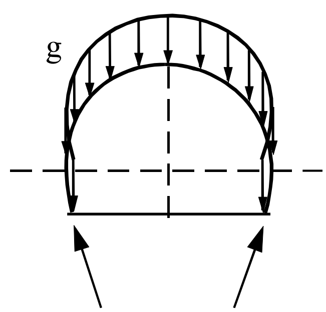
A.4.1 Uniform internal pressure



$$\sigma_x = \frac{p_n r}{2t}$$

$$\sigma_\theta = \frac{p_n r}{2t}$$

A.4.2 Uniform self-weight load



$$\sigma_x = -\frac{gr}{t}\left(\frac{1}{1+\cos\phi}\right)$$

$$\sigma_\theta = -\frac{gr}{t}\left(\cos\phi - \frac{1}{1+\cos\phi}\right)$$

ANNEX B (normative)
Additional expressions for plastic collapse resistances

B.1 General

B.1.1 Resistances

The resistances calculated using the expressions in this annex may be assumed to provide characteristic values of the resistance when characteristic values of the geometric parameters and material properties are adopted.

B.1.2 Notation

The notation used in this annex for the geometrical dimensions, stresses and loads follows 1.4, and in addition, the following notation is used.

Roman upper case letters

A_r	cross-sectional area of a ring
P_R	characteristic value of small deflection theory plastic mechanism resistance

Roman lower case letters

b	thickness of a ring
ℓ	effective length of shell which acts with a ring
r	radius of the cylinder
s_e	dimensionless von Mises equivalent stress parameter
s_m	dimensionless combined stress parameter
s_x	dimensionless axial stress parameter
s_θ	dimensionless circumferential stress parameter

Subscripts

r	relating to a ring
R	resistance

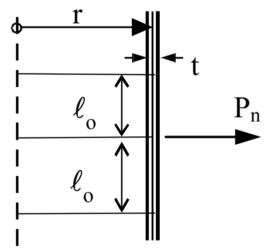
B.1.3 Boundary conditions

(1) The boundary condition notations should be taken as detailed in 5.2.2.

(2) The term “clamped” should be taken to refer to BC1r and the term “pinned” to refer to BC2f.

B.2 Unstiffened cylindrical shells

B.2.1 Cylinder: Radial line load



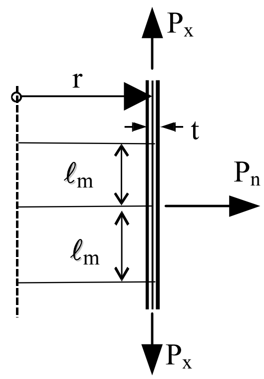
Reference quantities:

$$\ell_o = 0{,}975\sqrt{rt}$$

The plastic resistance P_{nR} (force per unit circumference) is given by:

$$\frac{P_{nR}}{2\ell_o} = f_y \frac{t}{r}$$

B.2.2 Cylinder: Radial line load and axial load



Reference quantities:

$$s_x = \frac{P_x}{f_y\, t} \qquad \ell_o = 0{,}975\sqrt{rt}$$

Range of applicability:

$$-1 \le s_x \le +1$$

Dependent parameters:

If $P_n > 0$ (outward) then: $A = +s_x - 1{,}50$

If $P_n < 0$ (inward) then: $A = -s_x - 1{,}50$

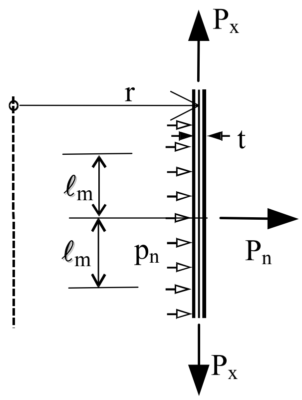
$$s_m = A + \sqrt{A^2 + 4(1 - s_x^2)}$$

If $s_x \neq 0$ then: $\ell_m = s_m\, \ell_o$

The plastic resistance P_{nR} (force per unit circumference) is given by:

$$\frac{P_{nR}}{2\ell_m} = f_y \frac{t}{r}$$

B.2.3 Cylinder: Radial line load, constant internal pressure and axial load



Reference quantities:

$$s_x = \frac{P_x}{f_y\, t} \qquad s_\theta = \frac{p_n\, r}{f_y\, t}$$

$$\ell_o = 0{,}975\sqrt{rt} \qquad s_e = \sqrt{s_\theta^2 + s_x^2 - s_x s_\theta}$$

Range of applicability:

$$-1 \le s_x \le +1 \qquad -1 \le s_\theta \le +1$$

Dependent parameters:

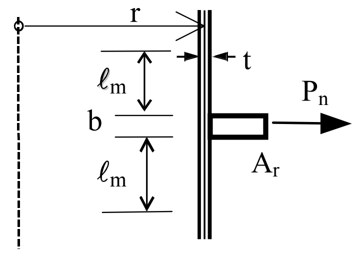
Outward directed ring load $P_n > 0$		Inward directed ring load $P_n < 0$	
Condition	Expressions	Condition	Expressions
$s_e < 1{,}00$ and $s_\theta \le 0{,}975$	$A = +s_x - 2s_\theta - 1{,}50$ $s_m = A + \sqrt{A^2 + 4(1 - s_e^2)}$ $\ell_m = \ell_o\left(\frac{s_m}{1 - s_\theta}\right)$	$s_e < 1{,}00$ and $s_\theta \ge -0{,}975$	$A = -s_x + 2s_\theta - 1{,}50$ $s_m = A + \sqrt{A^2 + 4(1 - s_e^2)}$ $\ell_m = \ell_o\left(\frac{s_m}{1 + s_\theta}\right)$
$s_e = 1{,}00$ or $s_\theta > 0{,}975$	$\ell_m = 0{,}0$	$s_e = 1{,}00$ or $s_\theta < -0{,}975$	$\ell_m = 0{,}0$

The plastic resistance is given by (P_n and p_n always positive outwards):

$$\frac{P_{nR}}{2\ell_m} + p_n = f_y \frac{t}{r}$$

B.3 Ring stiffened cylindrical shells

B.3.1 Ring stiffened Cylinder: Radial line load

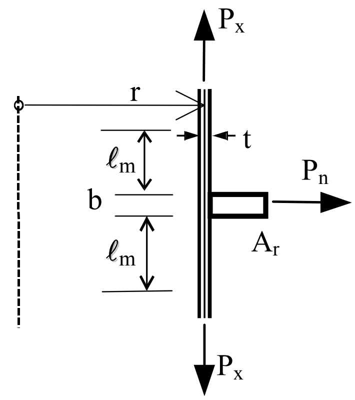


The plastic resistance P_{nR} (force per unit circumference) is given by:

$$P_{nR} = f_y \left(\frac{A_r + (b + 2\ell_m)t}{r} \right)$$

$$\ell_m = \ell_o = 0{,}975\sqrt{rt}$$

B.3.2 Ring stiffened Cylinder: Radial line load and axial load



Reference quantities:

$$s_x = \frac{P_x}{f_y\, t} \qquad \ell_o = 0{,}975\sqrt{rt}$$

Range of applicability:

$$-1 \le s_x \le +1$$

Dependent parameters:

If $P_n > 0$ then: $A = +s_x - 1{,}50$

If $P_n < 0$ then: $A = -s_x - 1{,}50$

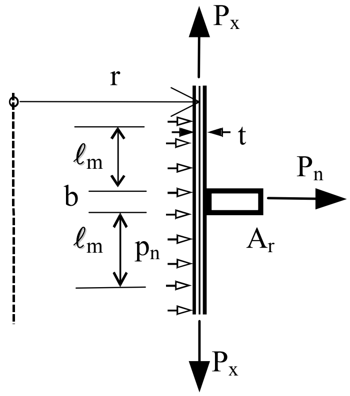
$$s_m = A + \sqrt{A^2 + 4(1 - s_x^2)}$$

If $s_x \neq 0$ then: $\ell_m = s_m\, \ell_o$

The plastic resistance P_{nR} (force per unit circumference) is given by:

$$P_{nR} = f_y \left(\frac{A_r + (b + 2\ell_m)t}{r} \right)$$

B.3.3 Ring stiffened cylinder: Radial line load, constant internal pressure and axial load



Reference quantities:

$$s_x = \frac{P_x}{f_y t} \qquad s_\theta = \frac{p_n r}{f_y t}$$

$$\ell_o = 0{,}975\sqrt{rt} \qquad s_e = \sqrt{s_\theta^2 + s_x^2 - s_x s_\theta}$$

Range of applicability:

$$-1 \le s_x \le +1 \qquad -1 \le s_\theta \le +1$$

Dependent parameters:

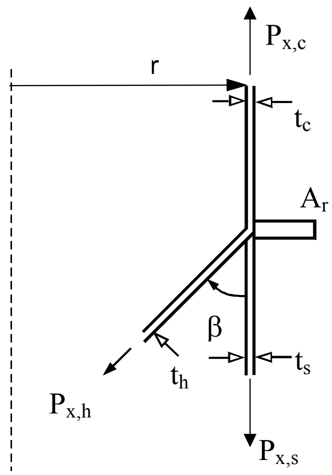
Outward directed ring load $P_n > 0$		Inward directed ring load $P_n < 0$	
Condition	Expressions	Condition	Expressions
$s_e < 1{,}00$ and $s_\theta \le 0{,}975$	$A = +s_x - 2s_\theta - 1{,}50$ $s_m = A + \sqrt{A^2 + 4(1 - s_e^2)}$ $\ell_m = \ell_o\left(\frac{s_m}{1 - s_\theta}\right)$	$s_e < 1{,}00$ and $s_\theta \ge -0{,}975$	$A = -s_x + 2s_\theta - 1{,}50$ $s_m = A + \sqrt{A^2 + 4(1 - s_e^2)}$ $\ell_m = \ell_o\left(\frac{s_m}{1 + s_\theta}\right)$
$s_e = 1{,}00$ or $s_\theta > 0{,}975$	$\ell_m = 0{,}0$	$s_e = 1{,}00$ or $s_\theta < -0{,}975$	$\ell_m = 0{,}0$

The plastic resistance is given by (P_n and p_n always positive outwards):

$$P_{nR} + p_n(b + 2\ell_m) = f_y\left(\frac{A_r + (b + 2\ell_m)t}{r}\right)$$

B.4 Junctions between shells

B.4.1 Junction under meridional loading only (simplified)



Range of applicability:

$t_c^2 \le t_s^2 + t_h^2$ $|P_{xs}| << t_s f_y$, $|P_{xh}| << t_h f_y$, and $|P_{xc}| << t_c f_y$.

Dependent parameters:

$\eta = \sqrt{\dfrac{t_c^2}{t_s^2 + t_h^2}}$ $\psi_s = \psi_h = 0{,}7 + 0{,}6\eta^2 - 0{,}3\eta^3$

For the cylinder $\ell_{oc} = 0{,}975 \sqrt{rt_c}$

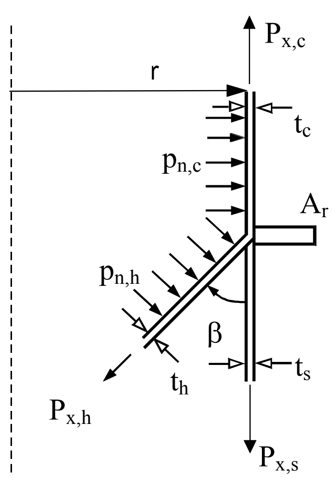
For the skirt $\ell_{os} = 0{,}975\ \psi_s \sqrt{rt_s}$

For the conical segment $\ell_{oh} = 0{,}975\ \psi_h \sqrt{\dfrac{rt_h}{\cos\beta}}$

The plastic resistance is given by:

$$P_{xhR}\ r \sin\beta = f_y\,(A_r + \ell_{oc}t_c + \ell_{os}t_s + \ell_{oh}t_h)$$

B.4.2 Junction under internal pressure and axial loading



Reference quantities:

$$s_{xc} = \frac{P_{xc}}{f_y\, t_c} \qquad s_{xs} = \frac{P_{xs}}{f_y\, t_s} \qquad s_{xh} = \frac{P_{xh}}{f_y\, t_h}$$

$$s_{\theta c} = \frac{p_{nc}}{f_y}\frac{r}{t_c} \qquad s_{\theta s} = 0 \qquad s_{\theta h} = \frac{p_{nh}}{f_y}\frac{r}{t_h \cos\beta}$$

for i = c, s, h in turn $\quad s_{ei} = \sqrt{s_{\theta i}^2 + s_{xi}^2 - s_{xi}s_{\theta i}}$

Range of applicability:

$$-1 \le s_{xi} \le +1 \qquad -1 \le s_{\theta i} \le +1$$

Equivalent thickness evaluation:

Lower plate group thicker $t_c^2 \le t_s^2 + t_h^2$	Upper plate group thicker $t_c^2 > t_s^2 + t_h^2$
$\eta = \sqrt{\frac{t_c^2}{t_s^2 + t_h^2}}$ $\psi_c = 1{,}0$ $\psi_s = \psi_h = 0{,}7 + 0{,}6\eta^2 - 0{,}3\eta^3$	$\eta = \sqrt{\frac{t_s^2 + t_h^2}{t_c^2}}$ $\psi_c = 0{,}7 + 0{,}6\eta^2 - 0{,}3\eta^3$ $\psi_s = \psi_h = 1{,}0$

Dependent parameters:

For the cylindrical segments $\quad \ell_{oi} = 0{,}975\, \psi_i \sqrt{rt_i}$

For the conical segment $\quad \ell_{oh} = 0{,}975\, \psi_h \sqrt{\frac{rt_i}{\cos\beta}}$

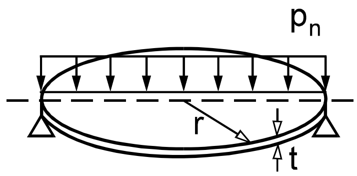
For each shell segment i separately	
Condition	Expressions
$s_{ei} < 1{,}00$ and $s_{\theta i} \ge -0{,}975$	$A_i = -s_{xi} + 2s_{\theta i} - 1{,}50$ $s_{mi} = A_i + \sqrt{A_i^2 + 4(1 - s_{ei}^2)}$ $\ell_{mi} = \ell_{oi}\left(\frac{s_{mi}}{1 + s_{\theta i}}\right)$
$s_{ei} = 1{,}00$	$\ell_{mi} = 0{,}0$
$s_{\theta i} < -0{,}975$	$\ell_{mi} = 0{,}0$

Plastic resistance is given by:

$$P_{xhR}\, r \sin\beta = f_y\,(A_r + \ell_{mc}t_c + \ell_{ms}t_s + \ell_{mh}t_h) + r\,(p_{nc}\ell_{mc} + p_{nh}\ell_{mh}\cos\beta)$$

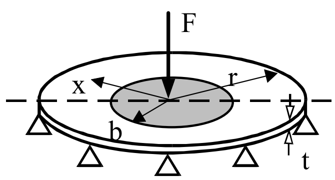
B.5 Circular plates with axisymmetric boundary conditions

B.5.1 Uniform load, simply supported boundary



$$p_{n,R} = 1{,}625 \left(\frac{t}{r}\right)^2 \; f_y$$

B.5.2 Local distributed load, simply supported boundary



Uniform pressure p_n on circular patch of radius b

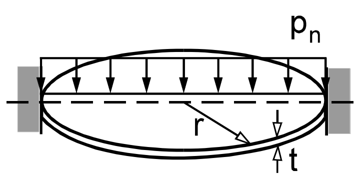
$F = p_n \, \pi \, b^2$

$$F_R = K \frac{\pi}{2} \; t^2 f_y$$

with $K = \min$

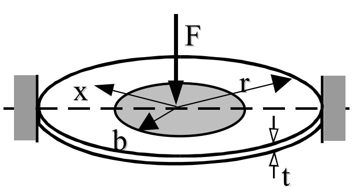
$$\begin{bmatrix} 1{,}0 + 1{,}10 \dfrac{b}{r} + 1{,}15 \left(\dfrac{b}{r}\right)^4 \\ \dfrac{1}{\sqrt{3}} \dfrac{b}{t} \end{bmatrix}$$

B.5.3 Uniform load, clamped boundary



$$p_{n,R} = 3{,}125 \left(\frac{t}{r}\right)^2 \; f_y$$

B.5.4 Local distributed load, clamped boundary



Uniform pressure p_n on circular patch of radius b

$F = p_n \, \pi \, b^2$

$$F_R = K \frac{\pi}{2} \; t^2 f_y$$

with $K = \min$

$$\begin{bmatrix} 1{,}40 + 2{,}85 \dfrac{b}{r} + 2{,}0 \left(\dfrac{b}{r}\right)^4 \\ \dfrac{1}{\sqrt{3}} \dfrac{b}{t} \end{bmatrix}$$

ANNEX C (normative)
Expressions for linear elastic membrane and bending stresses

C.1 General

C.1.1 Action effects

The action effects calculated using the expressions in this annex may be assumed to provide characteristic values of the action effect when characteristic values of the actions, geometric parameters and material properties are adopted.

C.1.2 Notation

The notation used in this annex for the geometrical dimensions, stresses and loads follows 1.4, and in addition, the following notation is used.

Roman characters

b	radius at which local load on plate terminates
r	outside radius of circular plate
x	axial coordinate on cylinder or radial coordinate on circular plate

Greek symbols

$\sigma_{eq,m}$	von Mises equivalent stress associated with only membrane stress components
$\sigma_{eq,s}$	von Mises equivalent stress derived from surface stresses
σ_{MT}	reference stress derived from membrane theory
σ_{bx}	meridional bending stress
$\sigma_{b\theta}$	circumferential bending stress
σ_{sx}	meridional surface stress
$\sigma_{s\theta}$	circumferential surface stress
τ_{xn}	transverse shear stress associated with meridional bending

Subscripts

n	normal
r	relating to a ring
y	first yield value

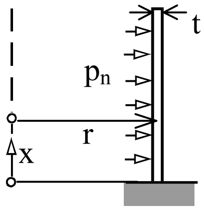
C.1.3 Boundary conditions

(1) The boundary condition notations should be taken as detailed in 5.2.2.

(2) The term “clamped” should be taken to refer to BC1r and the term “pinned” to refer to BC2f.

C.2 Clamped base unstiffened cylindrical shells

C.2.1 Cylinder, clamped: uniform internal pressure

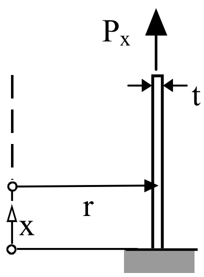


$$\sigma_{MT\theta} = p_n \frac{r}{t}$$

BC1r

Maximum σ_{sx}	Maximum $\sigma_{s\theta}$	Maximum τ_{xn}	Maximum $\sigma_{eq,s}$	Maximum $\sigma_{eq,m}$
$\pm 1{,}816\ \sigma_{MT\theta}$	$+1{,}080\ \sigma_{MT\theta}$	$1{,}169\sqrt{t/r}\ \sigma_{MT\theta}$	$1{,}614\ \sigma_{MT\theta}$	$1{,}043\ \sigma_{MT\theta}$

C.2.2 Cylinder, clamped: axial loading

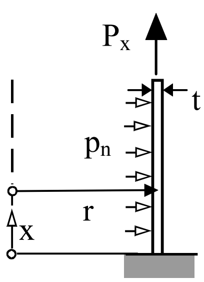


$$\sigma_{MTx} = \frac{P_x}{t}$$

BC1r

Maximum σ_{sx}	Maximum $\sigma_{s\theta}$	Maximum τ_{xn}	Maximum $\sigma_{eq,s}$	Maximum $\sigma_{eq,m}$
$1{,}545\ \sigma_{MTx}$	$+0{,}455\ \sigma_{MTx}$	$0{,}351\sqrt{t/r}\ \sigma_{MTx}$	$1{,}373\ \sigma_{MTx}$	$1{,}000\ \sigma_{MTx}$

C.2.3 Cylinder, clamped: uniform internal pressure with axial loading



$$\sigma_{MT\theta} = p_n \frac{r}{t}$$

$$\sigma_{MTx} = \frac{P_x}{t}$$

BC1r

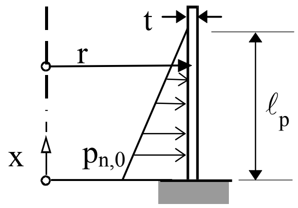
$$\text{Maximum } \sigma_{eq,m} = \sigma_{MT\theta} \sqrt{1 - \left(\frac{\sigma_{MTx}}{\sigma_{MT\theta}}\right) + \left(\frac{\sigma_{MTx}}{\sigma_{MT\theta}}\right)^2}$$

$$\text{Maximum } \sigma_{eq,m} = k\, \sigma_{MT\theta}$$

$\left(\frac{\sigma_{MTx}}{\sigma_{MT\theta}}\right)$	−2,0	0	0	2,0
	Outer surface controls		Inner surface controls	
k	4,360	1,614	1,614	2,423

Linear interpolation may be used between values where the same surface controls

C.2.4 Cylinder, clamped: hydrostatic internal pressure



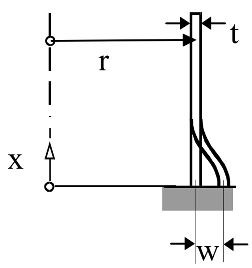
$$\sigma_{MT\theta} = p_{n0}\frac{r}{t}$$

BC1r

Maximum σ_{sx}	Maximum $\sigma_{s\theta}$	Maximum τ_{xn}	Maximum $\sigma_{eq,s}$	Maximum $\sigma_{eq,m}$
$k_x\,\sigma_{MT\theta}$	$k_\theta\,\sigma_{MT\theta}$	$k_\tau\sqrt{t/r}\;\sigma_{MT\theta}$	$k_{eq,s}\,\sigma_{MT\theta}$	$k_{eq,m}\,\sigma_{MT\theta}$

$\left(\frac{\sqrt{rt}}{\ell_p}\right)$	k_x	k_θ	k_τ	$k_{eq,s}$	$k_{eq,m}$
0	1,816	1,080	1,169	1,614	1,043
0,2	1,533	0,733	1,076	1,363	0,647

C.2.5 Cylinder, clamped: radial outward displacement

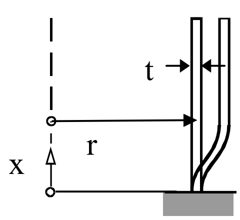


$$\sigma_{MT\theta} = \frac{wE}{r}$$

BC1r

Maximum σ_{sx}	Maximum $\sigma_{s\theta}$	Maximum τ_{xn}	Maximum $\sigma_{eq,s}$	Maximum $\sigma_{eq,m}$
1,816 $\sigma_{MT\theta}$	1,545 $\sigma_{MT\theta}$	1,169$\sqrt{t/r}\;\sigma_{MT\theta}$	2,081 $\sigma_{MT\theta}$	1,000 $\sigma_{MT\theta}$

C.2.6 Cylinder, clamped: uniform temperature rise



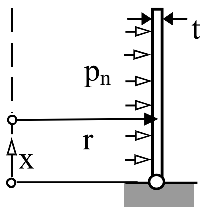
$$\sigma_{MT\theta} = \alpha\,E\,T$$

BC1r

Maximum σ_{sx}	Maximum $\sigma_{s\theta}$	Maximum τ_{xn}	Maximum $\sigma_{eq,s}$	Maximum $\sigma_{eq,m}$
1,816 $\sigma_{MT\theta}$	1,545 $\sigma_{MT\theta}$	1,169$\sqrt{t/r}\;\sigma_{MT\theta}$	2,081 $\sigma_{MT\theta}$	1,000 $\sigma_{MT\theta}$

C.3 Pinned base unstiffened cylindrical shells

C.3.1 Cylinder, pinned: uniform internal pressure

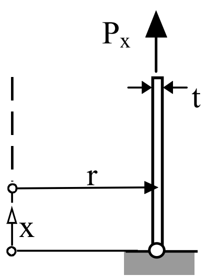


$$\sigma_{MT\theta} = p_n \frac{r}{t}$$

BC1f

Maximum σ_{sx}	Maximum $\sigma_{s\theta}$	Maximum τ_{xn}	Maximum $\sigma_{eq,s}$	Maximum $\sigma_{eq,m}$
$\pm 0{,}585\ \sigma_{MT\theta}$	$+1{,}125\ \sigma_{MT\theta}$	$0{,}583\sqrt{t/r}\ \sigma_{MT\theta}$	$1{,}126\ \sigma_{MT\theta}$	$1{,}067\ \sigma_{MT\theta}$

C.3.2 Cylinder, pinned: axial loading

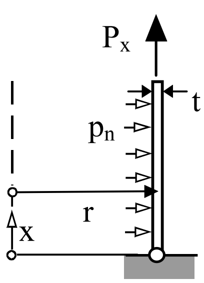


$$\sigma_{MTx} = \frac{P_x}{t}$$

BC1f

Maximum σ_{sx}	Maximum $\sigma_{s\theta}$	Maximum τ_{xn}	Maximum $\sigma_{eq,s}$	Maximum $\sigma_{eq,m}$
$+1{,}176\ \sigma_{MTx}$	$+0{,}300\ \sigma_{MTx}$	$0{,}175\sqrt{t/r}\ \sigma_{MTx}$	$1{,}118\ \sigma_{MTx}$	$1{,}010\ \sigma_{MTx}$

C.3.3 Cylinder, pinned: uniform internal pressure with axial loading



$$\sigma_{MT\theta} = p_n \frac{r}{t}$$

$$\sigma_{MTx} = \frac{P_x}{t}$$

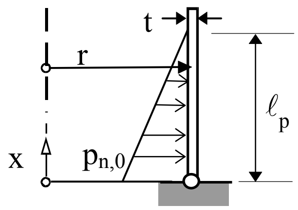
BC1f

$$\text{Maximum } \sigma_{eq,m} = \sigma_{MT\theta}\sqrt{1 - \left(\frac{\sigma_{MTx}}{\sigma_{MT\theta}}\right) + \left(\frac{\sigma_{MTx}}{\sigma_{MT\theta}}\right)^2}$$

$$\text{Maximum } \sigma_{eq,s} = k\,\sigma_{MT\theta}$$

$\left(\frac{\sigma_{MTx}}{\sigma_{MT\theta}}\right)$	−2,0	−1,0	−0,5	0,0	0,25	0,50	1,00	2,0
k	3,146	3,075	1,568	1,126	0,971	0,991	1,240	1,943

C.3.4 Cylinder, pinned: hydrostatic internal pressure



$$\sigma_{MT\theta} = p_{n0}\frac{r}{t}$$

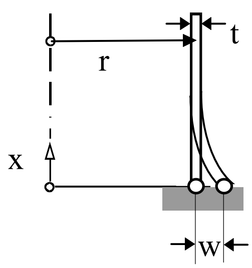
BC1f

Maximum σ_{sx}	Maximum $\sigma_{s\theta}$	Maximum τ_{xn}	Maximum $\sigma_{eq,s}$	Maximum $\sigma_{eq,m}$
$k_x\,\sigma_{MT\theta}$	$k_\theta\,\sigma_{MT\theta}$	$k_\tau\sqrt{t/r}\;\sigma_{MT\theta}$	$k_{eq,s}\,\sigma_{MT\theta}$	$k_{eq,m}\,\sigma_{MT\theta}$

$\left(\frac{\sqrt{rt}}{\ell_p}\right)$	k_x	k_θ	k_τ	$k_{eq,s}$	$k_{eq,m}$
0	0,585	1,125	0,583	1,126	1,067
0,2	0,585	0,873	0,583	0,919	0,759

Linear interpolation in $\left(\frac{\sqrt{rt}}{\ell_p}\right)$ may be used for different values of ℓ_p.

C.3.5 Cylinder, pinned: radial outward displacement

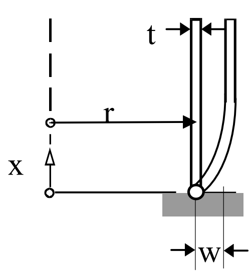


$$\sigma_{MT\theta} = \frac{wE}{r}$$

BC1f

Maximum σ_{sx}	Maximum $\sigma_{s\theta}$	Maximum τ_{xn}	Maximum $\sigma_{eq,s}$	Maximum $\sigma_{eq,m}$
$\pm 0{,}585\;\sigma_{MT\theta}$	$1{,}000\;\sigma_{MT\theta}$	$0{,}583\sqrt{t/r}\;\sigma_{MT\theta}$	$1{,}000\;\sigma_{MT\theta}$	$1{,}000\;\sigma_{MT\theta}$

C.3.6 Cylinder, pinned: uniform temperature rise

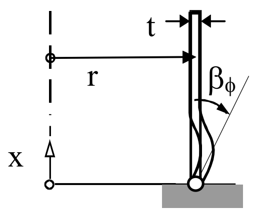


$$\sigma_{MT\theta} = \alpha\,E\,T$$
$$w = \alpha\,r\,T$$

BC1f

Maximum σ_{sx}	Maximum $\sigma_{s\theta}$	Maximum τ_{xn}	Maximum $\sigma_{eq,s}$	Maximum $\sigma_{eq,m}$
$\pm 0{,}585\;\sigma_{MT\theta}$	$1{,}000\;\sigma_{MT\theta}$	$0{,}583\sqrt{t/r}\;\sigma_{MT\theta}$	$1{,}000\;\sigma_{MT\theta}$	$1{,}000\;\sigma_{MT\theta}$

C.3.7 Cylinder, pinned: rotation of boundary



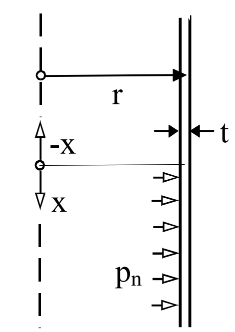
$$\sigma_{MT\theta} = E\sqrt{\frac{t}{r}}\,\beta_\phi$$

BC1f

Maximum σ_{sx}	Maximum $\sigma_{s\theta}$	Maximum τ_{xn}	Maximum $\sigma_{eq,s}$	Maximum $\sigma_{eq,m}$
$\pm 1,413\ \sigma_{MT\theta}$	$0,470\ \sigma_{MT\theta}$	$0,454\sqrt{t/r}\ \sigma_{MT\theta}$	$1,255\ \sigma_{MT\theta}$	$0,251\ \sigma_{MT\theta}$

C.4 Internal conditions in unstiffened cylindrical shells

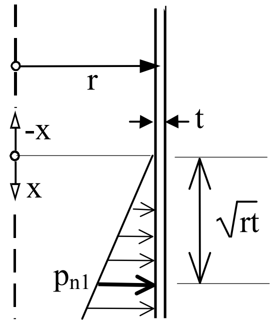
C.4.1 Cylinder: step change of internal pressure



$$\sigma_{MT\theta} = p_n \frac{r}{t}$$

Maximum σ_{sx}	Maximum $\sigma_{s\theta}$	Maximum τ_{xn}	Maximum $\sigma_{eq,s}$	Maximum $\sigma_{eq,m}$
$\pm 0,293\ \sigma_{MT\theta}$	$1,062\ \sigma_{MT\theta}$	$0,467\sqrt{t/r}\ \sigma_{MT\theta}$	$1,056\ \sigma_{MT\theta}$	$1,033\ \sigma_{MT\theta}$

C.4.2 Cylinder: hydrostatic internal pressure termination



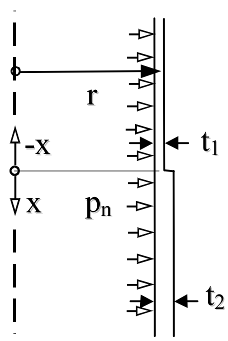
$$\sigma_{MT\theta} = p_{n1}\,\frac{r}{t}$$

p_{n1} is the pressure at a depth of $\sqrt{rt}$ below the surface

Maximum σ_{sx}	Maximum $\sigma_{s\theta}$	Maximum τ_{xn}	Maximum $\sigma_{eq,s}$	Maximum $\sigma_{eq,m}$
$k_x\,\sigma_{MT\theta}$	$k_\theta\,\sigma_{MT\theta}$	$k_\tau\sqrt{t/r}\;\sigma_{MT\theta}$	$k_{eq,s}\,\sigma_{MT\theta}$	$k_{eq,m}\,\sigma_{MT\theta}$

k_x	k_θ	k_τ	$k_{eq,s}$	$k_{eq,m}$
−1,060	0,510	0,160	1,005	0,275

C.4.3 Cylinder: step change of thickness



$$\sigma_{MT\theta} = p_n\,\frac{r}{t_1}$$

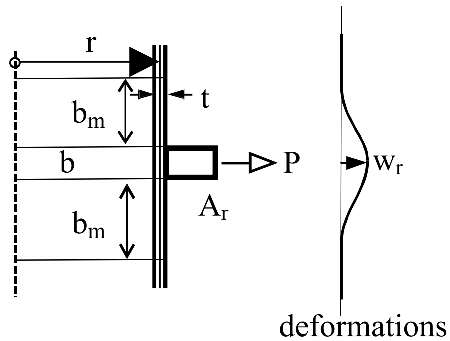
Maximum σ_{sx}	Maximum $\sigma_{s\theta}$	Maximum τ_{xn}	Maximum $\sigma_{eq,s}$	Maximum $\sigma_{eq,m}$
$k_x\,\sigma_{MT\theta}$	$k_\theta\,\sigma_{MT\theta}$	$k_\tau\sqrt{t/r}\;\sigma_{MT\theta}$	$k_{eq,s}\,\sigma_{MT\theta}$	$k_{eq,m}\,\sigma_{MT\theta}$

$\left(\frac{t_1}{t_2}\right)$	k_x	k_θ	k_τ	$k_{eq,s}$	$k_{eq,m}$
1,0	0,0	1,0	0,0	1,0	1,0
0,8	0,0256	1,010	0,179	1,009	0,895
0,667	0,0862	1,019	0,349	1,015	0,815
0,571	0,168	1,023	0,514	1,019	0,750
0,5	0,260	1,027	0,673	1,023	0,694

C.5 Ring stiffener on cylindrical shell

C.5.1 Ring stiffened cylinder: radial force on ring

The stresses in the shell should be determined using the calculated value of w from this clause introduced into the expressions given in C.2.5.
Where there is a change in the shell thickness at the ring, the method set out in 8.2.2 of EN 1993-4-1 should be used.



$$w = w_r$$

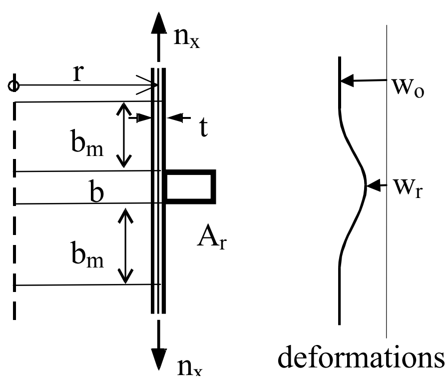
$$b_m = 0{,}778\sqrt{rt}$$

$$\left(\frac{wE}{r}\right) = \frac{Pr}{A_r + (b+2b_m)t}$$

$$\sigma_{\theta r} = \frac{Pr}{A_r + (b+2b_m)t}$$

C.5.2 Ring stiffened cylinder: axial loading

The stresses in the shell should be determined using the calculated value of w from this clause introduced into the expressions given in C.2.5 and C.2.2.



$$\sigma_{MTx} = \frac{n_x}{t}$$

$$w = w_r - w_o$$

$$w_o = -\nu\sigma_{MTx}\frac{r}{E}$$

$$b_m = 0{,}778\sqrt{rt}$$

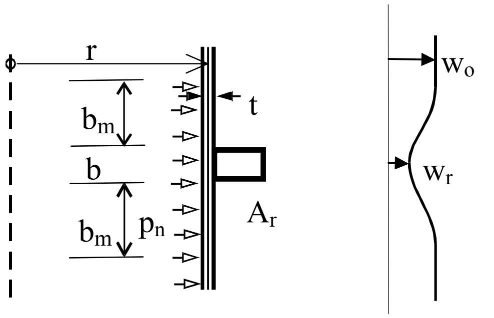
$$w_r = w_o\frac{(b+2b_m)t}{A_r + (b+2b_m)t}$$

$$w = -w_o\frac{A_r}{A_r + (b+2b_m)t}$$

$$\sigma_{\theta r} = E\frac{w_r}{r}$$

C.5.3 Ring stiffened cylinder: uniform internal pressure

The stresses in the shell should be determined using the calculated value of w from this clause introduced into the expressions given in C.2.5 and C.2.1.



deformations

$$\sigma_{MT\theta} = \frac{p_n r}{t}$$

$$w = w_r - w_o$$

$$w_o = \sigma_{MT\theta}\frac{r}{E}$$

$$b_m = 0{,}778\sqrt{rt}$$

$$w_r = w_o(1-\kappa)$$

$$w = -w_o\,\kappa$$

$$\kappa = \frac{A_r}{A_r + (b+2b_m)t}$$

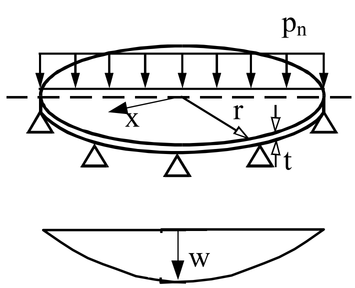
$$\sigma_{\theta r} = E\frac{w_r}{r}$$

Maximum σ_{sx}	Maximum $\sigma_{s\theta}$	Maximum τ_{xn}	Maximum $\sigma_{eq,s}$	Maximum $\sigma_{eq,m}$
$k_x\,\sigma_{MT\theta}$	$k_\theta\,\sigma_{MT\theta}$	$k_\tau\sqrt{t/r}\;\sigma_{MT\theta}$	$k_{eq,s}\,\sigma_{MT\theta}$	$k_{eq,m}\,\sigma_{MT\theta}$

κ	k_x	k_θ	k_τ	$k_{eq,s}$	$k_{eq,m}$
1,0	1,816	1,080	1,169	1,614	1,043
0,75	1,312	1,060	0,877	1,290	1,032
0,50	0,908	1,040	0,585	1,014	1,021
0,0	0,0	1,000	0,0	1,000	1,000

C.6 Circular plates with axisymmetric boundary conditions

C.6.1 Plate with simply supported boundary: uniform load



deflected shape

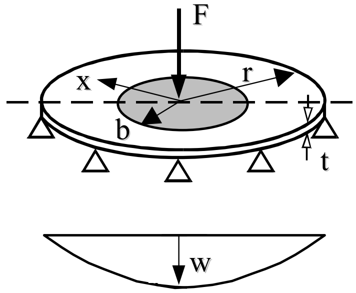
$$w = 0{,}696\,\frac{p_n r^4}{Et^3}$$

$$\text{max. } \sigma_{xb} = 1{,}238\,p_n\left(\frac{r}{t}\right)^2$$

$$\text{max. } \sigma_{\theta b} = 1{,}238\,p_n\left(\frac{r}{t}\right)^2$$

$$p_{n,y} = 0{,}808\left(\frac{t}{r}\right)^2 f_y$$

C.6.2 Plate with local distributed load: simply supported boundary



deflected shape

Uniform pressure p_n on circular patch of radius b

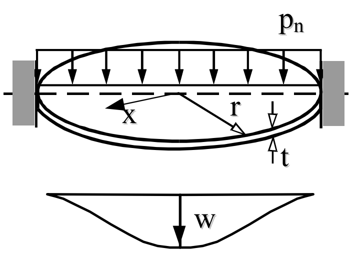
$F = p_n \pi b^2$ $\qquad b < 0{,}2\, r$

$$w = 0{,}606 \frac{Fr^2}{Et^3}$$

$$\max.\ \sigma_{xb} = \max.\ \sigma_{\theta b} = 0{,}621 \frac{F}{t^2} \left(\ln\frac{b}{r} + 0{,}769 \right)$$

$$F_y = 1{,}611 \frac{t^2}{\left(\ln\frac{b}{r} + 0{,}769 \right)} f_y$$

C.6.3 Plate with fixed boundary: uniform load



deflected shape

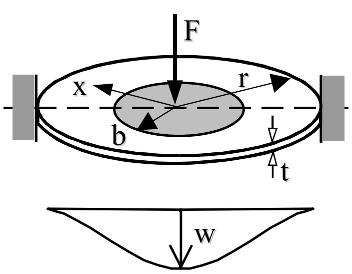
$$w = 0{,}171 \frac{p_n r^4}{Et^3}$$

$$\sigma_o = p_n \left(\frac{r}{t}\right)^2$$

$$p_{n,y} = 1{,}50 \left(\frac{t}{r}\right)^2 f_y \qquad \text{(at edge)}$$

Maximum σ_{bx} at centre	Maximum $\sigma_{b\theta}$ at centre	Maximum σ_{eq} at centre	Maximum σ_{bx} at edge	Maximum $\sigma_{b\theta}$ at edge	Maximum σ_{eq} at edge
$0{,}488\ \sigma_o$	$0{,}488\ \sigma_o$	$0{,}488\ \sigma_o$	$0{,}75\ \sigma_o$	$0{,}225\ \sigma_o$	$0{,}667\ \sigma_o$

C.6.4 Plate with fixed boundary: local distributed load



deflected shape

Uniform pressure p_n on circular patch of radius b

$F = p_n \pi b^2$ $\qquad b < 0{,}2\, r$

$$w = 0{,}217 \frac{Fr^2}{Et^3}$$

$$\sigma_o = \frac{F}{t^2} \qquad F_y = 1{,}611 \frac{t^2}{\left(\ln\frac{b}{r} \right)} f_y \quad \text{at centre}$$

Maximum σ_{bx} at centre	Maximum $\sigma_{b\theta}$ at centre	Maximum σ_{eq} at centre	Maximum σ_{bx} at edge	Maximum $\sigma_{b\theta}$ at edge	Maximum σ_{eq} at edge
$0{,}621\left(\ln\frac{b}{r}\right)\sigma_o$	$0{,}621\left(\ln\frac{b}{r}\right)\sigma_o$	$0{,}621\left(\ln\frac{b}{r}\right)\sigma_o$	$0{,}477\ \sigma_o$	$0{,}143\ \sigma_o$	$0{,}424\ \sigma_o$

ANNEX D [normative]
Expressions for buckling stress design

D.1 Unstiffened cylindrical shells of constant wall thickness

D.1.1 Notation and boundary conditions

(1) Geometrical quantities

- ℓ cylinder length between defined boundaries
- r radius of cylinder middle surface
- t thickness of shell
- Δw_k characteristic imperfection amplitude

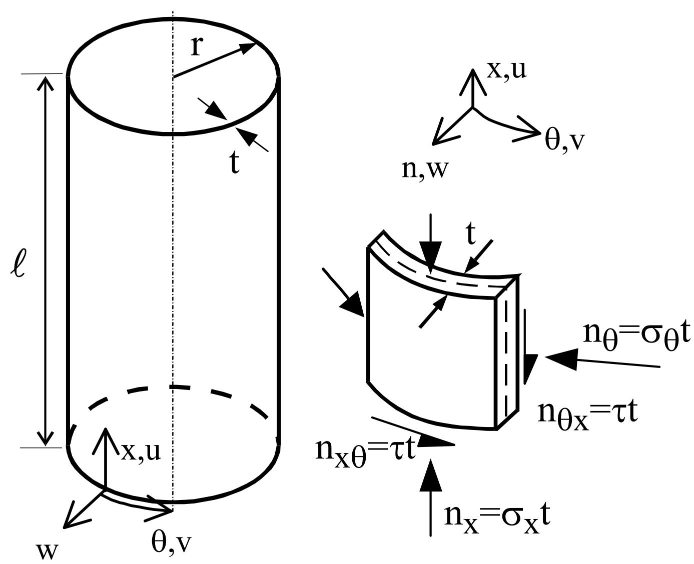


Figure D.1: Cylinder geometry, membrane stress resistances and stress resultant resistances

(2) The relevant boundary conditions are set out in 2.3, 5.2.2 and 8.3.

D.1.2 Meridional (axial) compression

D.1.2.1 Critical meridional buckling stresses

(1) The following expressions may only be used for shells with boundary conditions BC 1 or BC 2 at both edges.

(2) The length of the shell segment is characterised in terms of the dimensionless length parameter ω:

$$\omega = \frac{\ell}{r}\sqrt{\frac{r}{t}} = \frac{\ell}{\sqrt{rt}} \qquad \text{... (D.1)}$$

(3) The critical meridional buckling stress, using a value of C_x from (4), (5) or (6), should be obtained from:

$$\sigma_{x,Rcr} = 0{,}605\, E\, C_x \frac{t}{r} \qquad \text{... (D.2)}$$

(4) For medium-length cylinders, which are defined by:

$$1{,}7 \leq \omega \leq 0{,}5 \frac{r}{t} \qquad \text{... (D.3)}$$

the factor C_x should be taken as:

$$C_x = 1{,}0 \qquad \text{... (D.4)}$$

(5) For short cylinders, which are defined by:

$$\omega \leq 1{,}7 \qquad \text{... (D.5)}$$

the factor C_x may be taken as:

$$C_x = 1{,}36 - \frac{1{,}83}{\omega} + \frac{2{,}07}{\omega^2} \qquad \text{... (D.6)}$$

(6) For long cylinders, which are defined by:

$$\omega > 0{,}5 \frac{r}{t} \qquad \text{... (D.7)}$$

the factor C_x should be found as:

$$C_x = C_{x,N} \qquad \text{... (D.8)}$$

in which $C_{x,N}$ is the greater of:

$$C_{x,N} = 1 + \frac{0{,}2}{C_{xb}} \left[1 - 2\omega \frac{t}{r} \right] \qquad \text{... (D.9)}$$

and

$$C_{x,N} = 0{,}60 \qquad \text{... (D.10)}$$

where C_{xb} is a parameter depending on the boundary conditions and being taken from table D.1.

Table D.1: Parameter C_{xb} for the effect of boundary conditions on the critical meridional buckling stress in long cylinders

Case	Cylinder end	Boundary condition	C_{xb}
1	end 1 end 2	BC 1 BC 1	6
2	end 1 end 2	BC 1 BC 2	3
3	end 1 end 2	BC 2 BC 2	1

(7) For long cylinders as defined in (6) that satisfy the additional conditions:

$$\frac{r}{t} \leq 150 \quad \text{and} \quad \omega\left(\frac{t}{r}\right) \leq 6 \quad \text{and} \quad 500 \leq \frac{E}{f_{y,k}} \leq 1000 \qquad \text{... (D.11)}$$

the factor C_x may alternatively be obtained from:

$$C_x = C_{x,N}\left(\frac{\sigma_{xE,N}}{\sigma_{xE}}\right) + \left(\frac{\sigma_{xE,M}}{\sigma_{xE}}\right) \qquad \text{... (D.12)}$$

where:

σ_{xE}	is	the design value of the meridional stress $\sigma_{x,Ed}$
$\sigma_{xE,N}$	is	the component of $\sigma_{x,Ed}$ that derives from axial compression (circumferentially uniform component)
$\sigma_{xE,M}$	is	the component of $\sigma_{x,Ed}$ that derives from tubular global bending (peak value of the circumferentially varying component)

The following simpler expression may also be used in place of expression (D.12):

$$C_x = 0{,}60 + 0{,}40\left(\frac{\sigma_{xE,M}}{\sigma_{xE}}\right) \qquad \text{... (D.13)}$$

D.1.2.2 Meridional buckling parameters

(1) The meridional elastic imperfection factor should be obtained from:

$$\alpha_x = \frac{0{,}62}{1 + 1{,}91\,(\Delta w_k / t)^{1{,}44}} \qquad \text{... (D.14)}$$

where Δw_k is the characteristic imperfection amplitude:

$$\Delta w_k = \frac{1}{Q}\sqrt{\frac{r}{t}}\; t \qquad \text{... (D.15)}$$

where Q is the meridional compression fabrication quality parameter.

(2) The fabrication quality parameter Q should be taken from table D.2 for the specified fabrication tolerance quality class.

Table D.2: Values of fabrication quality parameter *Q*

Fabrication tolerance quality class	Description	Q
Class A	Excellent	40
Class B	High	25
Class C	Normal	16

(3) The meridional squash limit slenderness $\bar{\lambda}_{x0}$, the plastic range factor β, and the interaction exponent η should be taken as:

$$\bar{\lambda}_{x0} = 0{,}20 \qquad \beta = 0{,}60 \qquad \eta = 1{,}0 \qquad \text{... (D.16)}$$

(4) For long cylinders that satisfy the special conditions of D.1.2.1 (7), the meridional squash limit slenderness $\bar{\lambda}_{x0}$ may be obtained from:

$$\bar{\lambda}_{x0} = 0{,}20 + 0{,}10\left(\frac{\sigma_{xE,M}}{\sigma_{xE}}\right) \qquad \text{... (D.17)}$$

where:

σ_{xE}	is	the design value of the meridional stress $\sigma_{x,Ed}$
$\sigma_{xE,M}$	is	the component of $\sigma_{x,Ed}$ that derives from tubular global bending (peak value of the circumferentially varying component)

(5) Cylinders need not be checked against meridional shell buckling if they satisfy:

$$\frac{r}{t} \leq 0{,}03\frac{E}{f_{y,k}} \qquad \text{... (D.18)}$$

D.1.3 Circumferential (hoop) compression

D.1.3.1 Critical circumferential buckling stresses

(1) The following expressions may be applied to shells with all boundary conditions.

(2) The length of the shell segment should be characterised in terms of the dimensionless length parameter ω:

$$\omega = \frac{\ell}{r}\sqrt{\frac{r}{t}} = \frac{\ell}{\sqrt{rt}} \qquad \text{... (D.19)}$$

(3) For medium-length cylinders, which are defined by:

$$20 \leq \frac{\omega}{C_\theta} \leq 1{,}63\frac{r}{t} \qquad \text{... (D.20)}$$

the critical circumferential buckling stress should be obtained from:

$$\sigma_{\theta,\mathrm{Rcr}} = 0{,}92\,E\,\frac{C_\theta}{\omega}\,\frac{t}{r} \qquad \text{... (D.21)}$$

(4) The factor C_θ should be taken from table D.3, with a value that depends on the boundary conditions (see 5.2.2 and 8.3).

(5) For short cylinders, which are defined by:

$$\frac{\omega}{C_\theta} < 20 \qquad \text{... (D.22)}$$

the critical circumferential buckling stress should be obtained instead from:

$$\sigma_{\theta,\mathrm{Rcr}} = 0{,}92\,E\,\frac{C_{\theta s}}{\omega}\,\frac{t}{r} \qquad \text{... (D.23)}$$

(6) The factor $C_{\theta s}$ should be taken from table D.4, with a value that depends on the boundary conditions (see 5.2.2 and 8.3):

Table D.3: External pressure buckling factors for medium-length cylinders C_θ

Case	Cylinder end	Boundary condition	Value of C_θ
1	end 1 end 2	BC 1 BC 1	1,5
2	end 1 end 2	BC 1 BC 2	1,25
3	end 1 end 2	BC 2 BC 2	1,0
4	end 1 end 2	BC 1 BC 3	0,6
5	end 1 end 2	BC2 BC3	0
6	end 1 end 2	BC 3 BC 3	0

Table D.4: External pressure buckling factors for short cylinders $C_{\theta s}$

Case	Cylinder end	Boundary condition	$C_{\theta s}$
1	end 1 end 2	BC 1 BC 1	$1,5 + \frac{10}{\omega^2} - \frac{5}{\omega^3}$
2	end 1 end 2	BC 1 BC 2	$1,25 + \frac{8}{\omega^2} - \frac{4}{\omega^3}$
3	end 1 end 2	BC 2 BC 2	$1,0 + \frac{3}{\omega^{1,35}}$
4	end 1 end 2	BC 1 BC 3	$0,6 + \frac{1}{\omega^2} - \frac{0,3}{\omega^3}$
where $\omega = \frac{\ell}{\sqrt{rt}}$			

(7) For long cylinders, which are defined by:

$$\frac{\omega}{C_\theta} > 1,63\,\frac{r}{t} \qquad \text{... (D.24)}$$

the critical circumferential buckling stress should be obtained from:

$$\sigma_{\theta,\mathrm{Rcr}} = E\left(\frac{t}{r}\right)^2\left[0,275 + 2,03\left(\frac{C_\theta}{\omega}\,\frac{r}{t}\right)^4\right] \qquad \text{... (D.25)}$$

D.1.3.2 Circumferential buckling parameters

(1) The circumferential elastic imperfection factor should be taken from table D.5 for the specified fabrication tolerance quality class.

Table D.5 : Values of α_θ based on fabrication quality

Fabrication tolerance quality class	Description	α_θ
Class A	Excellent	0,75
Class B	High	0,65
Class C	Normal	0,50

(2) The circumferential squash limit slenderness $\overline{\lambda}_{\theta 0}$, the plastic range factor β, and the interaction exponent η should be taken as:

$$\overline{\lambda}_{\theta 0} = 0{,}40 \qquad \beta = 0{,}60 \qquad \eta = 1{,}0 \qquad \text{... (D.26)}$$

(3) Cylinders need not be checked against circumferential shell buckling if they satisfy:

$$\frac{r}{t} \leq 0{,}21\sqrt{\frac{E}{f_{y,k}}} \qquad \text{... (D.27)}$$

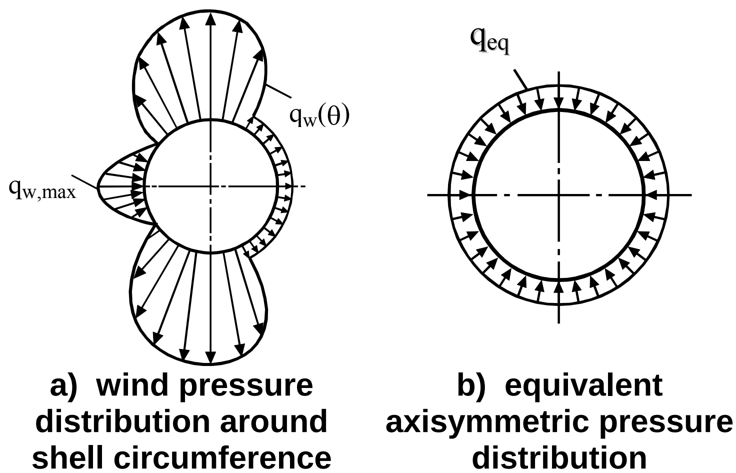


Figure D.2: Transformation of typical wind external pressure load distribution

(4) The non-uniform distribution of pressure q_w resulting from external wind loading on cylinders (see figure D.2) may, for the purpose of shell buckling design, be substituted by an equivalent uniform external pressure:

$$q_{eq} = k_w \; q_{w,max} \qquad \text{... (D.28)}$$

where $q_{w,max}$ is the maximum wind pressure, and k_w should be found as follows:

$$k_w = 0{,}46\left(1 + 0{,}1\sqrt{\frac{C_\theta}{\omega}\frac{r}{t}}\right) \qquad \text{... (D.29)}$$

with the value of k_w not outside the range $0{,}65 \leq k_w \leq 1$, and with C_θ taken from table D.3 according to the boundary conditions.

(5) The circumferential design stress to be introduced into 8.5 follows from:

$$\sigma_{\theta,Ed} = (q_{eq} + q_s)\frac{r}{t} \qquad \text{... (D.30)}$$

where q_s is the internal suction caused by venting, internal partial vacuum or other phenomena.

D.1.4 Shear

D.1.4.1 Critical shear buckling stresses

(1) The following expressions should be applied only to shells with boundary conditions BC1 or BC2 at both edges.

(2) The length of the shell segment should be characterised in terms of the dimensionless length parameter ω:

$$\omega = \frac{\ell}{r}\sqrt{\frac{r}{t}} = \frac{\ell}{\sqrt{rt}} \quad \text{... (D.31)}$$

(3) The critical shear buckling stress should be obtained from:

$$\tau_{x\theta,Rcr} = 0{,}75\, E\, C_\tau \sqrt{\frac{1}{\omega}}\, \frac{t}{r} \quad \text{... (D.32)}$$

(4) For medium-length cylinders, which are defined by:

$$10 \leq \omega \leq 8{,}7\frac{r}{t} \quad \text{... (D.33)}$$

the factor C_τ may be found as:

$$C_\tau = 1{,}0 \quad \text{... (D.34)}$$

(5) For short cylinders, which are defined by:

$$\omega < 10 \quad \text{... (D.35)}$$

the factor C_τ may be obtained from:

$$C_\tau = \sqrt{1 + \frac{42}{\omega^3}} \quad \text{... (D.36)}$$

(6) For long cylinders, which are defined by:

$$\omega > 8{,}7\frac{r}{t} \quad \text{... (D.37)}$$

the factor C_τ may be obtained from:

$$C_\tau = \frac{1}{3}\sqrt{\omega\frac{t}{r}} \quad \text{... (D.38)}$$

D.1.4.2 Shear buckling parameters

(1) The shear elastic imperfection factor should be taken from table D.6 for the specified fabrication tolerance quality class.

Table D.6: Values of α_τ based on fabrication quality

Fabrication tolerance quality class	Description	α_τ
Class A	Excellent	0,75
Class B	High	0,65
Class C	Normal	0,50

(2) The shear squash limit slenderness $\overline{\lambda}_{\tau 0}$, the plastic range factor β, and the interaction exponent η should be taken as:

$$\overline{\lambda}_{\tau 0} = 0{,}40 \qquad \beta = 0{,}60 \qquad \eta = 1{,}0 \quad \text{... (D.39)}$$

(3) Cylinders need not be checked against shear shell buckling if they satisfy:

$$\frac{r}{t} \leq 0{,}16 \left[\frac{E}{f_{y,k}}\right]^{0{,}67} \qquad \text{... (D.40)}$$

D.1.5 Meridional (axial) compression with coexistent internal pressure

D.1.5.1 Pressurised critical meridional buckling stress

(1) The critical meridional buckling stress $\sigma_{x,Rcr}$ may be assumed to be unaffected by the presence of internal pressure and may be obtained as specified in D.1.2.1.

D.1.5.2 Pressurised meridional buckling parameters

(1) The pressurised meridional buckling stress should be verified analogously to the unpressurised meridional buckling stress as specified in 8.5 and D.1.2.2. However, the unpressurised elastic imperfection factor α_x may be replaced by the pressurised elastic imperfection factor α_{xp}.

(2) The pressurised elastic imperfection factor α_{xp} should be taken as the smaller of the two following values:

α_{xpe} is a factor covering pressure-induced elastic stabilisation;

α_{xpp} is a factor covering pressure-induced plastic destabilisation

(3) The factor α_{xpe} should be obtained from:

$$\alpha_{xpe} = \alpha_x + (1-\alpha_x)\left[\frac{\overline{p}_s}{\overline{p}_s + 0{,}3/\alpha_x^{0{,}5}}\right] \qquad \text{... (D.41)}$$

$$\overline{p}_s = \frac{p_s}{\sigma_{x,Rcr}}\frac{r}{t} \qquad \text{... (D.42)}$$

where:

p_s is the smallest design value of local internal pressure at the location of the point being assessed, guaranteed to coexist with the meridional compression,

α_x is the unpressurised meridional elastic imperfection factor according to D.1.2.2, and

$\sigma_{x,Rcr}$ is the elastic critical meridional buckling stress according to D.1.2.1 (3).

(4) The factor α_{xpe} should not be applied to cylinders that are long according to D.1.2.1 (6). In addition, it should not be applied unless one of the following two conditions are met:

- the cylinder is medium-length according to D.1.2.1 (4);
- the cylinder is short according to D.1.2.1 (5) and $C_x = 1$ has been adopted in D.1.2.1 (3).

(5) The factor α_{xpp} should be obtained from:

$$\alpha_{xpp} = \left\{1 - \left(\frac{\overline{p}_g}{\overline{\lambda}_x^2}\right)^2\right\}\left[1 - \frac{1}{1{,}12 + s^{3/2}}\right]\left[\frac{s^2 + 1{,}21\,\overline{\lambda}_x^2}{s\,(s+1)}\right] \qquad \text{... (D.43)}$$

$$\overline{p}_g = \frac{p_g}{\sigma_{x,Rcr}}\frac{r}{t} \qquad \text{... (D.44)}$$

$$s = \frac{1}{400}\frac{r}{t} \qquad \text{... (D.45)}$$

where:

p_g is the largest design value of local internal pressure at the location of the point being assessed, and possibly coexistent with the meridional compression;

$\overline{\lambda}_x$ is the dimensionless shell slenderness parameter according to 8.5.2 (5);
$\sigma_{x,Rcr}$ is the elastic critical meridional buckling stress according to D.1.2.1 (3).

D.1.6 Combinations of meridional (axial) compression, circumferential (hoop) compression and shear

(1) The buckling interaction parameters to be used in 8.5.3 (3) may be obtained from:

$$k_x = 1{,}25 + 0{,}75\ \chi_x \qquad \text{... (D.46)}$$

$$k_\theta = 1{,}25 + 0{,}75\ \chi_\theta \qquad \text{... (D.47)}$$

$$k_\tau = 1{,}75 + 0{,}25\ \chi_\tau \qquad \text{... (D.48)}$$

$$k_i = (\chi_x\ \chi_\theta)^2 \qquad \text{... (D.49)}$$

where:

χ_x, χ_θ, χ_τ are the buckling reduction factors defined in 8.5.2, using the buckling parameters given in D.1.2 to D.1.4.

(2) The three membrane stress components should be deemed to interact in combination at any point in the shell, except those adjacent to the boundaries. The buckling interaction check may be omitted for all points that lie within the boundary zone length ℓ_R adjacent to either end of the cylindrical segment. The value of ℓ_R is the smaller of:

$$\ell_R = 0{,}1L \qquad \text{... (D.50)}$$

and

$$\ell_R \leq 0{,}16\, r\sqrt{r/t} \qquad \text{... (D.51)}$$

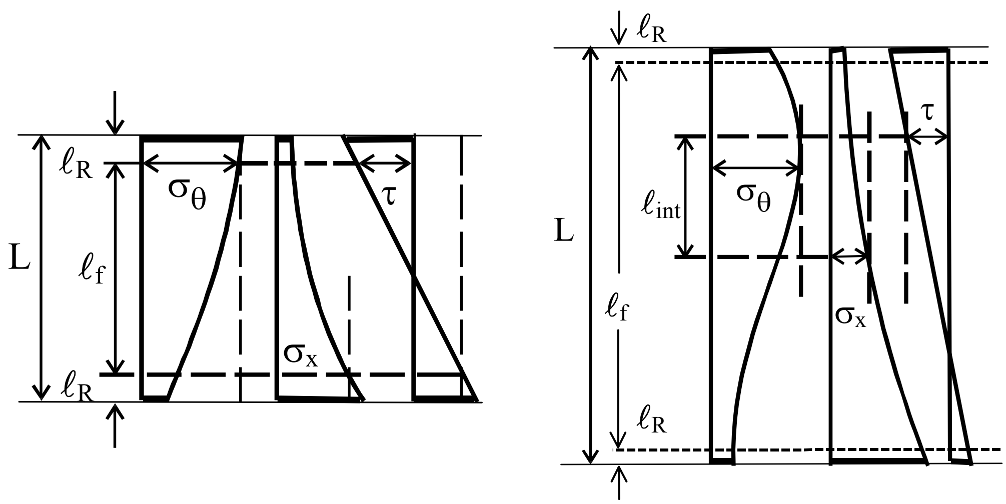
(3) Where checks of the buckling interaction at all points is found to be onerous, the following provisions of (4) and (5) permit a simpler conservative assessment. If the maximum value of any of the buckling-relevant membrane stresses in a cylindrical shell occurs in a boundary zone of length ℓ_R adjacent to either end of the cylinder, the interaction check of 8.5.3 (3) may be undertaken using the values defined in (4).

(4) Where the conditions of (3) are met, the maximum value of any of the buckling-relevant membrane stresses occurring over the free length ℓ_f that is outside the boundary zones (see figure D.3a) may be used in the interaction check of 8.5.3 (3), where:

$$\ell_f = L - 2\ell_R \qquad \text{... (D.52)}$$

(5) For long cylinders as defined in D.1.2.1 (6), the interaction-relevant groups introduced into the interaction check may be restricted further than the provisions of paragraphs (3) and (4). The stresses deemed to be in interaction-relevant groups may then be restricted to any section of length ℓ_{int} falling within the free remaining length ℓ_f for the interaction check (see figure D.3b), where:

$$\ell_{int} = 1{,}3\, r\sqrt{r/t} \qquad \text{... (D.53)}$$



a) in a short cylinder b) in a long cylinder

Figure D.3: Examples of interaction-relevant groups of membrane stress components

(6) If (3)-(5) above do not provide specific provisions for defining the relative locations or separations of interaction-relevant groups of membrane stress components, and a simple conservative treatment is still required, the maximum value of each membrane stress, irrespective of location in the shell, may be adopted into expression (8.19).

D.2 Unstiffened cylindrical shells of stepwise variable wall thickness

D.2.1 General

D.2.1.1 Notation and boundary conditions

(1) In this clause the following notation is used:

- L overall cylinder length
- r radius of cylinder middle surface
- j an integer index denoting the individual cylinder sections with constant wall thickness (from $j = 1$ to $j = n$)
- t_j the constant wall thickness of section j of the cylinder
- ℓ_j the length of section j of the cylinder

(2) The following expressions may only be used for shells with boundary conditions BC 1 or BC 2 at both edges (see 5.2.2 and 8.3), with no distinction made between them.

D.2.1.2 Geometry and joint offsets

(1) Provided that the wall thickness of the cylinder increases progressively stepwise from top to bottom (see figure D.4), the procedures given in this clause D.2 may be used.

(2) Intended offsets e_0 between plates of adjacent sections (see figure D.4) may be treated as covered by the following expressions provided that the intended value e_0 is less than the permissible value $e_{0,p}$ which should be taken as the smaller of:

$$e_{0,p} = 0{,}5\,(t_{max} - t_{min}) \qquad \text{... (D.54)}$$

and

$$e_{0,p} = 0{,}5\ t_{min} \qquad \text{... (D.55)}$$

where:

t_{max} is the thickness of the thicker plate at the joint;
t_{min} is the thickness of the thinner plate at the joint.

(3) For cylinders with permissible intended offsets between plates of adjacent sections according to (2), the radius r may be taken as the mean value of all sections.

(4) For cylinders with overlapping joints (lap joints), the provisions for lap-jointed construction given in D.3 below should be used.

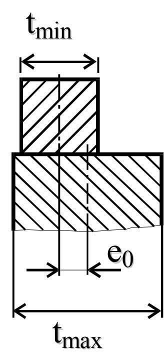


Figure D.4: Intended offset e_0 in a butt-jointed shell

D.2.2 Meridional (axial) compression

(1) Each cylinder section j of length ℓ_j should be treated as an equivalent cylinder of overall length $\ell = L$ and of uniform wall thickness $t = t_j$ according to D.1.2.

(2) For long equivalent cylinders, as governed by D.1.2.1 (6), the parameter C_{xb} should be conservatively taken as $C_{xb} = 1$, unless a better value is justified by more rigorous analysis.

D.2.3 Circumferential (hoop) compression

D.2.3.1 Critical circumferential buckling stresses

(1) If the cylinder consists of three sections with different wall thickness, the procedure of (4) to (7) should be applied to the real sections a, b and c (see figure D.5b).

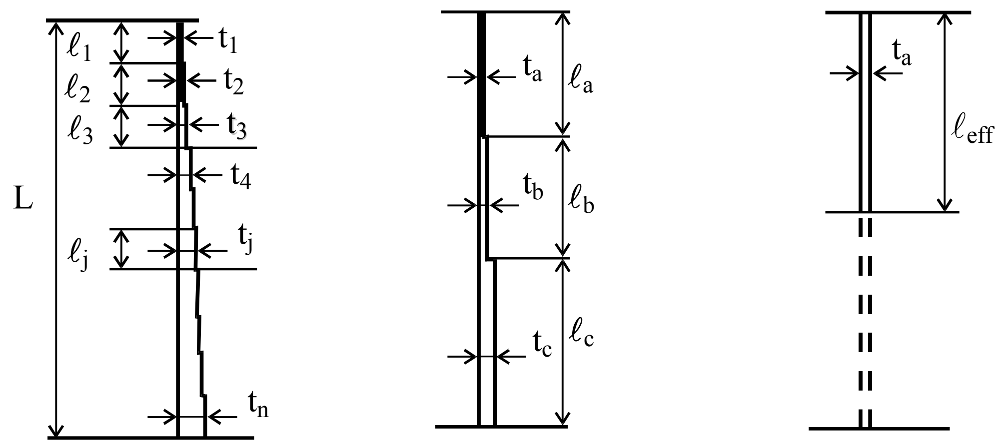
(2) If the cylinder consists of only one section (i.e. constant wall thickness), D.1 should be applied.

(3) If the cylinder consists of two sections of different wall thickness, the procedure of (4) to (7) should be applied, treating two of the three fictitious sections, a and b, as being of the same thickness.

(4) If the cylinder consists of more than three sections with different wall thicknesses (see figure D.5a), it should first be replaced by an equivalent cylinder comprising three sections a, b and c (see figure D.5b). The length of its upper section, ℓ_a, should extend to the upper edge of the first section that has a wall thickness greater than 1,5 times the smallest wall thickness t_1, but should not comprise more than half the total length L of the cylinder. The length of the two other sections ℓ_b and ℓ_c should be obtained as follows:

$$\ell_b = \ell_a \ \text{ and } \ \ell_c = L - 2\ell_a, \qquad \text{if } \ \ell_a \leq L/3 \qquad \text{... (D.56)}$$

$$\ell_b = \ell_c = 0{,}5\,(L - \ell_a), \qquad \text{if} \quad L/3 < \ell_a \leq L/2 \qquad \text{... (D.57)}$$



(a) Cylinder of stepwise variable wall thickness (b) Equivalent cylinder comprising three sections (c) Equivalent single cylinder with uniform wall thickness

Figure D.5: Transformation of stepped cylinder into equivalent cylinder

(5) The fictitious wall thicknesses t_a, t_b and t_c of the three sections should be determined as the weighted average of the wall thickness over each of the three fictitious sections:

$$t_a = \frac{1}{\ell_a} \sum_a \ell_j\, t_j \qquad \text{... (D.58)}$$

$$t_b = \frac{1}{\ell_b} \sum_b \ell_j\, t_j \qquad \text{... (D.59)}$$

$$t_c = \frac{1}{\ell_c} \sum_c \ell_j\, t_j \qquad \text{... (D.60)}$$

(6) The three-section-cylinder (i.e. the equivalent one or the real one respectively) should be replaced by an equivalent single cylinder of effective length ℓ_{eff} and of uniform wall thickness $t = t_a$ (see figure D.5c). The effective length should be determined from:

$$\ell_{eff} = \ell_a / \kappa \qquad \text{... (D.61)}$$

in which κ is a dimensionless factor obtained from figure D.6.

(7) For cylinder sections of moderate or short length, the critical circumferential buckling stress of each cylinder section j of the original cylinder of stepwise variable wall thickness should be determined from:

$$\sigma_{\theta,\mathrm{Rcr},j} = \frac{t_a}{t_j}\, \sigma_{\theta,\mathrm{Rcr,eff}} \qquad \text{... (D.62)}$$

where $\sigma_{\theta,\mathrm{Rcr,eff}}$ is the critical circumferential buckling stress derived from D.1.3.1 (3), D.1.3.1 (5) or D.1.3.1 (7), as appropriate, of the equivalent single cylinder of length ℓ_{eff} according to paragraph (6). The factor C_θ in these expressions should be given the value $C_\theta = 1{,}0$.

(8) The length of the shell segment is characterised in terms of the dimensionless length parameter ω_j:

$$\omega_j = \frac{\ell_j}{r}\sqrt{\frac{r}{t_j}} = \frac{\ell_j}{\sqrt{r t_j}} \qquad \text{... (D.63)}$$

(9) Where the cylinder section j is long, a second additional assessment of the buckling stress should be made. The smaller of the two values derived from (7) and (10) should be used for the buckling design check of the cylinder section j.

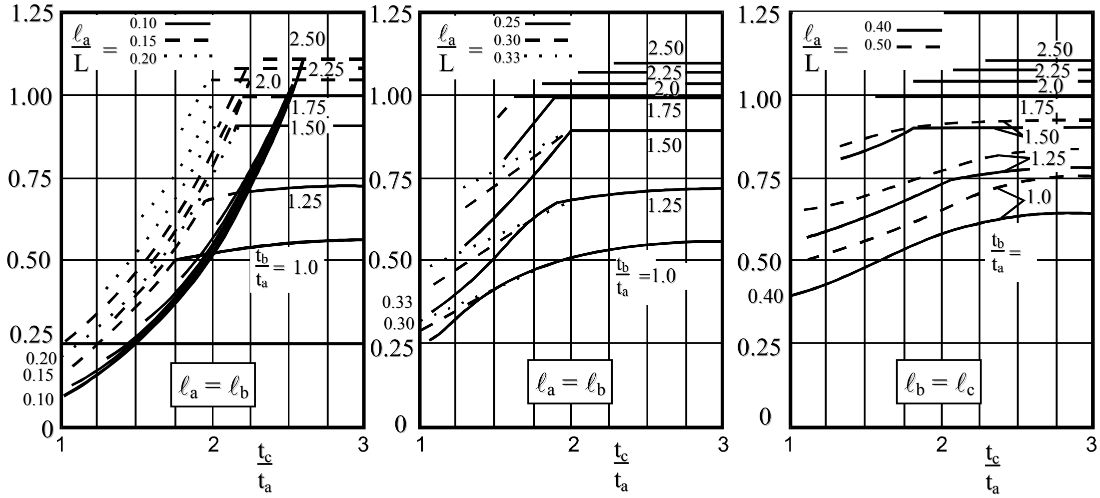


Figure D.6: Factor κ for determination of the effective length ℓ_{eff}

(10) The cylinder section j should be treated as long if:

$$\omega_j > 1{,}63 \frac{r}{t_j} \qquad \text{... (D.64)}$$

in which case the critical circumferential buckling stress should be determined from:

$$\sigma_{\theta,\mathrm{Rcr},j} = E\left(\frac{t_j}{r}\right)^2\left[0{,}275 + 2{,}03\left(\frac{1}{\omega_j}\frac{r}{t_j}\right)^4\right] \qquad \text{... (D.65)}$$

D.2.3.2 Buckling strength verification for circumferential compression

(1) For each cylinder section j, the conditions of 8.5 should be met, and the following check should be carried out:

$$\sigma_{\theta,\mathrm{Ed},j} \le \sigma_{\theta,\mathrm{Rd},j} \qquad \text{... (D.66)}$$

where:

$\sigma_{\theta,\mathrm{Ed},j}$ is the key value of the circumferential compressive membrane stress, as detailed in the following clauses;

$\sigma_{\theta,\mathrm{Rd},j}$ is the design circumferential buckling stress, as derived from the critical circumferential buckling stress according to D.1.3.2.

(2) Provided that the design value of the circumferential stress resultant $n_{\theta,\mathrm{Ed}}$ is constant throughout the length L, the key value of the circumferential compressive membrane stress in the section j, should be taken as the simple value:

$$\sigma_{\theta,\mathrm{Ed},j} = n_{\theta,\mathrm{Ed}} / t_j \qquad \text{... (D.67)}$$

(3) If the design value of the circumferential stress resultant $n_{\theta,Ed}$ varies within the length L, the key value of the circumferential compressive membrane stress should be taken as a fictitious value $\sigma_{\theta,Ed,j,mod}$ determined from the maximum value of the circumferential stress resultant $n_{\theta,Ed}$ anywhere within the length L divided by the local thickness t_j (see figure D.7), determined as:

$$\sigma_{\theta,Ed,j,mod} = \max(n_{\theta,Ed}) / t_j \qquad \text{... (D.68)}$$

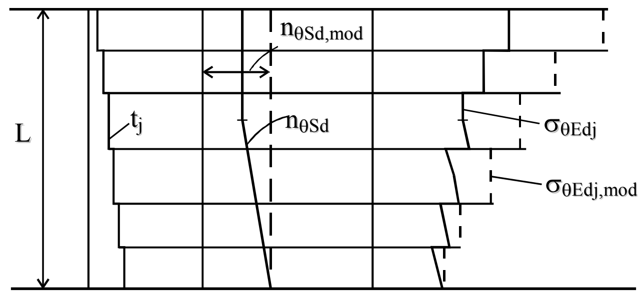


Figure D.7: Key values of the circumferential compressive membrane stress in cases where $n_{\theta,Ed}$ varies within the length L

D.2.4 Shear

D.2.4.1 Critical shear buckling stresses

(1) If no specific rule for evaluating an equivalent single cylinder of uniform wall thickness is available, the expressions of D.2.3.1 (1) to (6) may be applied.

(2) The further determination of the critical shear buckling stresses may on principle be performed as in D.2.3.1 (7) to (10), but replacing the circumferential compression expressions from D.1.3.1 by the relevant shear expressions from D.1.4.1.

D.2.4.2 Buckling strength verification for shear

(1) The rules of D.2.3.2 may be applied, but replacing the circumferential compression expressions by the relevant shear expressions.

D.3 Unstiffened lap jointed cylindrical shells

D.3.1 General

D.3.1.1 Definitions

D.3.1.1.1
circumferential lap joint
A lap joint that runs in the circumferential direction around the shell axis.

D.3.1.1.2
meridional lap joint
A lap joint that runs parallel to the shell axis (meridional direction).

D.3.1.2 Geometry and stress resultants

(1) Where a cylindrical shell is constructed using lap joints (see figure D.8), the following provisions may be used in place of those set out in D.2.

(2) The following provisions apply both to lap joints that increase, and to lap joints that decrease the radius of the middle surface of the shell. Where the lap joint runs in a circumferential direction around the shell axis (circumferential lap joint), the provisions of D.3.2 should be used for meridional compression. Where many lap joints run in a circumferential direction around the shell axis (circumferential lap joints) with changes of plate thickness down the shell, the provisions of D.3.3 should be used for circumferential compression. Where a single lap joint runs parallel to the shell axis (meridional lap joint), the provisions of D.3.3 should be used for circumferential compression. In other cases, no special consideration need be given for the influence of lap joints on the buckling resistance.

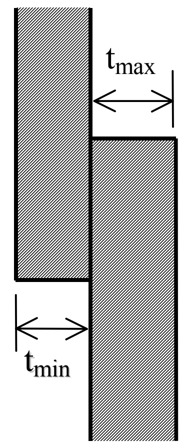


Figure D.8: Lap jointed shell

D.3.2 Meridional (axial) compression

(1) Where a lap jointed cylinder is subject to meridional compression, with meridional lap joints, the buckling resistance may be evaluated as for a uniform or stepped-wall cylinder, as appropriate, but with the design resistance reduced by the factor 0,70.

(2) Where a change of plate thickness occurs at the lap joint, the design buckling resistance may be taken as the same value as for that of the thinner plate as determined in (1).

D.3.3 Circumferential (hoop) compression

(1) Where a lap jointed cylinder is subject to circumferential compression across meridional lap joints, the design buckling resistance may be evaluated as for a uniform or stepped-wall cylinder, as appropriate, but with a reduction factor of 0,90.

(2) Where a lap jointed cylinder is subject to circumferential compression, with many circumferential lap joints and a changing plate thickness down the shell, the procedure of D.2 should be used without the geometric restrictions on joint eccentricity, and with the design buckling resistance reduced by the factor 0,90.

(3) Where the lap joints are used in both directions, with staggered placement of the meridional lap joints in alternate strakes or courses, the design buckling resistance should be evaluated as the lower of those found in (1) or (2), but no further resistance reduction need be applied.

D.3.4 Shear

(1) Where a lap jointed cylinder is subject to membrane shear, the buckling resistance may be evaluated as for a uniform or stepped-wall cylinder, as appropriate.

D.4 Unstiffened complete and truncated conical shells

D.4.1 General

D.4.1.1 Notation

In this clause the following notation is used:

h	is	the axial length (height) of the truncated cone;
L	is	the meridional length of the truncated cone;
r	is	the radius of the cone middle surface, perpendicular to axis of rotation, that varies linearly down the length;
r_1	is	the radius at the small end of the cone;
r_2	is	the radius at the large end of the cone;
β	is	the apex half angle of cone.

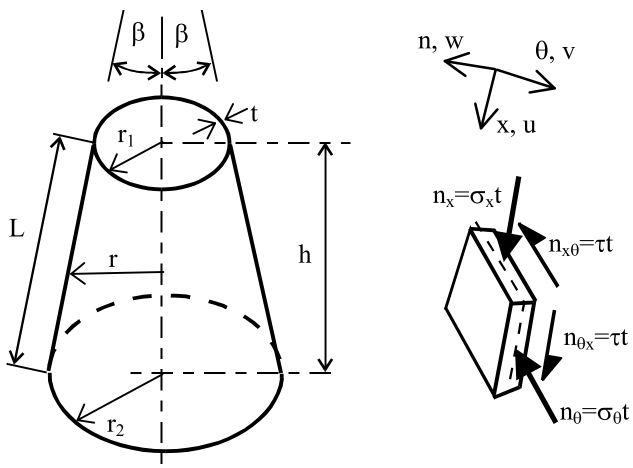


Figure D.9: Cone geometry, membrane stresses and stress resultants

D.4.1.2 Boundary conditions

(1) The following expressions should be used only for shells with boundary conditions BC 1 or BC 2 at both edges (see 5.2.2 and 8.3), with no distinction made between them. They should not be used for a shell in which any boundary condition is BC 3.

(2) The rules in this clause D.3 should be used only for the following two radial displacement restraint boundary conditions, at either end of the cone:

“cylinder condition” $w = 0$;
“ring condition” $u \sin\beta + w \cos\beta = 0$.

D.4.1.3 Geometry

(1) Only truncated cones of uniform wall thickness and with apex half angle $\beta \leq 65°$ (see figure D.9) are covered by the following rules.

D.4.2 Design buckling stresses

D.4.2.1 Equivalent cylinder

(1) The design buckling stresses that are needed for the buckling strength verification according to 8.5 may be all be found by treating the conical shell as an equivalent cylinder of length ℓ_e and of radius r_e in which ℓ_e and r_e depend on the type of stress distribution in the conical shell.

D.4.2.2 Meridional compression

(1) For cones under meridional compression, the equivalent cylinder length ℓ_e should be taken as:

$$\ell_e = L \qquad \text{... (D.69)}$$

(2) The equivalent cylinder radius r_e should be taken as:

$$r_e = \frac{r}{\cos\beta} \qquad \text{... (D.70)}$$

D.4.2.3 Circumferential (hoop) compression

(1) For cones under circumferential compression, the equivalent cylinder length ℓ_e should be taken as:

$$\ell_e = L \qquad \text{... (D.71)}$$

(2) The equivalent cylinder radius r_e should be taken as:

$$r_e = \frac{(r_1 + r_2)}{2\cos\beta} \qquad \text{... (D.72)}$$

D.4.2.4 Uniform external pressure

(1) For cones under uniform external pressure q, that have either the boundary conditions BC1 at both ends or the boundary conditions BC2 at both ends, the following procedure may be used to produce a more economic design.

(2) The equivalent cylinder length ℓ_e should be taken as the lesser of:

$$\ell_e = L \qquad \text{... (D.73)}$$

and

$$\ell_e = \left(\frac{r_2}{\sin\beta}\right)(0{,}53 + 0{,}125\,\beta \qquad) \qquad \text{... (D.74)}$$

where the cone apex half angle β is measured in radians.

(3) For shorter cones, where the equivalent length ℓ_e is given by expression (D.73), the equivalent cylinder radius r_e should be taken as:

$$r_e = \frac{(0{,}55 r_1 + 0{,}45 r_2}{(\cos\beta} \qquad) \qquad \text{... (D.75)}$$

(4) For longer cones, where the equivalent length ℓ_e is given by expression (D.74), the equivalent cylinder radius r_e should be taken as:

$$r_e = 0{,}71\, r_2 \left(\frac{1 - 0{,}1\beta}{\cos\beta}\right) \quad \text{... (D.76)}$$

(5) The buckling strength verification should be based on the membrane stress:

$$\sigma_{\theta E} = q\left(\frac{r_e}{t}\right) \quad \text{... (D.77)}$$

in which q is the external pressure, and no account is taken of the meridional membrane stress induced by the external pressure.

D.4.2.5 Shear

(1) For cones under shear stress, the equivalent cylinder length ℓ_e should be taken as:

$$\ell_e = h \quad \text{... (D.78)}$$

(2) The equivalent cylinder radius r_e should be taken as:

$$r_e = \left[1 + \rho - \frac{1}{\rho}\right] r_1 \cos\beta \quad \text{... (D.79)}$$

in which:

$$\rho = \sqrt{\frac{r_1 + r_2}{2 r_1}} \quad \text{... (D.80)}$$

D.4.2.6 Uniform torsion

(1) For cones under membrane shear stress, where this is produced by uniform torsion (inducing a shear that varies linearly down the meridian), the following procedure may be used to produce a more economic design, provided $\rho \le 0{,}8$ and the boundary conditions are BC2 at both ends.

(2) The equivalent cylinder length ℓ_e should be taken as:

$$\ell_e = L \quad \text{... (D.81)}$$

(3) The equivalent cylinder radius r_e should be taken as:

$$r_e = r_1 \cos\beta \left(1 - \rho^{2{,}5}\right)^{0{,}4} \quad \text{... (D.82)}$$

in which:

$$\rho = \frac{L \sin\beta}{r_2} \quad \text{... (D.83)}$$

D.4.3 Buckling strength verification

D.4.3.1 Meridional compression

(1) The buckling design check should be carried out at that point of the cone where the combination of acting design meridional stress and design buckling stress according to D.3.2.2 is most critical.

(2) In the case of meridional compression caused by a constant axial force on a truncated cone, both the small radius r_1 and the large radius r_2 should be considered as possibly the location of the most critical position.

(3) In the case of meridional compression caused by a constant global bending moment on the cone, the small radius r_1 should be taken as the most critical.

(4) The design buckling stress should be determined for the equivalent cylinder according to D.1.2.

D.4.3.2 Circumferential (hoop) compression and uniform external pressure

(1) Where the circumferential compression is caused by uniform external pressure, the buckling design check should be carried out using the acting design circumferential stress $\sigma_{\theta E,d}$ determined using expression D.77 and the design buckling stress $\sigma_{\theta R,d}$ according to D.3.2.1 and D.3.2.3.

(2) Where the circumferential compression is caused by actions other than uniform external pressure, the calculated stress distribution $\sigma_{\theta E}(x)$ should be replaced by a stress distribution $\sigma_{\theta E,env}(x)$ that everywhere exceeds the calculated value, but which would arise from a fictitious uniform external pressure. The buckling design check should then be carried out as in paragraph (1), but using $\sigma_{\theta E,env}$ instead of $\sigma_{\theta E}$.

(3) The design buckling stress should be determined for the equivalent cylinder according to D.1.3.

D.4.3.3 Shear and uniform torsion

(1) In the case of shear caused by a constant global torque on the cone, the buckling design check should be carried out using the acting design shear stress $\tau_{E,d}$ at the point with $r = r_e \cos\beta$ and the design buckling stress $\tau_{R,d}$ according to D.3.2.1 and D.3.2.4.

(2) Where the shear is caused by actions other than a constant global torque (such as a global shear force on the cone), the calculated stress distribution $\tau_E(x)$ should be replaced by a fictitious stress distribution $\tau_{E,env}(x)$ that everywhere exceeds the calculated value, but which would arise from a fictitious global torque. The buckling design check should then be carried out as in paragraph (1), but using $\tau_{E,env}$ instead of τ_E.

(3) The design buckling stress $\tau_{R,d}$ should be determined for the equivalent cylinder according to D.1.4.

FINAL DRAFT
prEN 1993-1-8

EUROPEAN STANDARD

NORME EUROPÉENNE

EUROPÄISCHE NORM

December 2003

ICS

Will supersede ENV 1993-1-1:1992

English version

Eurocode 3: Design of steel structures - Part 1-8: Design of joints

Eurocode 3: Calcul des structures en acier - Partie 1-8: Calcul des assemblages

Eurocode 3: Bemessung und Konstruktion von Stahlbauten - Teil 1-8: Bemessung von Anschlüssen

This draft European Standard is submitted to CEN members for formal vote. 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.

This draft European Standard was established by CEN in three official versions (English, French, German). A version in any other language made by translation under the responsibility of a CEN member into its own language and notified to the Management Centre has the same status as the official versions.

CEN members are the national standards bodies of Austria, Belgium, Czech Republic, Denmark, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Luxembourg, Malta, Netherlands, Norway, Portugal, Slovakia, Spain, Sweden, Switzerland and United Kingdom.

Warning : This document is not a European Standard. It is distributed for review and comments. It is subject to change without notice and shall not be referred to as a European Standard.

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

Management Centre: rue de Stassart, 36 B-1050 Brussels

Ref. No. prEN 1993-1-8:2003 E

Content

Foreword

This document (prEN 1993-1-8: 2003) has been prepared by Technical Committee CEN/TC 250 "Structural Eurocodes", the secretariat of which is held be BSI.

This document is currently submitted to the Formal Vote.

This document will supersede ENV 1993-1-1.

National Annex for EN1993-1-8

This standard gives alternative procedures, values and recommendations with notes indicating where national choices may have to be made. The National Standard implementing EN 1993-1-8 should have a National Annex containing all Nationally Determined Parameters for the design of steel structures to be constructed in the relevant country.

National choice is allowed in EN 1993-1-8 through:

- 2.2(2)
- 2.8 (Group 6: Rivets)
- 3.4.2(3)
- 6.2.7.2(9)

1 Introduction

1.1 Scope

(1) This part of EN 1993 gives design methods for the design of joints subject to predominantly static loading using steel grades S235, S275, S355 and S460.

1.2 Distinction between Principles and Application Rules

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

1.3 Terms and definitions

(1) The following terms and definitions apply:

– **basic component** (of a joint): Part of a joint that makes a contribution to one or more of its structural properties.

– **connection**: Location at which two or more elements meet. For design purposes it is the assembly of the basic components required to represent the behaviour during the transfer of the relevant internal forces and moments at the connection.

– **connected member**: Any member that is joined to a supporting member or element.

– **joint**: Zone where two or more members are interconnected. For design purposes it is the assembly of all the basic components required to represent the behaviour during the transfer of the relevant internal forces and moments between the connected members. A beam-to-column joint consists of a web panel and either one connection (single sided joint configuration) or two connections (double sided joint configuration), see Figure 1.1.

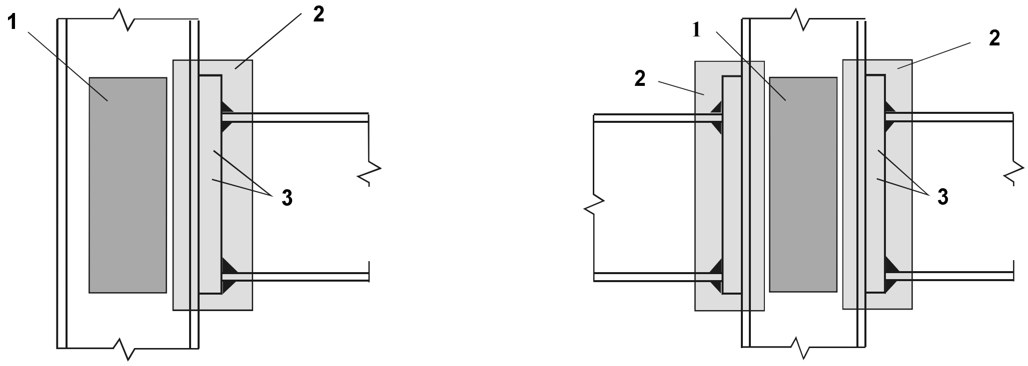
– **joint configuration**: Type or layout of the joint or joints in a zone within which the axes of two or more inter-connected members intersect, see Figure 1.2.

– **rotational capacity**: The angle through which the joint can rotate without failing.

– **rotational stiffness**: The moment required to produce unit rotation in a joint.

– **structural properties** (of a joint): Resistance to internal forces and moments in the connected members, rotational stiffness and rotation capacity.

– **uniplanar joint**: In a lattice structure a uniplanar joint connects members that are situated in a single plane.



Joint = web panel in shear + connection

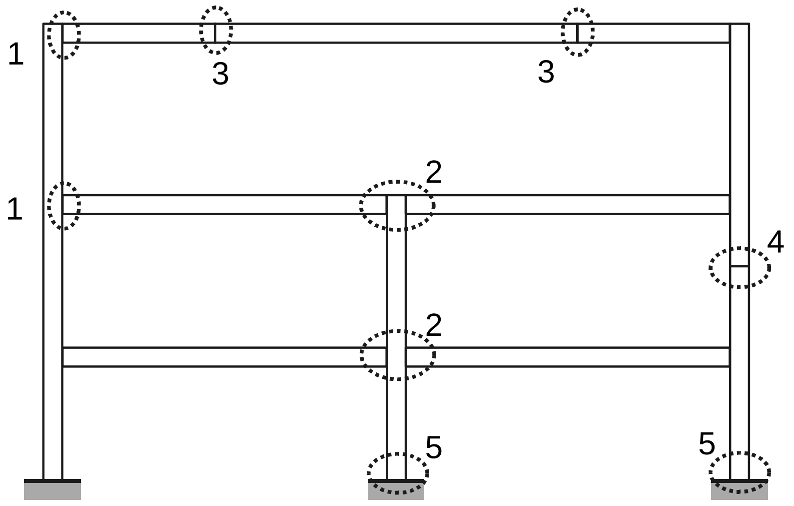
a) Single-sided joint configuration

Left joint = web panel in shear + left connection
Right joint = web panel in shear + right connection

b) Double-sided joint configuration

1 web panel in shear
2 connection
3 components (e.g. bolts, endplate)

Figure 1.1: Parts of a beam-to-column joint configuration



1 Single-sided beam-to-column joint configuration;

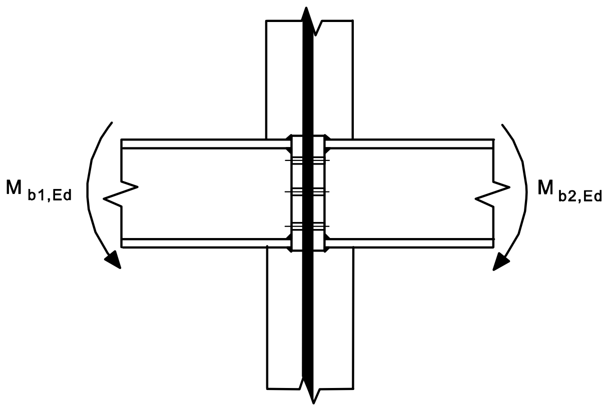
2 Double-sided beam-to-column joint configuration;

3 Beam splice;

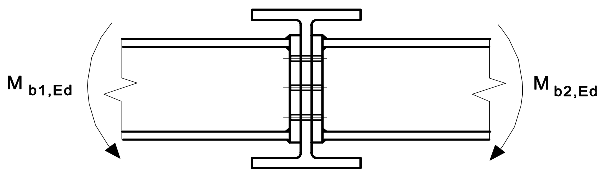
4 Column splice;

5 Column base.

a) Major-axis joint configurations



Double-sided beam-to-column joint configuration



Double-sided beam-to-beam joint configuration

b) Minor-axis joint configurations (to be used only for balanced moments $M_{b1,Ed} = M_{b2,Ed}$)

Figure 1.2: Joint configurations

1.4 Symbols

(1) The following symbols are used in this Standard:

d is the nominal bolt diameter, the diameter of the pin or the diameter of the fastener;

d_0 is the hole diameter for a bolt, a rivet or a pin ;

$d_{o,t}$ is the hole size for the tension face, generally the hole diameter, but for horizontally slotted holes the slot length should be used;

$d_{o,v}$ is the hole size for the shear face, generally the hole diameter, but for vertically slotted holes the slot length should be used;

d_c is the clear depth of the column web;

d_m is the mean of the across points and across flats dimensions of the bolt head or the nut, whichever is smaller;

$f_{H,Rd}$ is the design value of the Hertz pressure;

f_{ur} is the specified ultimate tensile strength of the rivet;

e_1 is the end distance from the centre of a fastener hole to the adjacent end of any part, measured in the direction of load transfer, see Figure 3.1;

e_2 is the edge distance from the centre of a fastener hole to the adjacent edge of any part, measured at right angles to the direction of load transfer, see Figure 3.1;

e_3 is the distance from the axis of a slotted hole to the adjacent end or edge of any part, see Figure 3.1;

e_4 is the distance from the centre of the end radius of a slotted hole to the adjacent end or edge of any part, see Figure 3.1;

ℓ_{eff} is the effective length of fillet weld;

n is the number of the friction surfaces or the number of fastener holes on the shear face;

p_1 is the spacing between centres of fasteners in a line in the direction of load transfer, see Figure 3.1;

$p_{1,0}$ is the spacing between centres of fasteners in an outer line in the direction of load transfer, see Figure 3.1;

$p_{1,i}$ is the spacing between centres of fasteners in an inner line in the direction of load transfer, see Figure 3.1;

p_2 is the spacing measured perpendicular to the load transfer direction between adjacent lines of fasteners, see Figure 3.1;

r is the bolt row number;

NOTE: In a bolted connection with more than one bolt-row in tension, the bolt-rows are numbered starting from the bolt-row furthest from the centre of compression.

s_s is the length of stiff bearing.

t_a is the thickness of the angle cleat.

t_{fc} is the thickness of the column flange;

t_p is the thickness of the plate under the bolt or the nut;

t_w is the thickness of the web or bracket;

t_{wc} is the thickness of the column web;

A is the gross cross-section area of bolt;

A_0 is the area of the rivet hole;

A_{vc} is the shear area of the column, see EN 1993-1-1;

A_s is the tensile stress area of the bolt or of the anchor bolt;

$A_{v,eff}$ is the effective shear area;

$B_{p,Rd}$ is the design punching shear resistance of the bolt head and the nut

E is the elastic modulus;

$F_{p,Cd}$ is the design preload force;

$F_{t,Ed}$ is the design tensile force per bolt for the ultimate limit state;

$F_{t,Rd}$ is the design tension resistance per bolt;

$F_{T,Rd}$ is the tension resistance of an equivalent T-stub flange;

$F_{v,Rd}$ is the design shear resistance per bolt;

$F_{b,Rd}$ is the design bearing resistance per bolt;

$F_{s,Rd,ser}$ is the design slip resistance per bolt at the serviceability limit state;

$F_{s,Rd}$ is the design slip resistance per bolt at the ultimate limit state;

$F_{v,Ed,ser}$ is the design shear force per bolt for the serviceability limit state;

$F_{v,Ed}$ is the design shear force per bolt for the ultimate limit state;

$M_{j,Rd}$ is the design moment resistance of a joint;

S_j is the rotational stiffness of a joint;

$S_{j,ini}$ is the initial rotational stiffness of a joint;

$V_{wp,Rd}$ is the plastic shear resistance of a column web panel;

z is the lever arm;

μ is the slip factor;

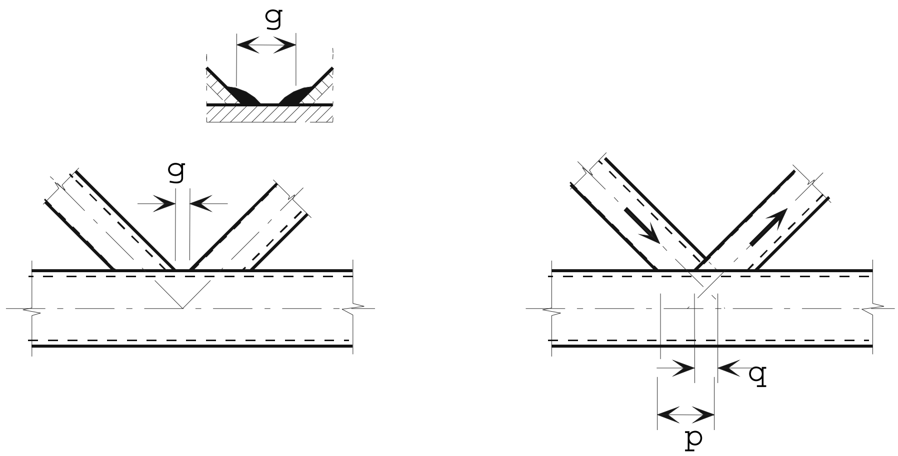
ϕ is the rotation of a joint.

(2) The following standard abbreviations are used in section 7:

CHS for "circular hollow section";

RHS for "rectangular hollow section", which in this context includes square hollow sections.

gap g overlap λ_{ov} = (q/p) x 100 %



(a) Definition of gap (b) Definition of overlap

Figure 1.3: Gap and overlap joints

(3) The following symbols are used in section 7:

A_i is the cross-sectional area of member i (i = 0, 1, 2 or 3);

A_v is the shear area of the chord;

$A_{v,eff}$ is the effective shear area of the chord;

L is the system length of a member;

$M_{ip,i,Rd}$ is the design value of the resistance of the joint, expressed in terms of the in-plane internal moment in member i (i = 0, 1, 2 or 3);

$M_{ip,i,Ed}$ is the design value of the in-plane internal moment in member i (i = 0, 1, 2 or 3);

$M_{op,i,Rd}$ is the design value of the resistance of the joint, expressed in terms of the out-of-plane internal moment in member i (i = 0, 1, 2 or 3);

$M_{op,i,Ed}$ is the design value of the out-of-plane internal moment in member i (i = 0, 1, 2 or 3);

$N_{i,Rd}$ is the design value of the resistance of the joint, expressed in terms of the internal axial force in member i (i = 0, 1, 2 or 3);

$N_{i,Ed}$ is the design value of the internal axial force in member i (i = 0, 1, 2 or 3);

$W_{e\ell,i}$ is the elastic section modulus of member i (i = 0, 1, 2 or 3);

$W_{p\ell,i}$ is the plastic section modulus of member i (i = 0, 1, 2 or 3);

b_i is the overall out-of-plane width of RHS member i (i = 0, 1, 2 or 3);

b_{eff} is the effective width for a brace member to chord connection;

$b_{e,ov}$ is the effective width for an overlapping brace to overlapped brace connection;

$b_{e,p}$ is the effective width for punching shear;

b_p is the width of a plate;

b_w is the effective width for the web of the chord;

d_i is the overall diameter of CHS member i (i = 0, 1, 2 or 3);

d_w is the depth of the web of an I or H section chord member;

e is the eccentricity of a joint;

f_b is the buckling strength of the chord side wall;

f_{yi} is the yield strength of member i (i = 0, 1, 2 or 3);

f_{y0} is the yield strength of a chord member;

g is the gap between the brace members in a K or N joint (negative values of g represent an overlap q); the gap g is measured along the length of the connecting face of the chord, between the toes of the adjacent brace members, see Figure 1.3(a);

h_i is the overall in-plane depth of the cross-section of member i (i = 0, 1, 2 or 3);

k is a factor defined in the relevant table, with subscript g, m, n or p;

ℓ is the buckling length of a member;

p is the length of the projected contact area of the overlapping brace member onto the face of the chord, in the absence of the overlapped brace member, see Figure 1.3(b);

q is the length of overlap, measured at the face of the chord, between the brace members in a K or N joint, see Figure 1.3(b);

r is the root radius of an I or H section or the corner radius of a rectangular hollow section;

t_f is the flange thickness of an I or H section;

t_i is the wall thickness of member i (i = 0, 1, 2 or 3);

t_p is the thickness of a plate;

t_w is the web thickness of an I or H section;

α is a factor defined in the relevant table;

θ_i is the included angle between brace member i and the chord (i = 1, 2 or 3);

κ is a factor defined where it occurs;

μ is a factor defined in the relevant table;

φ is the angle between the planes in a multiplanar joint.

(4) The integer subscripts used in section 7 are defined as follows:

i is an integer subscript used to designate a member of a joint, $i = 0$ denoting a chord and i = 1, 2 or 3 the brace members. In joints with two brace members, $i = 1$ normally denotes the compression brace and $i = 2$ the tension brace, see Figure 1.4(b). For a single brace $i = 1$ whether it is subject to compression or tension, see Figure 1.4(a);

i and j are integer subscripts used in overlap type joints, i to denote the overlapping brace member and j to denote the overlapped brace member, see Figure 1.4(c).

(5) The stress ratios used in section 7 are defined as follows:

n is the ratio $(\sigma_{0,Ed}/f_{y0})/\gamma_{M5}$ (used for RHS chords);

n_p is the ratio $(\sigma_{p,Ed}/f_{y0})/\gamma_{M5}$ (used for CHS chords);

$\sigma_{0,Ed}$ is the maximum compressive stress in the chord at a joint;

$\sigma_{p,Ed}$ is the value of $\sigma_{0,Ed}$ excluding the stress due to the components parallel to the chord axis of the axial forces in the braces at that joint, see Figure 1.4.

(6) The geometric ratios used in section 7 are defined as follows:

β is the ratio of the mean diameter or width of the brace members, to that of the chord:

- for T, Y and X joints:

$$\frac{d_1}{d_0} \; ; \; \frac{d_1}{b_0} \text{ or } \frac{b_1}{b_0}$$

- for K and N joints:

$$\frac{d_1 + d_2}{2\, d_0} \; ; \; \frac{d_1 + d_2}{2\, b_0} \text{ or } \frac{b_1 + b_2 + h_1 + h_2}{4\, b_0}$$

- for KT joints:

$$\frac{d_1 + d_2 + d_3}{3\, d_0} \; ; \; \frac{d_1 + d_2 + d_3}{3\, b_0} \text{ or } \frac{b_1 + b_2 + b_3 + h_1 + h_2 + h_3}{6\, b_0}$$

β_p is the ratio b_i/b_p;

γ is the ratio of the chord width or diameter to twice its wall thickness:

$$\frac{d_0}{2\, t_0} \; ; \; \frac{b_0}{2\, t_0} \text{ or } \frac{b_0}{2\, t_f}$$

η is the ratio of the brace member depth to the chord diameter or width:

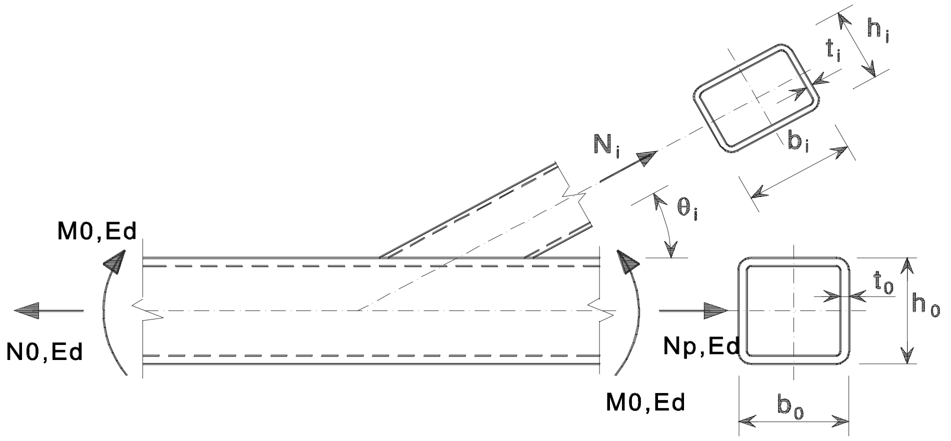
$$\frac{h_i}{d_0} \text{ or } \frac{h_i}{b_0}$$

η_p is the ratio h_i/b_p;

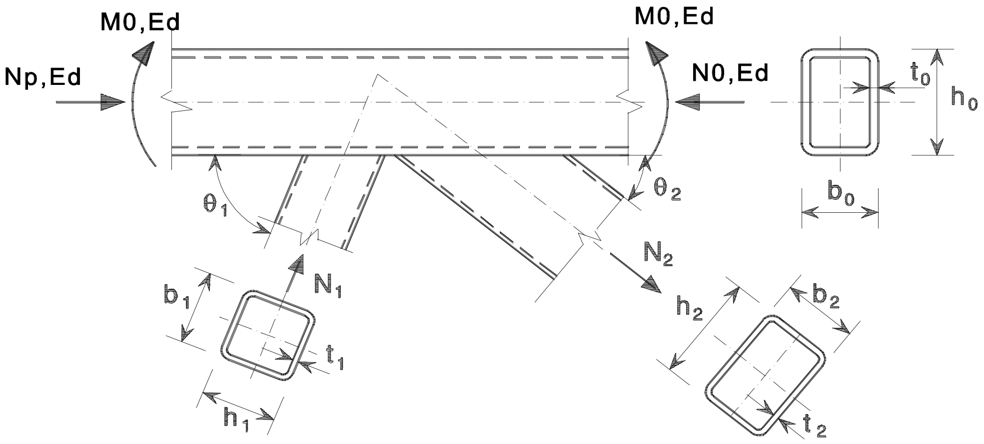
λ_{ov} is the overlap ratio, expressed as a percentage (λ_{ov} = (q/p) x 100%) as shown in figure 1.3(b).

(7) Other symbols are specified in appropriate clauses when they are used.

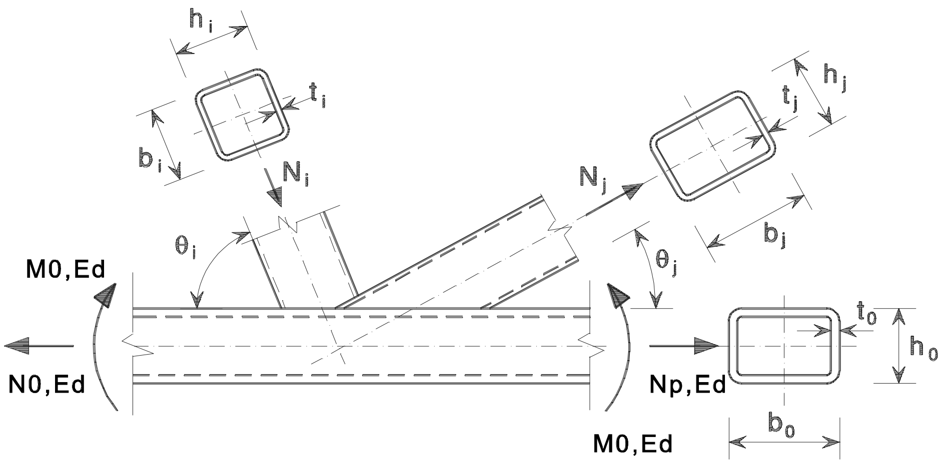
NOTE: Symbols for circular sections are given in Table 7.2.



a) Joint with single brace member



b) Gap joint with two brace members



c) Overlap joint with two brace members

Figure 1.4: Dimensions and other parameters at a hollow section lattice girder joint

2 Basis of design

2.1 Assumptions

(1) The design methods given in this part of EN 1993 assume that the standard of construction is as specified in the execution standards given in 2.8 and that the construction materials and products used are those specified in EN 1993 or in the relevant material and product specifications.

2.2 General requirements

(1) All joints shall have a design resistance such that the structure is capable of satisfying all the basic design requirements given in this Standard and in EN 1993-1-1.

(2) The partial safety factors γ_M for joints are given in Table 2.1.

Table 2.1: Partial safety factors for joints

Resistance of members and cross-sections	γ_{M0} , γ_{M1} and γ_{M2} see EN 1993-1-1
Resistance of bolts	γ_{M2}
Resistance of rivets	
Resistance of pins	
Resistance of welds	
Resistance of plates in bearing	
Slip resistance - for hybrid connections or connections under fatigue loading - for other design situations	 γ_{M3} γ_{M3}
Bearing resistance of an injection bolt	γ_{M4}
Resistance of joints in hollow section lattice girder	γ_{M5}
Resistance of pins at serviceability limit state	$\gamma_{M6,ser}$
Preload of high strength bolts	γ_{M7}
Resistance of concrete	γ_c see EN 1992

NOTE: Numerical values for γ_M may be defined in the National Annex. Recommended values are as follows: $\gamma_{M2} = 1,25$; $\gamma_{M3} = 1,25$ for hybrid connections or connections under fatigue loading and $\gamma_{M3} = 1,1$ for other design situations; $\gamma_{M4} = 1,0$; $\gamma_{M5} = 1,0$; $\gamma_{M6,ser} = 1,0$; $\gamma_{M7} = 1,1$.

(3) Joints subject to fatigue should also satisfy the principles given in EN 1993-1-9.

2.3 Applied forces and moments

(1) The forces and moments applied to joints at the ultimate limit state shall be determined according to the principles in EN 1993-1-1.

2.4 Resistance of joints

(1) The resistance of a joint shall be determined on the basis of the resistances of its basic components.

(2) Linear-elastic or elastic-plastic analysis may be used in the design of joints.

(3) Where fasteners with different stiffenesses are used to carry a shear load the fasteners with the highest stiffness should be designed to carry the design load. An exception to this design method is given in 3.9.3.

2.5 Design assumptions

(1) Joints shall be designed on the basis of a realistic assumption of the distribution of internal forces and moments. The following assumptions should be used to determine the distribution of forces:

(a) the internal forces and moments assumed in the analysis are in equilibrium with the forces and moments applied to the joints,

(b) each element in the joint is capable of resisting the internal forces and moments,

(c) the deformations implied by this distribution do not exceed the deformation capacity of the fasteners or welds and the connected parts,

(d) the assumed distribution of internal forces shall be realistic with regard to relative stiffnesses within the joint,

(e) the deformations assumed in any design model based on elastic-plastic analysis are based on rigid body rotations and/or in-plane deformations which are physically possible, and

(f) any model used is in compliance with the evaluation of test results (see EN 1990).

(2) The application rules given in this part satisfy 2.5(1).

2.6 Joints loaded in shear subject to impact, vibration and/or load reversal

(1) Where a joint loaded in shear is subject to impact or significant vibration one of the following jointing methods should be used:

- welding
- bolts with locking devices
- preloaded bolts
- injection bolts
- other types of bolt which effectively prevent movement of the connected parts
- rivets.

(2) Where slip is not acceptable in a joint (because it is subject to reversal of shear load or for any other reason), preloaded bolts in a Category B or C connection (see 3.4), fit bolts (see 3.6.1), rivets or welding should be used.

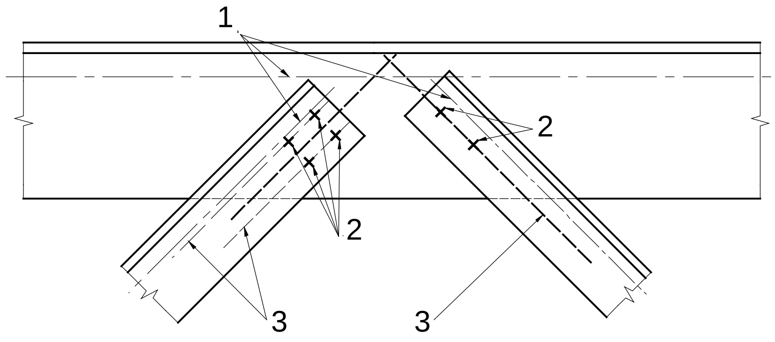
(3) For wind and/or stability bracings, bolts in Category A connections (see 3.4) may be used.

2.7 Eccentricity at intersections

(1) Where there is eccentricity at intersections, the joints and members should be designed for the resulting moments and forces, except in the case of particular types of structures where it has been demonstrated that it is not necessary, see 5.1.5.

(2) In the case of joints of angles or tees attached by either a single line of bolts or two lines of bolts any possible eccentricity should be taken into account in accordance with 2.7(1). In-plane and out-of-plane eccentricities should be determined by considering the relative positions of the centroidal axis of the member and of the setting out line in the plane of the connection (see Figure 2.1). For a single angle in tension connected by bolts on one leg the simplified design method given in 3.10.3 may be used.

NOTE: The effect of eccentricity on angles used as web members in compression is given in EN 1993-1-1, Annex BB 1.2.



1 Centroidal axes
2 Fasteners
3 Setting out lines

Figure 2.1: Setting out lines

2.8 References

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

Reference Standards, Group 1: Weldable structural steels

prEN 10025:2001 Hot rolled products of non-alloy structural steels - Technical delivery conditions

Reference Standards, Group 2: Tolerances, dimensions and technical delivery conditions

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

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

EN 10051:1991 Continuously hot-rolled uncoated plate, sheet and strip of non-alloy and alloy steels - Tolerances on dimensions and shape

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

EN 10056-1:1995 Structural steel equal and unequal leg angles - Part 1: Dimensions

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

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

Reference Standards, Group 3: Structural hollow sections

EN 10219-1:1997 Cold formed welded structural hollow sections of non-alloy and fine grain steels - Part 1: Technical delivery requirements

EN 10219-2:1997 Cold formed welded structural hollow sections of non-alloy and fine grain steels - Part 2: Tolerances, dimensions and sectional properties

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

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

Reference Standards, Group 4: Bolts, nuts and washers

EN 14399-1:2002	High strength structural bolting for preloading - Part 1 : General Requirements
EN 14399-2:2002	High strength structural bolting for preloading - Part 2 : Suitability Test for preloading
EN 14399-3:2002	High strength structural bolting for preloading - Part 3 : System HR -Hexagon bolt and nut assemblies
EN 14399-4:2002	High strength structural bolting for preloading - Part 4 : System HV -Hexagon bolt and nut assemblies
EN 14399-5:2002	High strength structural bolting for preloading - Part 5 : Plain washers for system HR
EN 14399-6:2002	High strength structural bolting for preloading - Part 6 : Plain chamfered washers for systems HR and HV
EN ISO 898-1:1999	Mechanical properties of fasteners made of carbon steel and alloy steel - Part 1: Bolts, screws and studs (ISO 898-1:1999)
EN 20898-2:1993	Mechanical properties of fasteners - Part 2: Nuts with special proof load values - Coarse thread (ISO 898-2:1992)
EN ISO 2320:1997	Prevailing torque type steel hexagon nuts - Mechanical and performance requirements (ISO 2320:1997)
EN ISO 4014:2001	Hexagon head bolts - Product grades A and B (ISO 4014:1999)
EN ISO 4016:2001	Hexagon head bolts - Product grade C (ISO 4016:1999)
EN ISO 4017:2001	Hexagon head screws - Product grades A and B (ISO 4017:1999)
EN ISO 4018:2001	Hexagon head screws - Product grade C (ISO 4018:1999)
EN ISO 4032:2001	Hexagon nuts, style 1 - Product grades A and B (ISO 4032:1999)
EN ISO 4033:2001	Hexagon nuts, style 2 - Product grades A and B (ISO 4033:1999)
EN ISO 4034:2001	Hexagon nuts - Product grade C (ISO 4034:1999)
EN ISO 7040:1997	Prevailing torque hexagon nuts (with non-metallic insert), style 1 - Property classes 5, 8 and 10
EN ISO 7042:1997	Prevailing torque all-metal hexagon nuts, style 2 - Property classes 5, 8, 10 and 12
EN ISO 7719:1997	Prevailing torque type all-metal hexagon nuts, style 1 - Property classes 5, 8 and 10
ISO 286- 2:1988	ISO system of limits and fits - Part 2: Tables of standard tolerance grades and limit deviations for hole and shafts
ISO 1891:1979	Bolts, screws, nuts and accessories - Terminology and nomenclature - Trilingual edition
EN ISO 7089:2000	Plain washers- Nominal series- Product grade A
EN ISO 7090:2000	Plain washers, chamfered - Normal series - Product grade A
EN ISO 7091:2000	Plain washers - Normal series - Product grade C
EN ISO 10511:1997	Prevailing torque type hexagon thin nuts (with non-metallic insert)
EN ISO 10512:1997	Prevailing torque type hexagon nuts thin nuts, style 1, with metric fine pitch thread - Property classes 6, 8 and 10
EN ISO 10513:1997	Prevailing torque type all-metal hexagon nuts, style 2, with metric fine pitch thread - Property classes 8, 10 and 12

Reference Standards, Group 5: Welding consumable and welding

EN 12345:1998	Welding-Multilingual terms for welded joints with illustrations. September 1998.
EN ISO 14555:1995	Welding-Arc stud welding of metallic materials. May 1995
Pr EN ISO 13918:1997	Welding-Studs for arc stud welding-January 1997

EN 288-3:1992	Specification and approval of welding procedures for metallic materials. Part 3: Welding procedure tests for arc welding of steels. 1992
Pr EN ISO 5817:2000	Arc-welded joints in steel - Guidance for quality levels for imperfections

Reference Standards, Group 6: Rivets

NOTE: Reference should be given in the National Annex.

Reference Standard, Group 7: Execution of steel structures

EN 1090	Requirements for the execution of steel structures

3 Connections made with bolts, rivets or pins

3.1 Bolts, nuts and washers

3.1.1 General

(1) All bolts, nuts and washers should comply with 2.8 Reference Standards: Group 4.

(2) The rules in this Standard are valid for the bolt classes given in Table 3.1.

(3) The yield strength f_{yb} and the ultimate tensile strength f_{ub} for bolt classes 4.6, 5.6, 6.8, 8.8 and 10.9 are given in Table 3.1. These values should be adopted as characteristic values in design calculations.

Table 3.1: Nominal values of the yield strength f_{yb} and the ultimate tensile strength f_{ub} for bolts

Bolt class	4.6	5.6	6.8	8.8	10.9
f_{yb} (N/mm^2)	240	300	480	640	900
f_{ub} (N/mm^2)	400	500	600	800	1000

3.1.2 Preloaded bolts

(1) Only bolt assemblies of classes 8.8 and 10.9 conforming to the requirements given in 2.8 Reference Standards: Group 4 for High Strength Structural Bolting with controlled tightening in accordance with the requirements in 2.8 Reference Standards: Group 7 may be used as preloaded bolts.

3.2 Rivets

(1) The material properties, dimensions and tolerances of steel rivets should comply with the requirements given in 2.8 Reference Standards: Group 6.

3.3 Anchor bolts

(1) The following materials may be used for anchor bolts:

- Steel grades conforming to 2.8 Reference Standards: Group 1;
- Steel grades conforming to 2.8 Reference Standards: Group 4;
- Steel grades used for reinforcing bars conforming to EN 10080,

provided that the nominal yield strength does not exceed 640 N/mm^2 when the anchor bolts are required to act in shear and not more than 900 N/mm^2 otherwise.

3.4 Categories of bolted connections

3.4.1 Shear connections

(1) Bolted connections loaded in shear should be designed as one of the following:

a) **Category A: Bearing type**
In this category bolts from class 4.6 up to and including class 10.9 should be used. No preloading and special provisions for contact surfaces are required. The design ultimate shear load should not exceed the design shear resistance, obtained from 3.6, nor the design bearing resistance, obtained from 3.6 and 3.7.

b) **Category B: Slip-resistant at serviceability limit state**
In this category preloaded bolts in accordance with 3.1.2(1) should be used. Slip should not occur at the serviceability limit state. The design serviceability shear load should not exceed the design slip resistance, obtained from 3.9. The design ultimate shear load should not exceed the design shear resistance, obtained from 3.6, nor the design bearing resistance, obtained from 3.6 and 3.7.

c) **Category C: Slip-resistant at ultimate limit state**
In this category preloaded bolts in accordance with 3.1.2(1) should be used. Slip should not occur at the ultimate limit state. The design ultimate shear load should not exceed the design slip resistance, obtained from 3.9, nor the design bearing resistance, obtained from 3.6 and 3.7. In addition for a connection in tension, the design plastic resistance of the net cross-section at bolt holes $N_{net,Rd}$, (see 6.2 of EN 1993-1-1), should be checked, at the ultimate limit state.

The design checks for these connections are summarised in Table 3.2.

3.4.2 Tension connections

(1) Bolted connection loaded in tension should be designed as one of the following:

a) **Category D: non-preloaded**
In this category bolts from class 4.6 up to and including class 10.9 should be used. No preloading is required. This category should not be used where the connections are frequently subjected to variations of tensile loading. However, they may be used in connections designed to resist normal wind loads.

b) **Category E: preloaded**
In this category preloaded 8.8 and 10.9 bolts with controlled tightening in conformity with 2.8 Reference Standards: Group 7 should be used.

The design checks for these connections are summarised in Table 3.2.

Table 3.2: Categories of bolted connections

Category	Criteria	Remarks
	Shear connections	
A bearing type	$F_{v,Ed} \leq F_{v,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$	No preloading required. Bolt classes from 4.6 to 10.9 may be used.
B slip-resistant at serviceability	$F_{v,Ed.ser} \leq F_{s,Rd,ser}$ $F_{v,Ed} \leq F_{v,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$	Preloaded 8.8 or 10.9 bolts should be used. For slip resistance at serviceability see 3.9.
C slip-resistant at ultimate	$F_{v,Ed} \leq F_{s,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$ $F_{v,Ed} \leq N_{net,Rd}$	Preloaded 8.8 or 10.9 bolts should be used. For slip resistance at ultimate see 3.9. $N_{net,Rd}$ see EN 1993-1-1
	Tension connections	
D non-preloaded	$F_{t,Ed} \leq F_{t,Rd}$ $F_{t,Ed} \leq B_{p,Rd}$	No preloading required. Bolt classes from 4.6 to 10.9 may be used. $B_{p,Rd}$ see Table 3.4.
E preloaded	$F_{t,Ed} \leq F_{t,Rd}$ $F_{t,Ed} \leq B_{p,Rd}$	Preloaded 8.8 or 10.9 bolts should be used. $B_{p,Rd}$ see Table 3.4.
The design tensile force $F_{t,Ed}$ should include any force due to prying action, see 3.11. Bolts subjected to both shear force and tensile force should also satisfy the criteria given in Table 3.4.		

NOTE: When the preload is not explicitly used in the design calculations for shear resistances but is required for execution purposes or as a quality measure (e.g. fordurability) then the level of preload can be specified in the National Annex.

3.5 Positioning of holes for bolts and rivets

(1) Minimum and maximum spacing and end and edge distances for bolts and rivets are given in Table 3.3.

(2) Minimum and maximum spacing, end and edge distances for structures subjected to fatigue, see EN 1993-1-9.

Table 3.3: Minimum and maximum spacing, end and edge distances

Distances and spacings, see Figure 3.1	Minimum	Maximum[1) 2) 3)]		
		Structures made from steels conforming to EN 10025 except steels conforming to EN 10025-5		Structures made from steels conforming to EN 10025-5
		Steel exposed to the weather or other corrosive influences	Steel not exposed to the weather or other corrosive influences	Steel used unprotected
End distance e_1	$1{,}2d_0$	$4t + 40$ mm		The larger of $8t$ or 125 mm
Edge distance e_2	$1{,}2d_0$	$4t + 40$ mm		The larger of $8t$ or 125 mm
Distance e_3 in slotted holes	$1{,}5d_0$ [4)]			
Distance e_4 in slotted holes	$1{,}5d_0$ [4)]			
Spacing p_1	$2{,}2d_0$	The smaller of $14t$ or 200 mm	The smaller of $14t$ or 200 mm	The smaller of $14t_{min}$ or 175 mm
Spacing $p_{1,0}$		The smaller of $14t$ or 200 mm		
Spacing $p_{1,i}$		The smaller of $28t$ or 400 mm		
Spacing p_2 [5)]	$2{,}4d_0$	The smaller of $14t$ or 200 mm	The smaller of $14t$ or 200 mm	The smaller of $14t_{min}$ or 175 mm

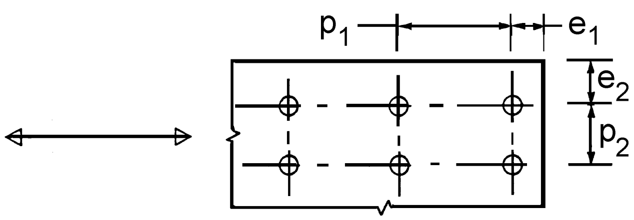
1) Maximum values for spacings, edge and end distances are unlimited, except in the following cases:
- for compression members in order to avoid local buckling and to prevent corrosion in exposed members and;
- for exposed tension members to prevent corrosion.

2) The local buckling resistance of the plate in compression between the fasteners should be calculated according to EN 1993-1-1 using $0{,}6\,p_i$ as buckling length. Local buckling between the fasteners need not to be checked if p_1/t is smaller than $9\,\varepsilon$. The edge distance should not exceed the local buckling requirements for an outstand element in the compression members, see EN 1993-1-1. The end distance is not affected by this requirement.

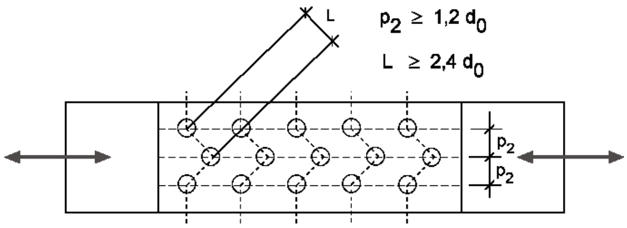
3) t is the thickness of the thinner outer connected part.

4) The dimensional limits for slotted holes are given in 2.8 Reference Standards: Group 7.

5) For staggered rows of fasteners a minimum line spacing of $p_2 = 1{,}2d_0$ may be used, provided that the minimum distance, L, between any two fasteners is greater than $2{,}4d_0$, see Figure 3.1b).

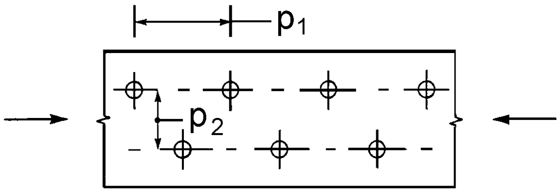


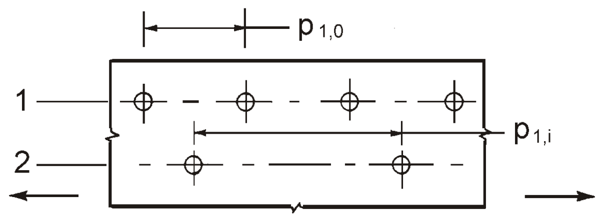
a) Symbols for spacing of fasteners



Staggered Rows of fasteners

b) Symbols for staggered spacing



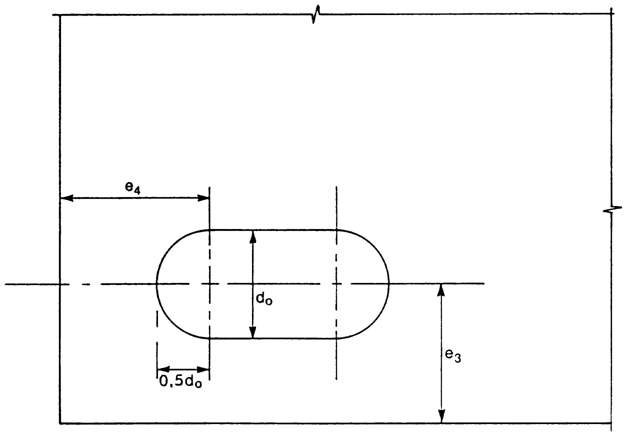


$p_1 \leq 14\,t$ and ≤ 200 mm $\quad$ $p_2 \leq 14\,t$ and ≤ 200 mm $\quad$ $p_{1,0} \leq 14\,t$ and ≤ 200 mm $\quad$ $p_{1,i} \leq 28\,t$ and ≤ 400 mm

1 outer row $\quad$ *2 inner row*

c) Staggered spacing – compression

d) Spacing in tension members



e) End and edge distances for slotted holes

Figure 3.1: Symbols for end and edge distances and spacing of fasteners

3.6 Design resistance of individual fasteners

3.6.1 Bolts and rivets

(1) The design resistance for an individual fastener subjected to shear and/or tension is given in Table 3.4.

(2) For preloaded bolts in accordance with 3.1.2(1) the design preload, $F_{p,Cd}$,to be used in design calculations should be taken as:

$$F_{p,Cd} = 0{,}7\, f_{ub}\, A_s / \gamma_{M7} \qquad \text{... (3.1)}$$

NOTE: Where the preload is not used in design calculations the guidance given in the note to Table 3.2 should be followed.

(3) The design resistances for tension and for shear through the threaded portion of a bolt given in Table 3.4 should only be used for bolts manufactured in conformity with 2.8 Reference Standard: Group 4. For bolts with cut threads, such as anchor bolts or tie rods fabricated from round steel bars where the threads comply with EN1090, the relevant values from Table 3.4 should be used. For bolts with cut threads where the threads do not comply with EN1090 the relevant values from Table 3.4 should be multiplied by a factor of 0,85.

(4) The design shear resistance $F_{v,Rd}$ given in Table 3.4 should only be used where the bolts are used in holes with nominal clearances not exceeding those for normal holes as specified in 2.8 Reference Standards: Group 7.

(5) M12 and M14 bolts may also be used in 2 mm clearance holes provided that the design resistance of the bolt group based on bearing is greater or equal to the design resistance of the bolt group based on bolt shear. In addition for class 4.8, 5.8, 6.8, 8.8 and 10.9 bolts the design shear resistance $F_{v,Rd}$ should be taken as 0,85 times the value given in Table 3.4.

(6) Fit bolts should be designed using the method for bolts in normal holes.

(7) The thread of a fit bolt should not be included in the shear plane.

(8) The length of the threaded portion of a fit bolt included in the bearing length should not exceed 1/3 of the thickness of the plate, see Figure 3.2.

(9) The hole tolerance used for fit bolts should be in accordance with 2.8 Reference Standards: Group 7.

(10) In single lap joints with only one bolt row, see Figure 3.3, the bolts should be provided with washers under both the head and the nut. The design bearing resistance $F_{b,Rd}$ for each bolt should be limited to:

$$F_{b,Rd} \leq 1{,}5\, f_u\, d\, t \,/\, \gamma_{M2} \qquad \text{... (3.2)}$$

NOTE: Single rivets should not be used in single lap joints.

(11) In the case of class 8.8 or 10.9 bolts, hardened washers should be used for single lap joints with only one bolt or one row of bolts.

(12) Where bolts or rivets transmitting load in shear and bearing pass through packing of total thickness t_p greater than one-third of the nominal diameter d, see Figure 3.4, the design shear resistance $F_{v,Rd}$ calculated as specified in Table 3.4, should be multiplying by a reduction factor β_p given by:

$$\beta_p = \frac{9d}{8d + 3t_p} \quad \text{but } \beta_p \leq 1 \qquad \text{... (3.3)}$$

(13) For double shear connections with packing on both sides of the splice, t_p should be taken as the thickness of the thicker packing.

(14) Riveted connections should be designed to transfer shear forces. If tension is present the design tensile force $F_{t.Ed}$ should not exceed the design tension resistance $F_{t,Rd}$ given in Table 3.4.

(15) For grade S 235 steel the "as driven" value of f_{ur} may be taken as 400 N/mm^2.

(16) As a general rule, the grip length of a rivet should not exceed 4,5d for hammer riveting and 6,5d for press riveting.

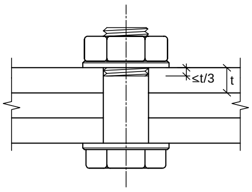


Figure 3.2: Threaded portion of the shank in the bearing length for fit bolts

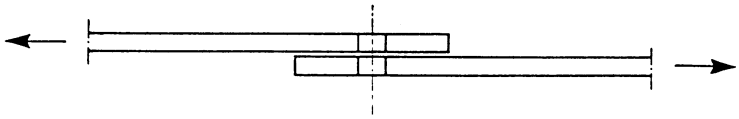


Figure 3.3: Single lap joint with one row of bolts

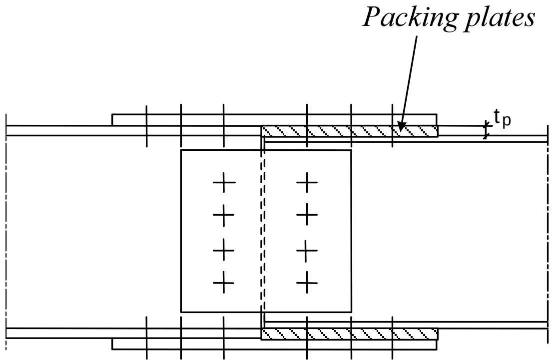


Figure 3.4: Fasteners through packings

Table 3.4: Design resistance for individual fasteners subjected to shear and/or tension

<table>
<tr><th>Failure mode</th><th>Bolts</th><th>Rivets</th></tr>
<tr><td>Shear resistance per shear plane</td><td>$F_{v,Rd} = \dfrac{\alpha_v \; f_{ub} \; A}{\gamma_{M2}}$
- where the shear plane passes through the threaded portion of the bolt (A is the tensile stress area of the bolt A_s):
- for classes 4.6, 5.6 and 8.8:
$\alpha_v = 0,6$
- for classes 4.8, 5.8, 6.8 and 10.9:
$\alpha_v = 0,5$
- where the shear plane passes through the unthreaded portion of the bolt (A is the gross cross section of the bolt): $\alpha_v = 0,6$</td><td>$F_{v,Rd} = \dfrac{0,6 \; f_{ur} \; A_0}{\gamma_{M2}}$</td></tr>
<tr><td>Bearing resistance [1), 2), 3)]</td><td colspan="2">$F_{b,Rd} = \dfrac{k_1 \, \alpha_b \, f_u \, d \, t}{\gamma_{M2}}$
where α_b is the smallest of α_d ; $\dfrac{f_{ub}}{f_u}$ or 1,0;
in the direction of load transfer:
- for end bolts: $\alpha_d = \dfrac{e_1}{3d_0}$; for inner bolts: $\alpha_d = \dfrac{p_1}{3d_0} - \dfrac{1}{4}$
perpendicular to the direction of load transfer:
- for edge bolts: k_1 is the smallest of $2,8\dfrac{e_2}{d_0} - 1,7$ or 2,5
- for inner bolts: k_1 is the smallest of $1,4\dfrac{p_2}{d_0} - 1,7$ or 2,5</td></tr>
<tr><td>Tension resistance [2)]</td><td>$F_{t,Rd} = \dfrac{k_2 \; f_{ub} \; A_s}{\gamma_{M2}}$
where $k_2 = 0,63$ for countersunk bolt,
otherwise $k_2 = 0,9$.</td><td>$F_{t,Rd} = \dfrac{0,6 \; f_{ur} \; A_0}{\gamma_{M2}}$</td></tr>
<tr><td>Punching shear resistance</td><td>$B_{p,Rd} = 0,6 \, \pi \, d_m \, t_p f_u / \gamma_{M2}$</td><td>No check needed</td></tr>
<tr><td>Combined shear and tension</td><td colspan="2">$\dfrac{F_{v,Ed}}{F_{v,Rd}} + \dfrac{F_{t,Ed}}{1,4 F_{t,Rd}} \leq 1,0$</td></tr>
</table>

1) The bearing resistance $F_{b,Rd}$ for bolts

- in oversized holes is 0,8 times the bearing resistance for bolts in normal holes.
- in slotted holes, where the longitudinal axis of the slotted hole is perpendicular to the direction of the force transfer, is 0,6 times the bearing resistance for bolts in round, normal holes.

2) For countersunk bolt:

- the bearing resistance $F_{b,Rd}$ should be based on a plate thickness t equal to the thickness of the connected plate minus half the depth of the countersinking.
- for the determination of the tension resistance $F_{t,Rd}$ the angle and depth of countersinking should conform with 2.8 Reference Standards: Group 4, otherwise the tension resistance $F_{t,Rd}$ should be adjusted accordingly.

3) When the load on a bolt is not parallel to the edge, the bearing resistance may be verified separately for the bolt load components parallel and normal to the end.

3.6.2 Injection bolts

3.6.2.1 General

(1) Injection bolts may be used as an alternative to ordinary bolts and rivets for category A, B and C connections specified in 3.4.

(2) Fabrication and erection details for injection bolts are given in 2.8 Reference Standards: Group 7.

3.6.2.2 Design resistance

(1) The design method given in 3.6.2.2(2) to 3.6.2.2(6) should be used for connections with injection bolts of class 8.8 or 10.9. Bolt assemblies should conform with the requirements given in 2.8 Reference Standards: Group 4, but see 3.6.2.2(3) for when preloaded bolts are used.

(2) The design ultimate shear load of any bolt in a Category A connection shall not exceed the smaller of the following: the design shear resistance of the bolt as obtained from 3.6 and 3.7; the design bearing resistance of the resin as obtained from 3.6.2.2(5).

(3) Preloaded injection bolts should be used for category B and C connections, for which preloaded bolt assemblies in accordance with 3.1.2(1) should be used.

(4) The design serviceability shear load of any bolt in a category B connection and the design ultimate shear load of any bolt in a category C connection shall not exceed the design slip resistance of the bolt as obtained from 3.9 at the relevant limit state plus the design bearing resistance of the resin as obtained from 3.6.2.2(5) at the relevant limit state. In addition the design ultimate shear load of a bolt in a category B or C connection shall not exceed either the design shear resistance of the bolt as obtained from 3.6, nor the design bearing resistance of the bolt as obtained from 3.6 and 3.7.

(5) The design bearing resistance of the resin, $F_{b,Rd.resin}$, may be determined according to the following equation:

$$F_{\mathrm{b,Rd,resin}} = \frac{k_t\ k_s\ d\ t_{b,resin}\ \beta\ f_{b,resin}}{\gamma_{M4}} \qquad \text{... (3.4)}$$

where:

$F_{\mathrm{b,Rd,resin}}$ is the bearing strength of an injection bolt

β is a coefficient depending of the thickness ratio of the connected plates as given in Table 3.5 and Figure 3.5

$f_{\mathrm{b,resin}}$ is the bearing strength of the resin to be determined according to the 2.8 Reference Standards: Group 7.

$t_{\mathrm{b,\ resin}}$ is the effective bearing thickness of the resin, given in Table 3.5

k_t is 1,0 for serviceability limit state (long duration)
is 1,2 for ultimate limit state

k_s is taken as 1,0 for holes with normal clearances or (1,0 - 0,1 m), for oversized holes

m is the difference (in mm) between the normal and oversized hole dimensions. In the case of short slotted holes as specified in 2.8 Reference Standards: Group 7, m = 0.5 x (the difference (in mm) between the hole length and width).

(6) When calculating the bearing resistance of a bolt with a clamping length exceeding 3d, a value of not more than 3d should be taken to determine the effective bearing thickness $t_{\mathrm{b,resin}}$ (see Figure 3.6).

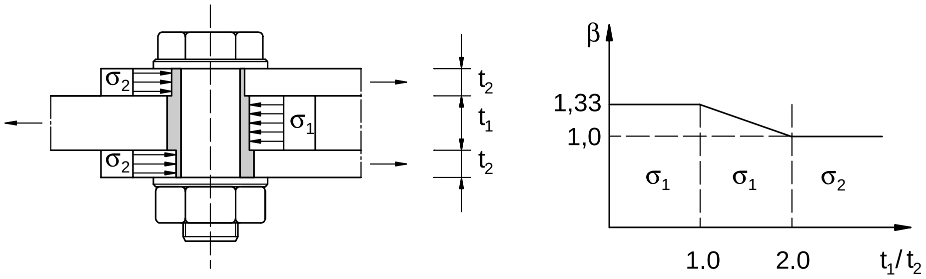


Figure 3.5: Factor *ß* as a function of the thickness ratio of the connected plates

Table 3.5: Values of *ß* and $t_{b,resin}$

t_1 / t_2	$ß$	$t_{b,resin}$
$\geq 2,0$ $1,0 < t_1 / t_2 < 2,0$ $\leq 1,0$	1,0 $1,66 - 0,33\ (t_1 / t_2)$ 1,33	$2\ t_2 \leq\ 1,5\ d$ $t_1 \leq\ 1,5\ d$ $t_1 \leq\ 1,5\ d$

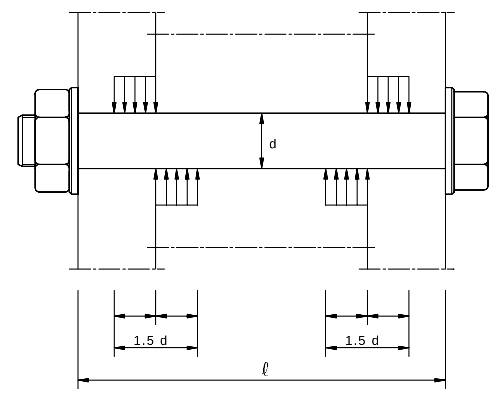


Figure 3.6: Limiting effective length for long injection bolts

3.7 Group of fasteners

(1) The design resistance of a group of fasteners may be taken as the sum of the design bearing resistances $F_{b,Rd}$ of the individual fasteners provided that the design shear resistance $F_{v,Rd}$ of each individual fastener is greater than or equal to the design bearing resistance $F_{b,Rd}$. Otherwise the design resistance of a group of fasteners should be taken as the number of fasteners multiplied by the smallest design resistance of any of the individual fasteners.

3.8 Long joints

(1) Where the distance L_j between the centres of the end fasteners in a joint, measured in the direction of force transfer (see Figure 3.7), is more than 15 d, the design shear resistance $F_{v,Rd}$ of all the fasteners calculated according to Table 3.4 should be reduced by multiplying it by a reduction factor β_{Lf}, given by:

$$\beta_{Lf} = 1 - \frac{L_j - 15d}{200d} \qquad \text{... (3.5)}$$

but $\beta_{Lf} \leq 1{,}0$ and $\beta_{Lf} \geq 0{,}75$

(2) The provision in 3.8(1) does not apply where there is a uniform distribution of force transfer over the length of the joint, e.g. the transfer of shear force between the web and the flange of a section.

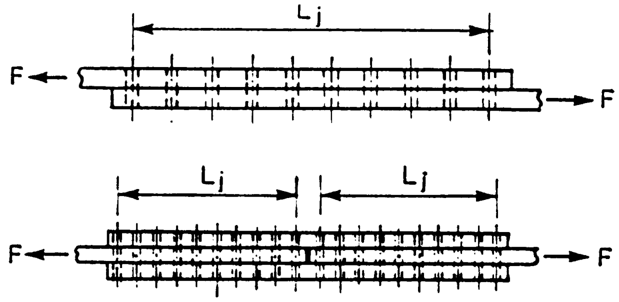


Figure 3.7: Long joints

3.9 Slip-resistant connections using 8.8 or 10.9 bolts

3.9.1 Design Slip resistance

(1) The design slip resistance of a preloaded class 8.8 or 10.9 bolt should be taken as:

$$F_{s,Rd} = \frac{k_s\ n\ \mu}{\gamma_{M3}}\ F_{p,C} \qquad \text{... (3.6)}$$

where:

k_s is given in Table 3.6

n is the number of the friction surfaces

μ is the slip factor obtained either by specific tests for the friction surface in accordance with 2.8 Reference Standards: Group 7 or when relevant as given in Table 3.7.

(2) For class 8.8 and 10.9 bolts conforming with 2.8 Reference Standards: Group 4, with controlled tightening in conformity with 2.8 Reference Standards: Group 7, the preloading force $F_{p,C}$ to be used in equation (3.6) should be taken as:

$$F_{p,C} = 0{,}7\, f_{ub}\, A_s \qquad \text{... (3.7)}$$

Table 3.6: Values of k_s

Description	k_s
Bolts in normal holes.	1,0
Bolts in either oversized holes or short slotted holes with the axis of the slot perpendicular to the direction of load transfer.	0,85
Bolts in long slotted holes with the axis of the slot perpendicular to the direction of load transfer.	0,7
Bolts in short slotted holes with the axis of the slot parallel to the direction of load transfer.	0,76
Bolts in long slotted holes with the axis of the slot parallel to the direction of load transfer.	0,63

Table 3.7: Slip factor, μ, for pre-loaded bolts

Class of friction surfaces (see 2.8 Reference Standard: Group 7)	Slip factor μ
A	0,5
B	0,4
C	0,3
D	0,2

NOTE 1: The requirements for testing and inspection are given in 2.8 Reference Standards: Group 7.

NOTE 2: The classification of any other surface treatment should be based on test specimens representative of the surfaces used in the structure using the procedure set out in 2.8 Reference Standards: Group 7.

NOTE 3: The definitions of the class of friction surface are given in 2.8 Reference Standards: Group 7.

NOTE 4: With painted surface treatments account should made for any loss of pre-load which occur over time.

3.9.2 Combined tension and shear

(1) If a slip-resistant connection is subjected to an applied tensile force, $F_{t,Ed}$ or $F_{t,Ed,serv}$, in addition to the shear force, $F_{v,Ed}$ or $F_{v,Ed,serv}$, tending to produce slip, the design slip resistance per bolt should be taken as follows:

for a category B connection:
$$F_{s,Rd,serv} = \frac{k_s \; n \; \mu \; (F_{p,C} - 0{,}8 F_{t,Ed,serv})}{\gamma_{M3}} \qquad \text{... (3.8a)}$$

for a category C connection:
$$F_{s,Rd} = \frac{k_s \; n \; \mu \; (F_{p,C} - 0{,}8 F_{t,Ed})}{\gamma_{M3}} \qquad \text{... (3.8b)}$$

(2) If, in a moment connection, a contact force on the compression side counterbalances the applied tensile force no reduction in slip resistance is required.

3.9.3 Hybrid connections

(1) As an exception to 2.4(3) , preloaded class 8.8 and 10.9 bolts in connections designed as slip-resistant at the ultimate limit state (Category C in 3.4) may be assumed to share load with welds, provided that the final tightening of the bolts is carried out after the welding is complete.

3.10 Deductions for fastener holes

3.10.1 General

(1) Deduction for holes in the member design should be made according to EN 1993-1-1.

3.10.2 Design for block tearing

(1) Block tearing consists of failure in shear at the row of bolts along the shear face of the hole group accompanied by tensile rupture along the line of bolt holes on the tension face of the bolt group. Figure 3.8 shows block tearing.

(2) For a symmetric bolt group subject to concentric loading the design block tearing resistance, $V_{eff,1,Rd}$ is given by:

$$V_{eff,1,Rd} = f_u A_{nt} / \gamma_{M2} + (1 / \sqrt{3}) f_y A_{nv} / \gamma_{M0} \quad \text{... (3.9)}$$

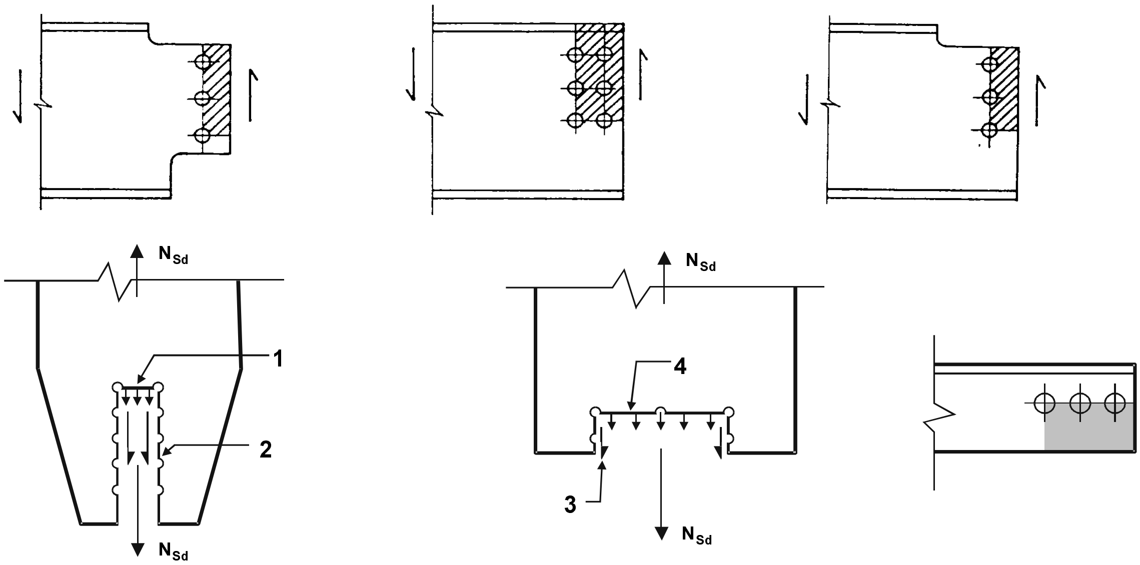
where:

A_{nt} is net area subjected to tension;

A_{nv} is net area subjected to shear.

(3) For a bolt group subject to eccentric loading the design block shear tearing resistance $V_{eff,2,Rd}$ is given by:

$$V_{eff,2,Rd} = 0{,}5\, f_u A_{nt} / \gamma_{M2} + (1 / \sqrt{3}) f_y A_{nv} / \gamma_{M0} \quad \text{... (3.10)}$$



1 small tension force
2 large shear force
3 small shear force
4 large tension force

Figure 3.8: Block tearing

3.10.3 Angles connected by one leg and other unsymmetrically connected members in tension

(1) The eccentricity in joints, see 2.7(1), and the effects of the spacing and edge distances of the bolts, shall be taken into account in determining the design resistance of:

- unsymmetrical members;
- symmetrical members that are connected unsymmetrically, such as angles connected by one leg.

(2) A single angle in tension connected by a single row of bolts in one leg, see Figure 3.9, may be treated as concentrically loaded over an effective net section for which the design ultimate resistance should be determined as follows:

with 1 bolt: $$N_{u,Rd} = \frac{2{,}0(e_2 - 0{,}5d_0)t\, f_u}{\gamma_{M2}} \qquad \text{... (3.11)}$$

with 2 bolts: $$N_{u,Rd} = \frac{\beta_2\, A_{net}\, f_u}{\gamma_{M2}} \qquad \text{... (3.12)}$$

with 3 or more bolts: $$N_{u,Rd} = \frac{\beta_3\, A_{net}\, f_u}{\gamma_{M2}} \qquad \text{... (3.13)}$$

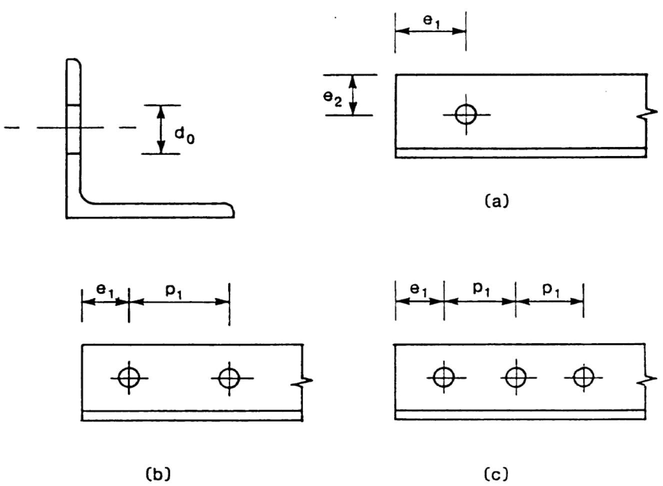
where:

β_2 and β_3 are reduction factors dependent on the pitch p_1 as given in Table 3.8. For intermediate values of p_1 the value of β may be determined by linear interpolation;

A_{net} is the net area of the angle. For an unequal-leg angle connected by its smaller leg, A_{net} should be taken as equal to the net section area of an equivalent equal-leg angle of leg size equal to that of the smaller leg.

Table 3.8: Reduction factors β_2 and β_3

Pitch	p_1	$\leq 2{,}5\ d_o$	$\geq 5{,}0\ d_o$
2 bolts	β_2	0,4	0,7
3 bolts or more	β_3	0,5	0,7



a) 1 bolt

b) 2 bolts

c) 3 bolts

Figure 3.9: Angles connected by one leg

3.10.4 Lug angles

(1) The Lug angle shown in Figure 3.10 connects angle members and their fasteners to a gusset or other supporting part and should be designed to transmit a force 1,2 times the force in the outstand of the angle connected.

(2) The fasteners connecting the lug angle to the outstand of the angle member should be designed to transmit a force 1,4 times the force in the outstand of the angle member.

(3) Lug angles connecting a channel or a similar member should be designed to transmit a force 1,1 times the force in the channel flanges to which they are attached.

(4) The fasteners connecting the lug angle to the channel or similar member should be designed to transmit a force 1,2 times the force in the channel flange which they connect.

(5) In no case should less than two bolts or rivets be used to attach a lug angle to a gusset or other supporting part.

(6) The connection of a lug angle to a gusset plate or other supporting part should terminate at the end of the member connected. The connection of the lug angle to the member should run from the end of the member to a point beyond the direct connection of the member to the gusset or other supporting part.

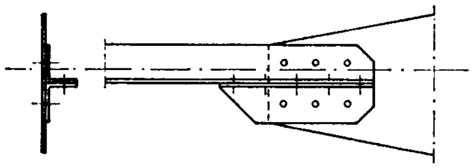


Figure 3.10: Lug angles

3.11 Prying forces

(1) Where fasteners are required to carry an applied tensile force, they should be designed to resist the additional force due to prying action, where this can occur.

NOTE: The rules given in 6.2.4 implicitly account for prying forces.

3.12 Distribution of forces between fasteners at the ultimate limit state

(1) When a moment is applied to a joint, the distribution of internal forces may be either linear (i.e. proportional to the distance from the centre of rotation) or plastic, (i.e. any distribution that is in equilibrium is acceptable provided that the resistances of the components are not exceeded and the ductility of the components is sufficient).

(2) The elastic linear distribution of internal forces should be used for the following:

- when bolts are used creating a category C slip-resistant connection,
- in shear connections where the design shear resistance $F_{v,Rd}$ of a fastener is less than the design bearing resistance $F_{b,Rd}$,
- where connections are subjected to impact, vibration or load reversal (except wind loads).

(3) When a joint is loaded by a concentric shear only, the load may be assumed to be uniformly distributed amongst the fasteners, provided that the size and the class of fasteners is the same.

3.13 Connections made with pins

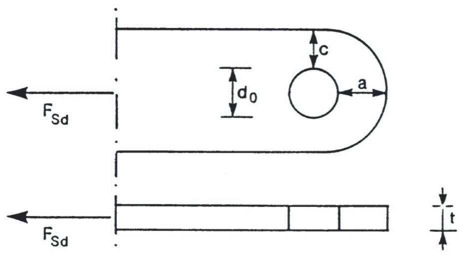
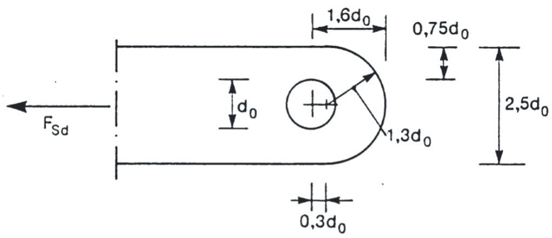
3.13.1 General

(1) Wherever there is a risk of pins becoming loose, they should be secured.

(2) Pin connections in which no rotation is required may be designed as single bolted connections, provided that the length of the pin is less than 3 times the diameter of the pin, see 3.6.1. For all other cases the method given in 3.13.2 should be followed.

(3) In pin-connected members the geometry of the unstiffnened element that contains a hole for the pin should satisfy the dimensional requirements given in Table 3.9.

Table 3.9: Geometrical requirements for pin ended members

Type A:	Given thickness t  $a \geq \frac{F_{Sd}\,\gamma_{M0}}{2tf_y} + \frac{2d_0}{3}$: $c \geq \frac{F_{Sd}\,\gamma_{M0}}{2tf_y} + \frac{d_0}{3}$
Type B:	Given geometry  $t \geq 0{,}7\left[\frac{F_{Sd}\,\gamma_{M0}}{f_y}\right]^{1/2}$: $d_0 \leq 2{,}5t$

(4) Pin connected members should be arranged such to avoid eccentricity and should be of sufficient size to distribute the load from the area of the member with the pin hole into the member away from the pin.

3.13.2 Design of pins

(1) The design requirements for solid circular pins are given in Table 3.10.

(2) The moments in a pin should be calculated on the basis that the connected parts form simple supports. It should be generally assumed that the reactions between the pin and the connected parts are uniformly distributed along the length in contact on each part as indicated in Figure 3.11.

(3) If the pin is intended to be replaceable, in addition to the provisions given in 3.13.1 to 3.13.2, the contact bearing stress should satisfy:

$$\sigma_{h,Ed} \leq f_{h,Rd} \qquad \text{... (3.14)}$$

where:

$$\sigma_{h,Ed} = 0{,}591 \sqrt{\frac{E\, F_{Ed,ser}\ (d_0 - d)}{d^2\ t}} \qquad \text{... (3.15)}$$

$$f_{h,Ed} = 2{,}5 f_y/\gamma_{M6,ser} \qquad \text{... (3.16)}$$

where:

d is the diameter of the pin;

d_0 is the diameter of the pin hole;

$F_{Ed,ser}$ is the design value of the force to be transferred in bearing, under the characteristic load combination for serviceability limit states.

Table 3.10: Design criteria for pin connections

Failure mode	Design requirements
Shear resistance of the pin	$F_{v,Rd} = 0{,}6\, A\, f_{up}/\gamma_{M2} \geq F_{v,Ed}$
Bearing resistance of the plate and the pin If the pin is intended to be replaceable this requirement should also be satisfied.	$F_{b,Rd} = 1{,}5\, t\, d\, f_y/\gamma_{M0} \geq F_{b,Ed}$ $F_{b,Rd,ser} = 0{,}6\, t\, d\, f_y/\gamma_{M6,ser} \geq F_{b,Ed,ser}$
Bending resistance of the pin If the pin is intended to be replaceable this requirement should also be satisfied.	$M_{Rd} = 1{,}5\, W_{e\ell}\, f_{yp}/\gamma_{M0} \geq M_{Ed}$ $M_{Rd,ser} = 0{,}8\, W_{e\ell}\, f_{yp}/\gamma_{M6,ser} \geq M_{Ed,ser}$
Combined shear and bending resistance of the pin	$\left[\frac{M_{Ed}}{M_{Rd}}\right]^2 + \left[\frac{F_{v,Ed}}{F_{v,Rd}}\right]^2 \leq 1$

d is the diameter of the pin;

f_y is the lower of the design strengths of the pin and the connected part;

f_{up} is the ultimate tensile strength of the pin;

f_{yp} is the yield strength of the pin;

t is the thickness of the connected part;

A is the cross-sectional area of a pin.

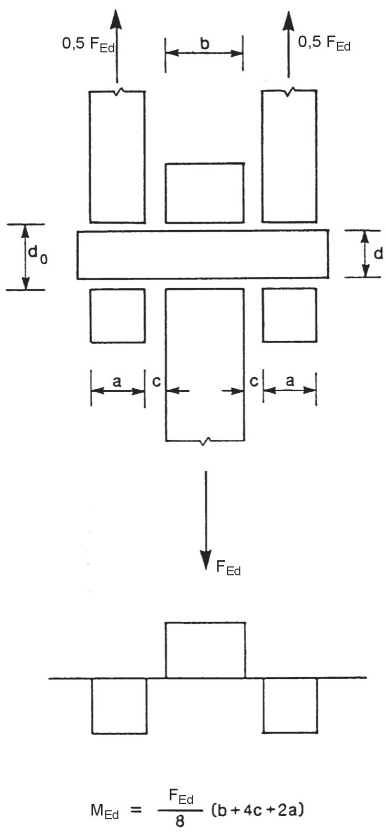


Figure 3.11: Bending moment in a pin

4 Welded connections

4.1 General

(1) The provisions in this section apply to weldable structural steels conforming to EN 1993-1-1 and to material thicknesses of 4 mm and over. The provisions also apply to joints in which the mechanical properties of the weld metal are compatible with those of the parent metal, see 4.2.

For welds in thinner material reference should be made to EN 1993 part 1.3 and for welds in structural hollow sections in material thicknesses of 2,5 mm and over guidance is given section 7 of this Standard.

For stud welding reference should be made to EN 1994-1-1.

NOTE: Further guidance on stud welding can be found in EN ISO 14555 and EN ISO 13918.

(2) Welds subject to fatigue shall also satisfy the principles given in EN 1993-1-9.

(3) Quality level C according to EN ISO 25817 is usually required, if not otherwise specified. The frequency of inspection of welds should be specified in accordance with the rules in 2.8 Reference Standards: Group 7. The quality level of welds should be chosen according to EN ISO 25817. For the quality level of welds used in fatigue loaded structures, see EN 1993-1-9.

(4) Lamellar tearing shall be avoided.

(5) Guidance on lamellar tearing is given in EN 1993-1-10.

4.2 Welding consumables

(1) All welding consumables should conform to the relevant standards specified in 2.8 Reference Standards; Group 5.

(2) The specified yield strength, ultimate tensile strength, elongation at failure and minimum Charpy V-notch energy value of the filler metal, should be equivalent to, or better than that specified for the parent material.

NOTE: Generally it is safe to use electrodes that are overmatched with regard to the steel grades being used.

4.3 Geometry and dimensions

4.3.1 Type of weld

(1) This Standard covers the design of fillet welds, fillet welds all round, butt welds, plug welds and flare groove welds. Butt welds may be either full penetration butt welds or partial penetration butt welds. Both fillet welds all round and plug welds may be either in circular holes or in elongated holes.

(2) The most common types of joints and welds are illustrated in EN 12345.

4.3.2 Fillet welds

4.3.2.1 General

(1) Fillet welds may be used for connecting parts where the fusion faces form an angle of between 60° and 120°.

(2) Angles smaller than 60° are also permitted. However, in such cases the weld should be considered to be a partial penetration butt weld.

(3) For angles greater than 120° the resistance of fillet welds should be determined by testing in accordance with EN 1990 Annex D: Design by testing.

(4) Fillet welds finishing at the ends or sides of parts should be returned continuously, full size, around the corner for a distance of at least twice the leg length of the weld, unless access or the configuration of the joint renders this impracticable.

NOTE: In the case of intermittent welds this rule applies only to the last intermittent fillet weld at corners.

(5) End returns should be indicated on the drawings.

(6) For eccentricity of single-sided fillet welds, see 4.12.

4.3.2.2 Intermittent fillet welds

(1) Intermittent fillet welds shall not be used in corrosive conditions.

(2) In an intermittent fillet weld, the gaps (L_1 or L_2) between the ends of each length of weld L_w should fulfil the requirement given in Figure 4.1.

(3) In an intermittent fillet weld, the gap (L_1 or L_2) should be taken as the smaller of the distances between the ends of the welds on opposite sides and the distance between the ends of the welds on the same side.

(4) In any run of intermittent fillet weld there should always be a length of weld at each end of the part connected.

(5) In a built-up member in which plates are connected by means of intermittent fillet welds, a continuous fillet weld should be provided on each side of the plate for a length at each end equal to at least three-quarters of the width of the narrower plate concerned (see Figure 4.1).

4.3.3 Fillet welds all round

(1) Fillet welds all round, comprising fillet welds in circular or elongated holes, may be used only to transmit shear or to prevent the buckling or separation of lapped parts.

(2) The diameter of a circular hole, or width of an elongated hole, for a fillet weld all round should not be less than four times the thickness of the part containing it.

(3) The ends of elongated holes should be semi-circular, except for those ends which extend to the edge of the part concerned.

(4) The centre to centre spacing of fillet welds all round should not exceed the value necessary to prevent local buckling, see Table 3.3.

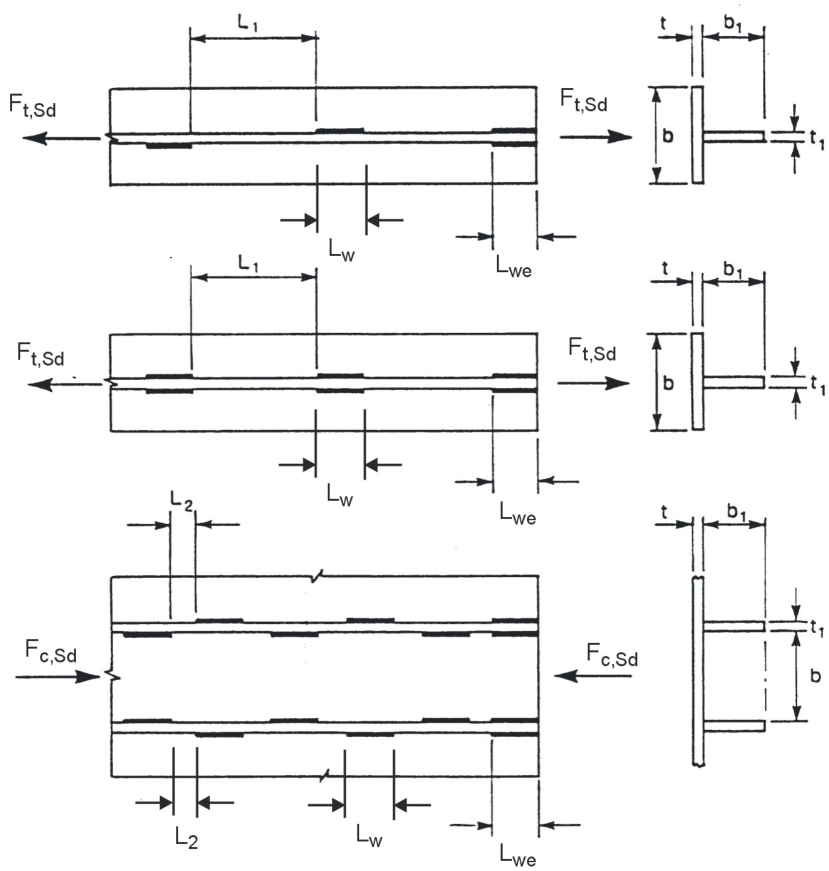
4.3.4 Butt welds

(1) A full penetration butt weld is defined as a weld that has complete penetration and fusion of weld and parent metal throughout the thickness of the joint.

(2) A partial penetration butt weld is defined as a weld that has joint penetration which is less than the full thickness of the parent material.

(3) Intermittent butt welds should not be used.

(4) For eccentricity in single-sided partial penetration butt welds, see 4.12.



The larger of $L_{we} \geq 0{,}75\ b$ and $0{,}75\ b_1$

For build-up members in tension:
The smallest of $L_1 \leq 16\ t$ and $16\ t_1$ and 200 mm

For build-up members in compression or shear:
The smallest of $L_2 \leq 12\ t$ and $12\ t_1$ and $0{,}25\ b$ and 200 mm

Figure 4.1: Intermittent fillet welds

4.3.5 Plug welds

(1) Plug welds may be used:

- to transmit shear,
- to prevent the buckling or separation of lapped parts, and
- to inter-connect the components of built-up members

but should not be used to resist externally applied tension.

(2) The diameter of a circular hole, or width of an elongated hole, for a plug weld should be at least 8 mm more than the thickness of the part containing it.

(3) The ends of elongated holes should either be semi-circular or else should have corners which are rounded to a radius of not less than the thickness of the part containing the slot, except for those ends which extend to the edge of the part concerned.

(4) The thickness of a plug weld in parent material up to 16 mm thick should be equal to the thickness of the parent material. The thickness of a plug weld in parent material over 16 mm thick should be at least half the thickness of the parent material and not less than 16 mm.

(5) The centre to centre spacing of plug welds should not exceed the value necessary to prevent local buckling, see Table 3.3.

4.3.6 Flare groove welds

(1) For solid bars the design throat thickness of flare groove welds, when fitted flush to the surface of the solid section of the bars, is defined in Figure 4.2. The definition of the design throat thickness of flare groove welds in rectangular hollow sections is given in 7.3.1(7).

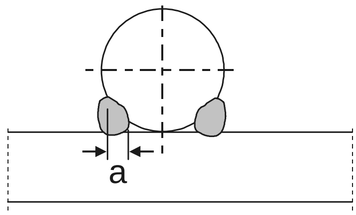


Figure 4.2: Effective throat thickness of flare groove welds in solid sections

4.4 Welds with packings

(1) In the case of welds with packing, the packing should be trimmed flush with the edge of the part that is to be welded.

(2) Where two parts connected by welding are separated by packing having a thickness less than the leg length of weld necessary to transmit the force, the required leg length should be increased by the thickness of the packing.

(3) Where two parts connected by welding are separated by packing having a thickness equal to, or greater than, the leg length of weld necessary to transmit the force, each of the parts should be connected to the packing by a weld capable of transmitting the design force.

4.5 Design resistance of a fillet weld

4.5.1 Length of welds

(1) The effective length of a fillet weld l should be taken as the length over which the fillet is full-size. This maybe taken as the overall length of the weld reduced by twice the effective throat thickness a. Provided that the weld is full size throughout its length including starts and terminations, no reduction in effective length need be made for either the start or the termination of the weld.

(2) A fillet weld with an effective length less than 30 mm or less than 6 times its throat thickness, whichever is larger, should not be designed to carry load.

4.5.2 Effective throat thickness

(1) The effective throat thickness, a, of a fillet weld should be taken as the height of the largest triangle (with equal or unequal legs) that can be inscribed within the fusion faces and the weld surface, measured perpendicular to the outer side of this triangle, see Figure 4.3.

(2) The effective throat thickness of a fillet weld should not be less than 3 mm.

(3) In determining the design resistance of a deep penetration fillet weld, account may be taken of its additional throat thickness, see Figure 4.4, provided that preliminary tests show that the required penetration can consistently be achieved.

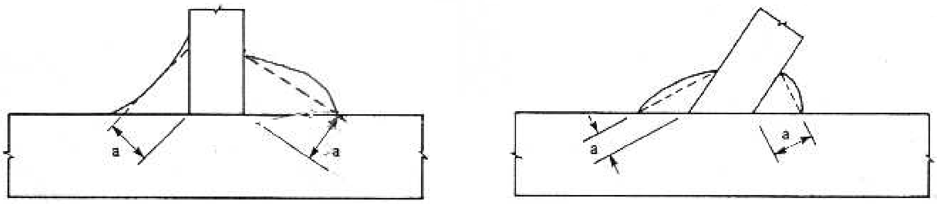


Figure 4.3: Throat thickness of a fillet weld

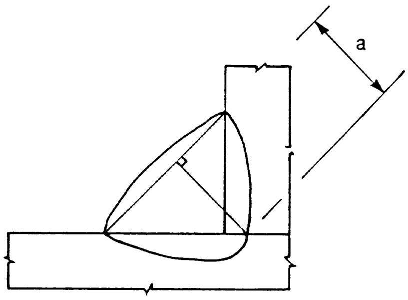


Figure 4.4: Throat thickness of a deep penetration fillet weld

4.5.3 Design Resistance of fillet welds

4.5.3.1 General

(1) The design resistance of a fillet weld should be determined using either the Directional method given in 4.5.3.2 or the Simplified method given in 4.5.3.3.

4.5.3.2 Directional method

(1) In this method, the forces transmitted by a unit length of weld are resolved into components parallel and transverse to the longitudinal axis of the weld and normal and transverse to the plane of its throat.

(2) The design throat area A_w should be taken as $A_w = \sum a\, \ell_{eff}$.

(3) The location of the design throat area should be assumed to be concentrated in the root.

(4) A uniform distribution of stress is assumed on the throat section of the weld, leading to the normal stresses and shear stresses shown in Figure 4.5, as follows:

- $\sigma_\perp$ is the normal stress perpendicular to the throat
- $\sigma_{\parallel}$ is the normal stress parallel to the axis of the weld
- $\tau_\perp$ is the shear stress (in the plane of the throat) perpendicular to the axis of the weld
- $\tau_{\parallel}$ is the shear stress (in the plane of the throat) parallel to the axis of the weld.

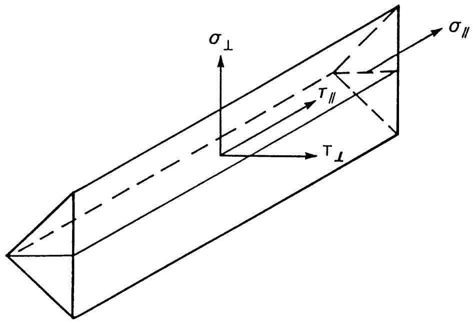


Figure 4.5: Stresses on the throat section of a fillet weld

(5) The normal stress $\sigma_\parallel$ parallel to the axis is not considered when verifying the design resistance of the weld.

(6) The design resistance of the fillet weld will be sufficient if the following are both satisfied:

$$[\sigma_\perp^2 + 3\,(\tau_\perp^2 + \tau_\parallel^2)]^{0,5} \leq f_u / (\beta_w\, \gamma_{M2}) \quad \text{and} \quad \sigma_\perp \leq f_u / \gamma_{M2} \qquad \text{... (4.1)}$$

where:

f_u is the nominal ultimate tensile strength of the weaker part joined;

β_w is the appropriate correlation factor taken from Table 4.1.

(7) Welds between parts with different material strength grades should be designed using the properties of the material with the lower strength grade.

Table 4.1: Correlation factor β_w for fillet welds

Standard and steel grade			Correlation factor β_w
EN 10025	**EN 10210**	**EN 10219**	
S 235 S 235 W	S 235 H	S 235 H	0,8
S 275 S 275 N/NL S 275 M/ML	S 275 H S 275 NH/NLH	S 275 H S 275 NH/NLH S 275 MH/MLH	0,85
S 355 S 355 N/NL S 355 M/ML S 355 W	S 355 H S 355 NH/NLH	S 355 H S 355 NH/NLH S 355 MH/MLH	0,9
S 420 N/NL S 420 M/ML		S 420 MH/MLH	1,0
S 460 N/NL S 460 M/ML S 460 Q/QL/QL1	S 460 NH/NLH	S 460 NH/NLH S 460 MH/MLH	1,0

4.5.3.3 Simplified method for design resistance of fillet weld

(1) Alternatively to 4.5.3.2 the design resistance of a fillet weld may be assumed to be adequate if, at every point along its length, the resultant of all the forces per unit length transmitted by the weld satisfy the following criterion:

$$F_{w,Ed} \leq F_{w,Rd} \qquad \text{... (4.2)}$$

where:

$F_{w,Ed}$ is the design value of the weld force per unit length;

$F_{w,Rd}$ is the design weld resistance per unit length.

(2) Independent of the orientation of the weld throat plane to the applied force, the design resistance per unit length $F_{w,Rd}$ should be determined from:

$$F_{w,Rd} = f_{vw.d}\, a \qquad \text{... (4.3)}$$

where:

$f_{vw.d}$ is the design shear strength of the weld.

(3) The design shear strength $f_{vw.d}$ of the weld should be determined from:

$$f_{vw.d} = \frac{f_u / \sqrt{3}}{\beta_w \gamma_{M2}} \qquad \text{... (4.4)}$$

where:

f_u and β_w are defined in 4.5.3(7).

4.6 Design resistance of fillet welds all round

(1) The design resistance of a fillet weld all round should be determined using one of the methods given in 4.5.

4.7 Design resistance of butt welds

4.7.1 Full penetration butt welds

(1) The design resistance of a full penetration butt weld should be taken as equal to the design resistance of the weaker of the parts connected, provided that the weld is made with a suitable consumable which will produce all-weld tensile specimens having both a minimum yield strength and a minimum tensile strength not less than those specified for the parent metal.

4.7.2 Partial penetration butt welds

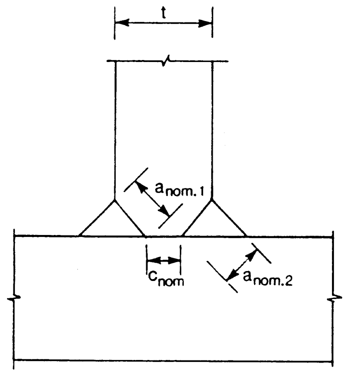
(1) The design resistance of a partial penetration butt weld should be determined using the method for a deep penetration fillet weld given in 4.5.2(3).

(2) The throat thickness of a partial penetration butt weld should not be greater than the depth of penetration that can be consistently achieved, see 4.5.2(3).

4.7.3 T-butt joints

(1) The design resistance of a T-butt joint, consisting of a pair of partial penetration butt welds reinforced by superimposed fillet welds, may be determined as for a full penetration butt weld (see 4.7.1) if the total nominal throat thickness, exclusive of the unwelded gap, is not less than the thickness t of the part forming the stem of the tee joint, provided that the unwelded gap is not more than (t / 5) or 3 mm, whichever is less, see Figure 4.6(a).

(2) The design resistance of a T-butt joint which does not meet the requirements given in 4.7.3(1) should be determined using the method for a fillet weld or a deep penetration fillet weld given in 4.5 depending on the amount of penetration. The throat thickness should be determined in conformity with the provisions for both fillet welds (see 4.5.2) and partial penetration butt welds (see 4.7.2).



$a_{nom,1} + a_{nom,2} \geq t$
The smaller of $c_{nom} \leq t/5$ and 3 mm

Figure 4.6: Effective full penetration of T-butt welds

4.8 Design resistance of plug welds

(1) The design resistance $F_{w,Rd}$ of a plug weld (see 4.3.5) should be taken as:

$$F_{w,Rd} = f_{vw,d} A_w, \quad \text{... (4.5)}$$

where

$f_{vw.d}$ is the design shear strength of a weld given in 4.5.3.3(4).

A_w is the design throat area and should be taken as the area of the hole.

4.9 Distribution of forces

(1) The distribution of forces in a welded connection may be calculated on the assumption of either elastic or plastic behaviour in conformity with 2.4 and 2.5.

(2) It is acceptable to assume a simplified load distribution within the welds.

(3) Residual stresses and stresses not subjected to transfer of load need not be included when checking the resistance of a weld. This applies specifically to the normal stress parallel to the axis of a weld.

(4) Welded joints should be designed to have adequate deformation capacity. However, ductility of the welds should not be relied upon.

(5) In joints where plastic hinges may form, the welds should be designed to provide at least the same design resistance as the weakest of the connected parts.

(6) In other joints where deformation capacity for joint rotation is required due to the possibility of excessive straining, the welds require sufficient strength not to rupture before general yielding in the adjacent parent material.

(7) If the design resistance of an intermittent weld is determined by using the total length ℓ_{tot}, the weld shear force per unit length $F_{w,Ed}$ should be multiplied by the factor $(e+\ell)/\ell$, see Figure 4.7.

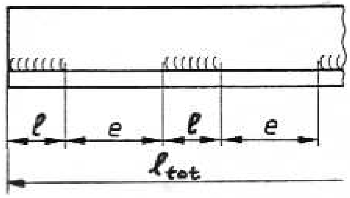


Figure 4.7: Calculation of weld forces for intermittent welds

4.10 Connections to unstiffened flanges

(1) Where a transverse plate (or beam flange) is welded to a supporting unstiffened flange of an I, H or other section, see Figure 4.8, and provided that the condition given in 4.10(3) is met, the applied force perpendicular to the unstiffened flange should not exceed any of the relevant design resistances as follows:

- that of the web of the supporting member of I or H sections as given in 6.2.6.2 or 6.2.6.3 as appropriate,
- those for a transverse plate on a RHS member as given in Table 7.13,
- that of the supporting flange as given by formula (6.20) in 6.2.6.4.3(1) calculated assuming the applied force is concentrated over an effective width, b_{eff}, of the flange as given in 4.10(2) or 4.10(4) as relevant."

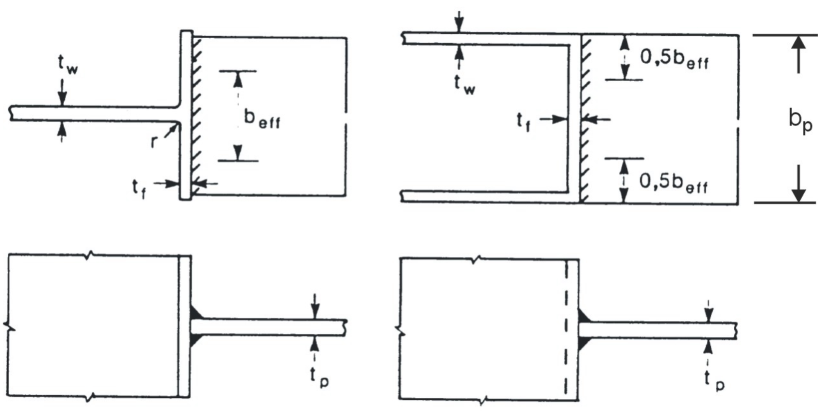


Figure 4.8: Effective width of an unstiffened T-joint

(2) For an unstiffened I or H section the effective width b_{eff} should be obtained from:

$$b_{eff} = t_w + 2s + 7kt_f \quad \text{... (4.6a)}$$

where:

$$k = (t_f / t_p)(f_{y,f} / f_{y,p}) \text{ but } k \leq 1 \quad \text{... (4.6b)}$$

$f_{y,f}$ is the yield strength of the flange of the I or H section;

$f_{y,p}$ is the yield strength of the plate welded to the I or H section.

The dimension s should be obtained from:

- for a rolled I or H section: $s = r$ ... (4.6c)
- for a welded I or H section: $s = \sqrt{2}\, a$ ... (4.6d)

(3) For an unstiffened flange of an I or H section , the following criterion should be satisfied:

$$b_{\mathrm{eff}} \geq (f_{y,p} / f_{u,p}) b_p \qquad \text{... (4.7)}$$

where:

$f_{\mathrm{u,p}}$ is the ultimate strength of the plate welded to the I or H section.

b_{p} is the width of the plate welded to the I or H section.

Otherwise the joint should be stiffened.

(4) For other sections such as box sections or channel sections where the width of the connected plate is similar to the width of the flange, the effective width b_{eff} should be obtained from:

$$b_{\mathrm{eff}} = 2t_{\mathrm{w}} + 5t_{\mathrm{f}} \quad \text{but} \quad b_{\mathrm{eff}} \leq 2t_{\mathrm{w}} + 5\, k\, t_{\mathrm{f}} \qquad \text{... (4.8)}$$

NOTE: For hollow sections, see Table 7.13.

(5) Even if $b_{\mathrm{eff}} \leq b_{\mathrm{p}}$, the welds connecting the plate to the flange need to be designed to transmit the design resistance of the plate $b_{\mathrm{P}}\, t_{\mathrm{P}} f_{\mathrm{y,P}} / \gamma_{\mathrm{M0}}$ assuming a uniform stress distribution.

4.11 Long joints

(1) In lap joints the design resistance of a fillet weld should be reduced by multiplying it by a reduction factor β_{Lw} to allow for the effects of non-uniform distribution of stress along its length.

(2) The provisions given in 4.11 do not apply when the stress distribution along the weld corresponds to the stress distribution in the adjacent base metal, as, for example, in the case of a weld connecting the flange and the web of a plate girder.

(3) Generally in lap joints longer than $150a$ the reduction factor β_{Lw} should be taken as $\beta_{\mathrm{Lw.1}}$ given by:

$$\beta_{\mathrm{Lw.1}} = 1{,}2 - 0{,}2L_{\mathrm{j}} /(150a) \quad \text{but} \quad \beta_{\mathrm{Lw.1}} \leq 1{,}0 \quad \text{... (4.9)}$$

where:

L_{j} is the overall length of the lap in the direction of the force transfer.

(4) For fillet welds longer than 1,7 metres connecting transverse stiffeners in plated members, the reduction factor β_{Lw} may be taken as $\beta_{\mathrm{Lw.2}}$ given by:

$$\beta_{\mathrm{Lw.2}} = 1{,}1 - L_{\mathrm{w}} /17 \quad \text{but} \quad \beta_{\mathrm{Lw.2}} \leq 1{,}0 \quad \text{and} \quad \beta_{\mathrm{Lw.2}} \geq 0{,}6 \qquad \text{... (4.10)}$$

where:

L_{w} is the length of the weld (in metres).

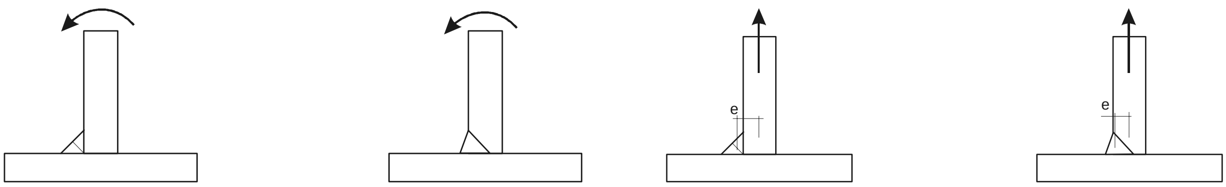
4.12 Eccentrically loaded single fillet or single-sided partial penetration butt welds

(1) Local eccentricity should be avoided whenever it is possible.

(2) Local eccentricity (relative to the line of action of the force to be resisted) should be taken into account in the following cases:

- Where a bending moment transmitted about the longitudinal axis of the weld produces tension at the root of the weld, see Figure 4.9(a);
- Where a tensile force transmitted perpendicular to the longitudinal axis of the weld produces a bending moment, resulting in a tension force at the root of the weld, see Figure 4.9(b).

(3) Local eccentricity need not be taken into account if a weld is used as part of a weld group around the perimeter of a structural hollow section.



(a) Bending moment produces tension at the root of the weld

(b) Tensile force produces tension at the root of the weld

Figure 4.9: Single fillet welds and single-sided partial penetration butt welds

4.13 Angles connected by one leg

(1) In angles connected by one leg, the eccentricity of welded lap joint end connections may be allowed for by adopting an effective cross-sectional area and then treating the member as concentrically loaded.

(2) For an equal-leg angle, or an unequal-leg angle connected by its larger leg, the effective area may be taken as equal to the gross area.

(3) For an unequal-leg angle connected by its smaller leg, the effective area should be taken as equal to the gross cross-sectional area of an equivalent equal-leg angle of leg size equal to that of the smaller leg, when determining the design resistance of the cross-section, see EN 1993-1-1. However when determining the design buckling resistance of a compression member, see EN 1993-1-1, the actual gross cross-sectional area should be used.

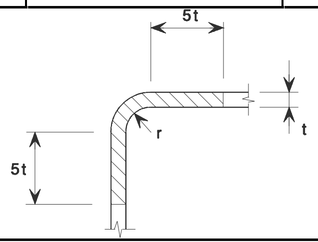
4.14 Welding in cold-formed zones

(1) Welding may be carried out within a length 5t either side of a cold-formed zone, see Table 4.2, provided that one of the following conditions is fulfilled:

- the cold-formed zones are normalized after cold-forming but before welding;
- the *r*/*t*-ratio satisfy the relevant value obtained from Table 4.2.

Table 4.2: Conditions for welding cold-formed zones and adjacent material

r/t	Strain due to cold forming (%)	Maximum thickness (mm)		
		Generally		Fully killed Aluminium-killed steel (Al ≥ 0,02 %)
		Predominantly static loading	Where fatigue predominates	
≥ 25	≥ 2	any	any	any
≥ 10	≥ 5	any	16	any
≥ 3,0	≥ 14	24	12	24
≥ 2,0	≥ 20	12	10	12
≥ 1,5	≥ 25	8	8	10
≥ 1,0	≥ 33	4	4	6



5 Analysis, classification and modelling

5.1 Global analysis

5.1.1 General

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

(2) To identify whether the effects of joint behaviour on the analysis need be taken into account, a distinction may be made between three simplified joint models as follows:

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

(3) The appropriate type of joint model should be determined from Table 5.1, depending on the classification of the joint and on the chosen method of analysis.

(4) The design moment-rotation characteristic of a joint used in the analysis may be simplified by adopting any appropriate curve, including a linearised approximation (e.g. bi-linear or tri-linear), provided that the approximate curve lies wholly below the design moment-rotation characteristic.

Table 5.1: Type of joint model

Method of global analysis	Classification of joint		
Elastic	Nominally pinned	Rigid	Semi-rigid
Rigid-Plastic	Nominally pinned	Full-strength	Partial-strength
Elastic-Plastic	Nominally pinned	Rigid and full-strength	Semi-rigid and partial-strength Semi-rigid and full-strength Rigid and partial-strength
Type of joint model	Simple	Continuous	Semi-continuous

5.1.2 Elastic global analysis

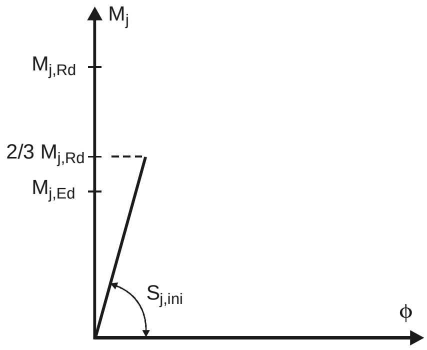
(1) The joints should be classified according to their rotational stiffness, see 5.2.2.

(2) The joints shall have sufficient strength to transmit the forces and moments acting at the joints resulting from the analysis.

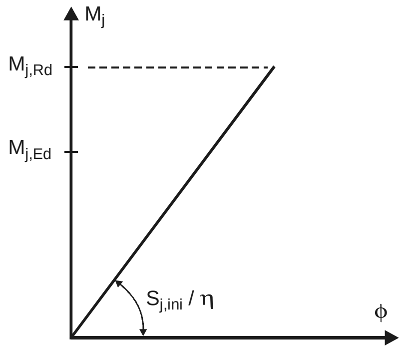
(3) In the case of a semi-rigid joint, the rotational stiffness S_j corresponding to the bending moment $M_{j,Ed}$ should generally be used in the analysis. If $M_{j,Ed}$ does not exceed 2/3 $M_{j,Rd}$ the initial rotational stiffness $S_{j,ini}$ may be taken in the global analysis, see Figure 5.1(a).

(4) As a simplification to 5.1.2(3), the rotational stiffness may be taken as $S_{j,ini}/\eta$ in the analysis, for all values of the moment $M_{j,Ed}$, as shown in Figure 5.1(b), where η is the stiffness modification coefficient from Table 5.2.

(5) For joints connecting H or I sections S_j is given in 6.3.1.



a) $M_{j,Ed} \leq 2/3\ M_{j,Rd}$



b) $M_{j,Ed} \leq M_{j,Rd}$

Figure 5.1: Rotational stiffness to be used in elastic global analysis

Table 5.2: Stiffness modification coefficient η

Type of connection	Beam-to-column joints	Other types of joints (beam-to-beam joints, beam splices, column base joints)
Welded	2	3
Bolted end-plate	2	3
Bolted flange cleats	2	3,5
Base plates	-	3

5.1.3 Rigid-plastic global analysis

(1) The joints should be classified according to their strength, see 5.2.3.

(2) For joints connecting H or I sections $M_{j,Rd}$ is given in 6.2.

(3) For joints connecting hollow sections the method given in section 7 may be used.

(4) The rotation capacity of a joint shall be sufficient to accommodate the rotations resulting from the analysis.

(5) For joints connecting H or I sections the rotation capacity should be checked according to 6.4.

5.1.4 Elastic- plastic global analysis

(1) The joints should be classified according to both stiffness (see 5.2.2) and strength (see 5.2.3).

(2) For joints connecting H or I sections $M_{j,Rd}$ is given in 6.2, S_j is given in 6.3.1 and ϕ_{Cd} is given in 6.4.

(3) For joints connecting hollow sections the method given in section 7 may be used.

(4) The moment rotation characteristic of the joints should be used to determine the distribution of internal forces and moments.

(5) As a simplification, the bi-linear design moment-rotation characteristic shown in Figure 5.2 may be adopted. The stiffness modification coefficient η should be obtained from Table 5.2.

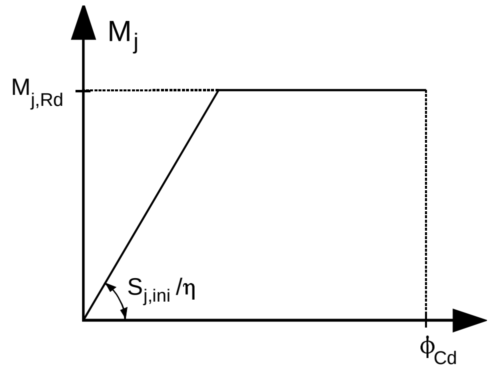


Figure 5.2: Simplified bi-linear design moment-rotation characteristic

5.1.5 Global analysis of lattice girders

(1) The provisions given in 5.1.5 apply only to structures whose joints are verified according to section 7.

(2) The distribution of axial forces in a lattice girder may be determined on the assumption that the members are connected by pinned joints (see also 2.7).

(3) Secondary moments at the joints, caused by the rotational stiffnesses of the joints, may be neglected both in the design of the members and in the design of the joints, provided that both of the following conditions are satisfied:

- the joint geometry is within the range of validity specified in Table 7.1, Table 7.8, Table 7.9 or Table 7.20 as appropriate;
- the ratio of the system length to the depth of the member in the plane of the lattice girder is not less than the appropriate minimum value. For building structures, the appropriate minimum value may be assumed to be 6. Larger values may apply in other parts of EN 1993.

(4) The moments resulting from transverse loads (whether in-plane or out-of-plane) that are applied between panel points, should be taken into account in the design of the members to which they are applied. Provided that the conditions given in 5.1.5(3) are satisfied:

- the brace members may be considered as pin-connected to the chords, so moments resulting from transverse loads applied to chord members need not be distributed into brace members, and vice versa;
- the chords may be considered as continuous beams, with simple supports at panel points.

(5) Moments resulting from eccentricities may be neglected in the design of tension chord members and brace members. They may also be neglected in the design of connections if the eccentricities are within the following limits:

- $-0{,}55\ d_0 \leq e \leq 0{,}25\ d_0$ ... (5.1a)
- $-0{,}55\ h_0 \leq e \leq 0{,}25\ h_0$ ... (5.1b)

where:

e is the eccentricity defined in Figure 5.3;

d_0 is the diameter of the chord;

h_0 is the depth of the chord, in the plane of the lattice girder.

(6) When the eccentricities are within the limits given in 5.1.5(5), the moments resulting from the eccentricities should be taken into account in the design of compression chord members. In this case the moments produced by the eccentricity should be distributed between the compression chord

members on each side of the joint, on the basis of their relative stiffness coefficients I/L, where L is the system length of the member, measured between panel points.

(7) When the eccentricities are outside the limits given in 5.1.5(5), the moments resulting from the eccentricities should be taken into account in the design of the connections and the compression chord members. In this case the moments produced by the eccentricity should be distributed between all the members meeting at the joint, on the basis of their relative stiffness coefficients I/L.

(8) The stresses in a chord resulting from moments taken into account in the design of the chord, should also be taken into account in determining the factors k_m, k_n and k_p used in the design of the connections, see Table 7.2 to Table 7.5, Table 7.10 and Table 7.12 to Table 7.14.

(9) The cases where moments should be taken into account are summarized in Table 5.3.

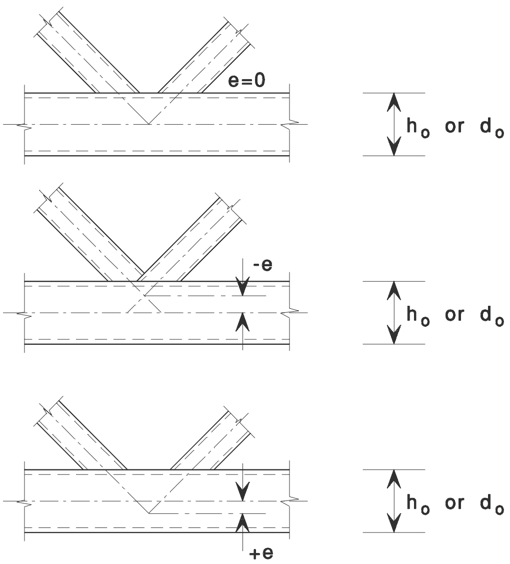


Figure 5.3: Eccentricity of joints

Table 5.3 Allowance for bending moments

<table>
<tr><td rowspan="2">Type of component</td><td colspan="3">Source of the bending moment</td></tr>
<tr><td>Secondary effects</td><td>Transverse loading</td><td>Eccentricity</td></tr>
<tr><td>Compression chord</td><td rowspan="4">Not if 5.1.5(3) is satisfied</td><td rowspan="4">Yes</td><td>Yes</td></tr>
<tr><td>Tension chord</td><td>No</td></tr>
<tr><td>Brace member</td><td>No</td></tr>
<tr><td>Connection</td><td>Not if 5.1.5(5) is satisfied</td></tr>
</table>

5.2 Classification of joints

5.2.1 General

(1) The details of all joints shall fulfil the assumptions made in the relevant design method, without adversely affecting any other part of the structure.

(2) Joints may be classified by their stiffness (see 5.2.2) and by their strength (see 5.2.3).

5.2.2 Classification by stiffness

5.2.2.1 General

(1) A joint may be classified as rigid, nominally pinned or semi-rigid according to its rotational stiffness, by comparing its initial rotational stiffness $S_{j,ini}$ with the classification boundaries given in 5.2.2.5.

NOTE: Rules for the determination of $S_{j,ini}$ for joints connecting H or I sections are given in 6.3.1. Rules for the determination of $S_{j,ini}$ for joints connecting hollow sections are not given in this Standard.

(2) A joint may be classified on the basis of experimental evidence, experience of previous satisfactory performance in similar cases or by calculations based on test evidence.

5.2.2.2 Nominally pinned joints

(1) A nominally pinned joint shall be capable of transmitting the internal forces, without developing significant moments which might adversely affect the members or the structure as a whole.

(2) A nominally pinned joint shall be capable of accepting the resulting rotations under the design loads.

5.2.2.3 Rigid joints

(1) Joints classified as rigid may be assumed to have sufficient rotational stiffness to justify analysis based on full continuity.

5.2.2.4 Semi-rigid joints

(1) A joint which does not meet the criteria for a rigid joint or a nominally pinned joint should be classified as a semi-rigid joint.

NOTE: Semi-rigid joints provide a predictable degree of interaction between members, based on the design moment-rotation characteristics of the joints.

(2) Semi-rigid joints should be capable of transmitting the internal forces and moments.

5.2.2.5 Classification boundaries

(1) Classification boundaries for joints other than column bases are given in 5.2.2.1(1) and Figure 5.4.

(2) Column bases may be classified as rigid provided the following conditions are satisfied:

– in frames where the bracing system reduces the horizontal displacement by at least 80 % and where the effects of deformation may be neglected

  – if $\bar{\lambda}_0 \leq 0,5$; ... (5.2a)

  – if $0,5 < \bar{\lambda}_0 < 3,93$ and $S_{j,ini} \geq 7 (2 \bar{\lambda}_0 - 1) EI_c / L_c$; ... (5.2b)

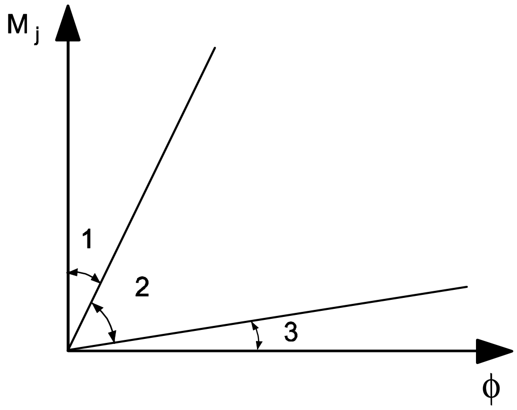
  – if $\bar{\lambda}_0 \geq 3,93$ and $S_{j,ini} \geq 48 EI_c / L_c$. ... (5.2c)

– otherwise if $S_{j,ini} \geq 30 EI_c / L_c$. ... (5.2d)

where:

$\bar{\lambda}_0$ is the slenderness of a column in which both ends are assumed to be pinned;

I_c, L_c are as given in Figure 5.4.



Zone 1: rigid, if $S_{j,ini} \geq k_b EI_b / L_b$

where

$k_b = 8$ for frames where the bracing system reduces the horizontal displacement by at least 80 %

$k_b = 25$ for other frames, provided that in every storey $K_b/K_c \geq 0,1$ *)

Zone 2: semi-rigid

All joints in zone 2 should be classified as semi-rigid. Joints in zones 1 or 3 may optionally also be treated as semi-rigid.

Zone 3: nominally pinned, if $S_{j,ini} \leq 0,5 EI_b / L_b$

*) For frames where $K_b/K_c < 0,1$ the joints should be classified as semi-rigid.

Key:

K_b is the mean value of I_b/L_b for all the beams at the top of that storey;

K_c is the mean value of I_c/L_c for all the columns in that storey;

I_b is the second moment of area of a beam;

I_c is the second moment of area of a column;

L_b is the span of a beam (centre-to-centre of columns);

L_c is the storey height of a column.

Figure 5.4: Classification of joints by stiffness

5.2.3 Classification by strength

5.2.3.1 General

(1) A joint may be classified as full-strength, nominally pinned or partial strength by comparing its design moment resistance $M_{j,Rd}$ with the design moment resistances of the members that it connects. When classifying joints, the design resistance of a member should be taken as that member adjacent to the joint.

5.2.3.2 Nominally pinned joints

(1) A nominally pinned joint shall be capable of transmitting the internal forces, without developing significant moments which might adversely affect the members or the structure as a whole.

(2) A nominally pinned joint shall be capable of accepting the resulting rotations under the design loads.

(3) A joint may be classified as nominally pinned if its design moment resistance $M_{j,Rd}$ is not greater than 0,25 times the design moment resistance required for a full-strength joint, provided that it also has sufficient rotation capacity.

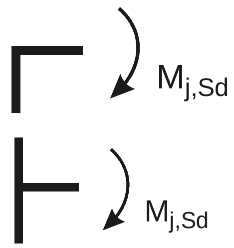
5.2.3.3 Full-strength joints

(1) The design resistance of a full strength joint shall be not less than that of the connected members.

(2) A joint may be classified as full-strength if it meets the criteria given in Figure 5.5.

5.2.3.4 Partial-strength joints

(1) A joint which does not meet the criteria for a full-strength joint or a nominally pinned joint should be classified as a partial-strength joint.



a) Top of column

Either $M_{j,Rd} \geq M_{b,p\ell,Rd}$

or $M_{j,Rd} \geq M_{c,p\ell,Rd}$

b) Within column height

Either $M_{j,Rd} \geq M_{b,p\ell,Rd}$

or $M_{j,Rd} \geq 2\, M_{c,p\ell,Rd}$

Key:

$M_{b,p\ell,Rd}$ is the design plastic moment resistance of a beam;
$M_{c,p\ell,Rd}$ is the design plastic moment resistance of a column.

Figure 5.5: Full-strength joints

5.3 Modelling of beam-to-column joints

(1) To model the deformational behaviour of a joint, account should be taken of the shear deformation of the web panel and the rotational deformation of the connections.

(2) Joint configurations should be designed to resist the internal bending moments $M_{b1,Ed}$ and $M_{b2,Ed}$, normal forces $N_{b1,Ed}$ and $N_{b2,Ed}$ and shear forces $V_{b1,Ed}$ and $V_{b2,Ed}$ applied to the connections by the connected members, see Figure 5.6.

(3) The resulting shear force $V_{wp,Ed}$ in the web panel should be obtained using:

$$V_{wp,Ed} = (M_{b1,Ed} - M_{b2,Ed})/z - (V_{c1,Ed} - V_{c2,Ed})/2 \quad \text{... (5.3)}$$

where:

z is the lever arm, see 6.2.7.

(4) To model a joint in a way that closely reproduces the expected behaviour, the web panel in shear and each of the connections should be modelled separately, taking account of the internal moments and forces in the members, acting at the periphery of the web panel, see Figure 5.6(a) and Figure 5.7.

(5) As a simplified alternative to 5.3(4), a single-sided joint configuration may be modelled as a single joint, and a double-sided joint configuration may be modelled as two separate but inter-acting joints, one on each side. As a consequence a double-sided beam-to-column joint configuration has two moment-rotation characteristics, one for the right-hand joint and another for the left-hand joint.

(6) In a double-sided, beam-to-column joint each joint should be modelled as a separate rotational spring, as shown in Figure 5.8, in which each spring has a moment-rotation characteristic that takes into account the behaviour of the web panel in shear as well as the influence of the relevant connection.

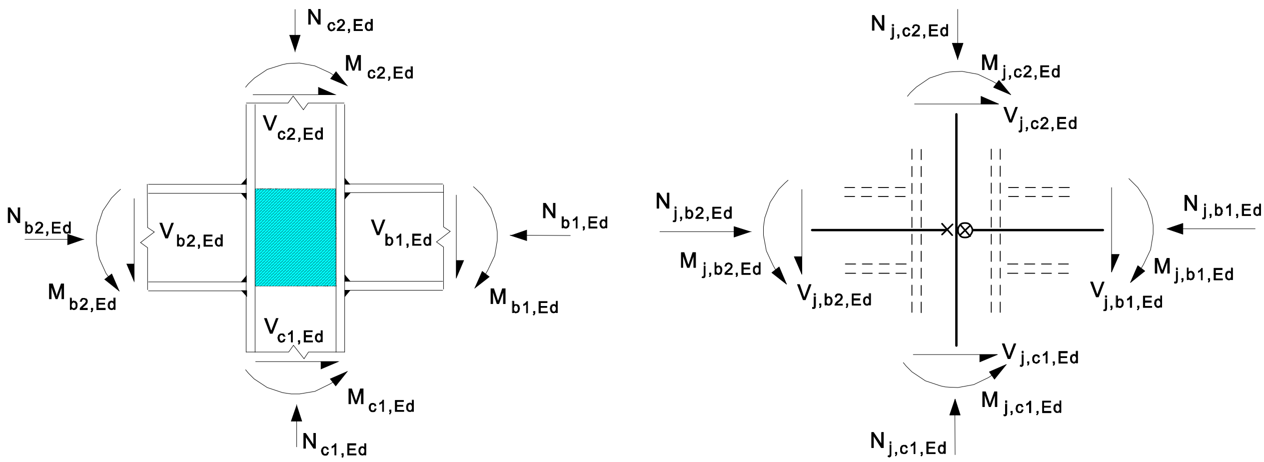
(7) When determining the design moment resistance and rotational stiffness for each of the joints, the possible influence of the web panel in shear should be taken into account by means of the transformation parameters β_1 and β_2, where:

β_1 is the value of the transformation parameter β for the right-hand side joint;

β_2 is the value of the transformation parameter β for the left-hand side joint.

NOTE: The transformation parameters β_1 and β_2 are used directly in 6.2.7.2(7) and 6.3.2(1). They are also used in 6.2.6.2(4) and 6.2.6.3(4) in connection with Table 6.3 to obtain the reduction factor ω for shear.

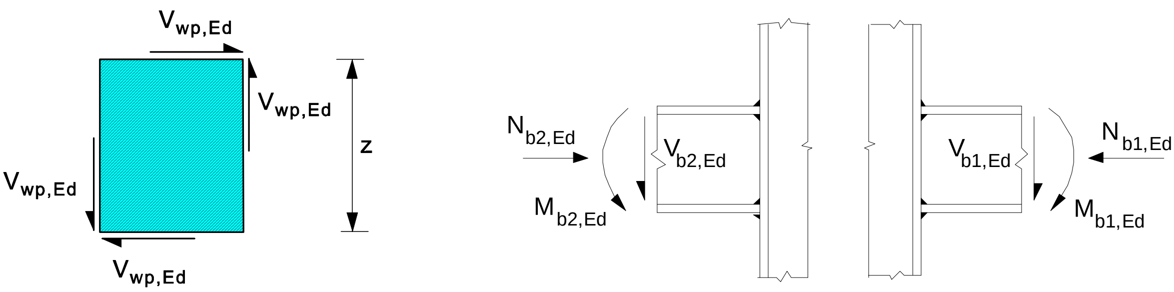
(8) Approximate values for β_1 and β_2 based on the values of the beam moments $M_{b1,Ed}$ and $M_{b2,Ed}$ at the periphery of the web panel, see Figure 5.6(a), may be obtained from Table 5.4.



a) Values at periphery of web panel b) Values at intersection of member centrelines

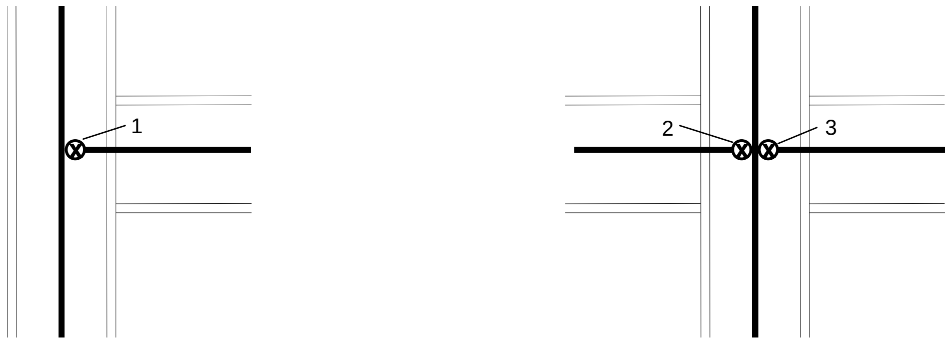
Direction of forces and moments are considered as positive in relation to equations (5.3) and (5.4)

Figure 5.6: Forces and moments acting on the joint



a) Shear forces in web panel b) Connections, with forces and moments in beams

Figure 5.7: Forces and moments acting on the web panel at the connections



Single-sided joint configuration

Double-sided joint configuration

1 Joint
2 Joint 2: left side
3 Joint 1: right side

Figure 5.8: Modelling the joint

(9) As an alternative to 5.3(8), more accurate values of β_1 and β_2 based on the values of the beam moments $M_{j,b1,Ed}$ and $M_{j,b2,Ed}$ at the intersection of the member centrelines, may be determined from the simplified model shown in Figure 5.6(b) as follows:

$$\beta_1 = \left|1 - M_{j,b2,Ed} / M_{j,b1,Ed}\right| \leq 2 \quad \text{... (5.4a)}$$

$$\beta_2 = \left|1 - M_{j,b1,Ed} / M_{j,b2,Ed}\right| \leq 2 \quad \text{... (5.4b)}$$

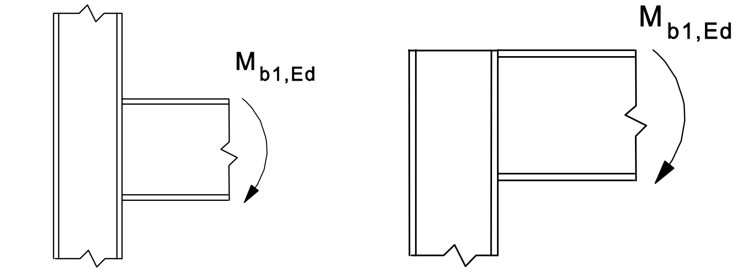
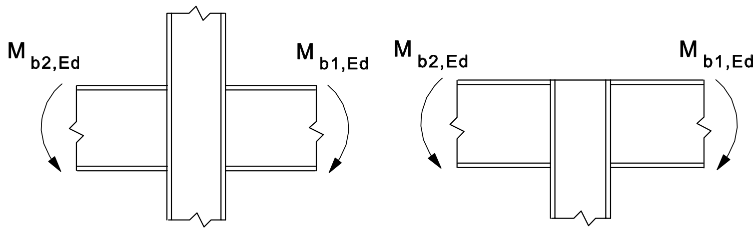
where:

$M_{j,b1,Ed}$ is the moment at the intersection from the right hand beam;

$M_{j,b2,Ed}$ is the moment at the intersection from the left hand beam.

(10) In the case of an unstiffened double-sided beam-to-column joint configuration in which the depths of the two beams are not equal, the actual distribution of shear stresses in the column web panel should be taken into account when determining the design moment resistance.

Table 5.4: Approximate values for the transformation parameter β

Type of joint configuration	Action	Value of β
	$M_{b1,Ed}$	$\beta \approx 1$
	$M_{b1,Ed} = M_{b2,Ed}$	$\beta = 0$ *)
	$M_{b1,Ed} / M_{b2,Ed} > 0$	$\beta \approx 1$
	$M_{b1,Ed} / M_{b2,Ed} < 0$	$\beta \approx 2$
	$M_{b1,Ed} + M_{b2,Ed} = 0$	$\beta \approx 2$
*) In this case the value of β is the exact value rather than an approximation.		

6 Structural joints connecting H or I sections

6.1 General

6.1.1 Basis

(1) This section contains design methods to determine the structural properties of joints in frames of any type. To apply these methods, a joint should be modelled as an assembly of basic components, see 1.3(1).

(2) The basic components used in this Standard are identified in Table 6.1 and their properties should be determined in accordance with the provisions given in this Standard. Other basic components may be used provided their properties are based on tests or analytical and numerical methods supported by tests, see EN 1990.

NOTE: The design methods for basic joint components given in this Standard are of general application and can also be applied to similar components in other joint configurations. However the specific design methods given for determining the design moment resistance, rotational stiffness and rotation capacity of a joint are based on an assumed distribution of internal forces for joint configurations indicated in Figure 1.2. For other joint configurations, design methods for determining the design moment resistance, rotational stiffness and rotation capacity should be based on appropriate assumptions for the distribution of internal forces.

6.1.2 Structural properties

6.1.2.1 Design moment-rotation characteristic

(1) A joint may be represented by a rotational spring connecting the centre lines of the connected members at the point of intersection, as indicated in Figure 6.1(a) and (b) for a single-sided beam-to-column joint configuration. The properties of the spring can be expressed in the form of a design moment-rotation characteristic that describes the relationship between the bending moment $M_{j,Ed}$ applied to a joint and the corresponding rotation ϕ_{Ed} between the connected members. Generally the design moment-rotation characteristic is non-linear as indicated in Figure 6.1(c).

(2) A design moment-rotation characteristic, see Figure 6.1(c) should define the following three main structural properties:

- moment resistance;
- rotational stiffness;
- rotation capacity.

NOTE: In certain cases the actual moment-rotation behaviour of a joint includes some rotation due to such effects as bolt slip, lack of fit and, in the case of column bases, foundation-soil interactions. This can result in a significant amount of initial hinge rotation that may need to be included in the design moment-rotation characteristic.

(3) The design moment-rotation characteristics of a beam-to-column joint shall be consistent with the assumptions made in the global analysis of the structure and with the assumptions made in the design of the members, see EN 1993-1-1.

(4) The design moment-rotation characteristic for joints and column bases of I and H sections as obtained from 6.3.1(4) may be assumed to satisfy the requirements of 5.1.1(4) for simplifying this characteristic for global analysis purposes.

6.1.2.2 Design Moment resistance

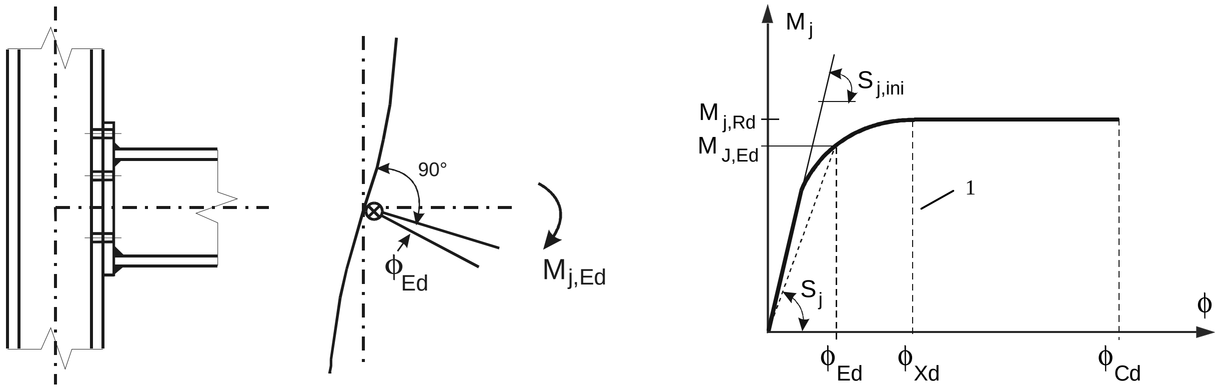
(1) The design moment resistance $M_{j,Rd}$, which is equal to the maximum moment of the design moment-rotation characteristic, see Figure 6.1(c), should be taken as that given by 6.1.3(4)

6.1.2.3 Rotational stiffness

(1) The rotational stiffness S_j, which is the secant stiffness as indicated in Figure 6.1(c), should be taken as that given by 5.1.1(4). For a design moment-rotation characteristic this definition of S_j applies up to the rotation ϕ_{Xd} at which $M_{j,Ed}$ first reaches $M_{j,Rd}$, but not for larger rotations, see Figure 6.1(c). The initial rotational stiffness $S_{j,ini}$, which is the slope of the elastic range of the design moment-rotation characteristic, should be taken as that given by 6.1.3(4).

6.1.2.4 Rotation capacity

(1) The design rotation capacity ϕ_{Cd} of a joint, which is equal to the maximum rotation of the design moment-rotation characteristic, see Figure 6.1(c), should be taken as that given by 6.1.3(4).



1 Limit for S_j

a) Joint b) Model c) Design moment-rotation characteristic

Figure 6.1: Design moment-rotation characteristic for a joint

6.1.3 Basic components of a joint

(1) The design moment-rotation characteristic of a joint should depend on the properties of its basic components, which should be among those identified in 6.1.3(2).

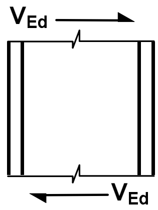
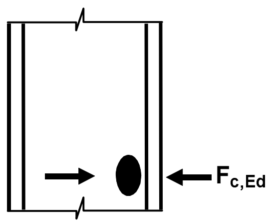
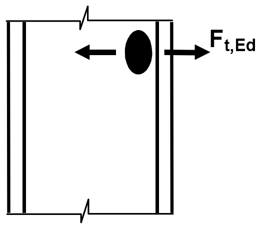
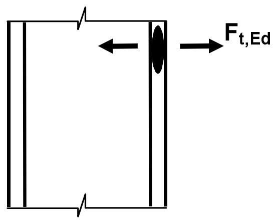
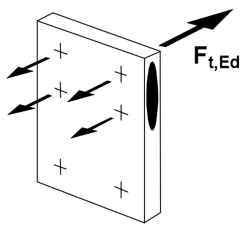
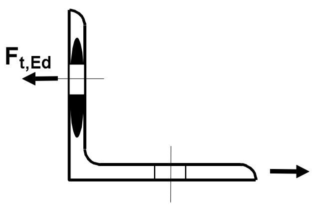
(2) The basic joint components should be those identified in Table 6.1, together with the reference to the application rules which should be used for the evaluation of their structural properties.

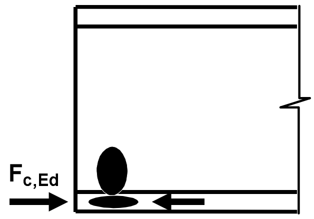
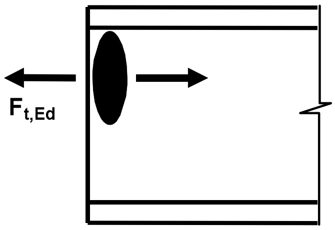
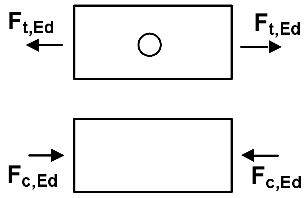
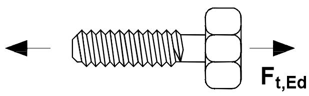
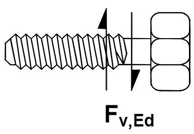
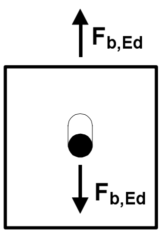
(3) Certain joint components may be reinforced. Details of the different methods of reinforcement are given in 6.2.4.3 and 6.2.6.

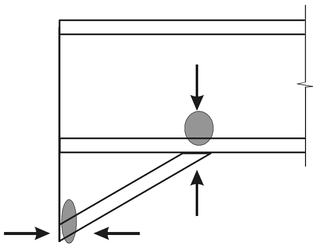
(4) The relationships between the properties of the basic components of a joint and the structural properties of the joint should be those given in the following clauses:

- for moment resistance in 6.2.7 and 6.2.8;
- for rotational stiffness in 6.3.1;
- for rotation capacity in 6.4.

Table 6.1: Basic joint components

Component			Reference to application rules		
			Design Resistance	Stiffness coefficient	Rotation capacity
1	Column web panel in shear		6.2.6.1	6.3.2	6.4(4)
2	Column web In transverse compression		6.2.6.2	6.3.2	6.4(5) and 6.4(6)
3	Column web in transverse tension		6.2.6.3	6.3.2	6.4(5)
4	Column flange in bending		6.2.6.4	6.3.2	6.4(7)
5	End-plate in bending		6.2.6.5	6.3.2	6.4(7)
6	Flange cleat in bending		6.2.6.6	6.3.2	6.4(7)

Component			Reference to application rules		
			Design Resistance	Stiffness coefficient	Rotation capacity
7	Beam or column flange and web in compression		6.2.6.7	6.3.2	*)
8	Beam web in tension		6.2.6.8	6.3.2	*)
9	Plate in tension or compression		in tension: - EN 1993-1-1 in compression: - EN 1993-1-1	6.3.2	*)
10	Bolts in tension		With column flange: - 6.2.6.4 with end-plate: - 6.2.6.5 with flange cleat: - 6.2.6.6	6.3.2	6.4(7)
11	Bolts in shear		3.6	6.3.2	6.4(2)
12	Bolts in bearing (on beam flange, column flange, end-plate or cleat)		3.6	6.3.2	*)
*)	No information available in this part.				

Component			Reference to application rules		
			Design Resistance	Stiffness coefficient	Rotation capacity
13	Concrete in compression including grout		6.2.6.9	6.3.2	*)
14	Base plate in bending under compression		6.2.6.10	6.3.2	*)
15	Base plate in bending under tension		6.2.6.11	6.3.2	*)
16	Anchor bolts in tension		6.2.6.12	6.3.2	*)
17	Anchor bolts in shear		6.2.2	*)	*)
18	Anchor bolts in bearing		6.2.2	*)	*)
19	Welds		4	6.3.2	*)
20	Haunched beam		6.2.6.7	6.3.2	*)
*) No information available in this part.					

6.2 Design Resistance

6.2.1 Internal forces

(1) The stresses due to the internal forces and moments in a member may be assumed not to affect the design resistances of the basic components of a joint, except as specified in 6.2.1(2) and 6.2.1(3).

(2) The longitudinal stress in a column should be taken into account when determining the design resistance of the column web in compression, see 6.2.6.2(2).

(3) The shear in a column web panel should be taken into account when determining the design resistance of the following basic components:

– column web in transverse compression, see 6.2.6.2;

– column web in transverse tension, see 6.2.6.3.

6.2.2 Shear forces

(1) In welded connections, and in bolted connections with end-plates, the welds connecting the beam web should be designed to transfer the shear force from the connected beam to the joint, without any assistance from the welds connecting the beam flanges.

(2) In bolted connections with end-plates, the design resistance of each bolt-row to combined shear and tension should be verified using the criterion given in Table 3.4, taking into account the total tensile force in the bolt, including any force due to prying action.

NOTE: As a simplification, bolts required to resist in tension may be assumed to provide their full design resistance in tension when it can be shown that the design shear force does not exceed the sum of:

a) the total design shear resistance of those bolts that are not required to resist tension and;

b) (0,4/1,4) times the total design shear resistance of those bolts that are also required to resist tension.

(3) In bolted connections with angle flange cleats, the cleat connecting the compression flange of the beam may be assumed to transfer the shear force in the beam to the column, provided that:

– the gap g between the end of the beam and the face of the column does not exceed the thickness t_a of the angle cleat;

– the force does not exceed the design shear resistance of the bolts connecting the cleat to the column;

– the web of the beam satisfies the requirement given in EN 1993-1-5, section 6.

(4) The design shear resistance of a joint may be derived from the distribution of internal forces within that joint, and the design resistances of its basic components to these forces, see Table 6.1.

(5) In base plates if no special elements for resisting shear are provided, such as block or bar shear connectors, it should be demonstrated that either the design friction resistance of the base plate, see 6.2.2(6), or, in cases where the bolt holes are not oversized, the design shear resistance of the anchor bolts, see 6.2.2(7), is sufficient to transfer the design shear force. The design bearing resistance of the block or bar shear connectors with respect to the concrete should be checked according to EN 1992.

(6) In a column base the design friction resistance $F_{f,Rd}$ between base plate and grout should be derived as follows:

$$F_{f,Rd} = C_{f,d}\ N_{c,Ed} \qquad \text{... (6.1)}$$

where:

$C_{f,d}$ is the coefficient of friction between base plate and grout layer. The following values may be used:

- for sand-cement mortar $C_{f,d} = 0,20$
- for other types of grout the coefficient of friction $C_{f,d}$ should be determined by testing in accordance with EN 1990, Annex D;

$N_{c,Ed}$ is the design value of the normal compressive force in the column.

NOTE: If the column is loaded by a tensile normal force, $F_{f,Rd} = 0$.

(7) In a column base the design shear resistance of an anchor bolt $F_{vb,Rd}$ should be taken as the smaller of $F_{1,vb,Rd}$ and $F_{2,vb,Rd}$ where

- $F_{1,vb,Rd}$ is the design bearing resistance of the anchor bolt, see 3.6.1
- $$F_{2,vb,Rd} = \frac{\alpha_b \, f_{ub} \, A_s}{\gamma_{Mb}} \quad \text{... (6.2)}$$

where:

$\alpha_b = 0,44 - 0,0003 \, f_{yb}$

f_{yb} is the yield strength of the anchor bolt, where 235 N/mm$^2 \leq f_{yb} \leq$640 N/mm^2

(8) The design shear resistance $F_{v,Rd}$ of a column base plate should be derived as follows:

$$F_{v,Rd} = F_{f,Rd} + n \, F_{vb,Rd} \quad \text{... (6.3)}$$

where:

n is the number of anchor bolts in the base plate.

(9) The concrete and reinforcement used in the base should be designed in accordance with EN 1992.

6.2.3 Bending moments

(1) The design moment resistance of any joint may be derived from the distribution of internal forces within that joint and the design resistances of its basic components to these forces, see Table 6.1.

(2) Provided that the axial force N_{Ed} in the connected member does not exceed 5% of the design resistance $N_{p\ell,Rd}$ of its cross-section, the design moment resistance $M_{j,Rd}$ of a beam-to column joint or beam splice may be determined using the method given in 6.2.7.

(3) The design moment resistance $M_{j,Rd}$ of a column base may be determined using the method given in 6.2.8.

(4) In all joints, the sizes of the welds should be such that the design moment resistance of the joint $M_{j,Rd}$ is always limited by the design resistance of its other basic components, and not by the design resistance of the welds.

(5) In a beam-to-column joint or beam splice in which a plastic hinge is required to form and rotate under any relevant load case, the welds should be designed to resist the effects of a moment equal to the smaller of:

- the design plastic moment resistance of the connected member $M_{p\ell,Rd}$
- α times the design moment resistance of the joint $M_{j,Rd}$

where

$\alpha = 1,4$ - for frames in which the bracing system satisfies the criterion (5.1) in EN1993-1-1 clause 5.2.1(3) with respect to sway;

$\alpha = 1,7$ - for all other cases.

(6) In a bolted connection with more than one bolt-row in tension, as a simplification the contribution of any bolt-row may be neglected, provided that the contributions of all other bolt-rows closer to the centre of compression are also neglected.

6.2.4 Equivalent T-stub in tension

6.2.4.1 General

(1) In bolted connections an equivalent T-stub in tension may be used to model the design resistance of the following basic components:

- column flange in bending;
- end-plate in bending;
- flange cleat in bending;
- base plate in bending under tension.

(2) Methods for modelling these basic components as equivalent T-stub flanges, including the values to be used for e_{min}, ℓ_{eff} and m, are given in 6.2.6.

(3) The possible modes of failure of the flange of an equivalent T-stub may be assumed to be similar to those expected to occur in the basic component that it represents.

(4) The total effective length $\sum\ell_{eff}$ of an equivalent T-stub, see Figure 6.2, should be such that the design resistance of its flange is equivalent to that of the basic joint component that it represents.

NOTE: The effective length of an equivalent T-stub is a notional length and does not necessarily correspond to the physical length of the basic joint component that it represents.

(5) The design tension resistance of a T-stub flange should be determined from Table 6.2.

NOTE: Prying effects are implicitly taken into account when determining the design tension resistance according to Table 6.2.

(6) In cases where prying forces may develop, see Table 6.2, the design tension resistance of a T-stub flange $F_{T,Rd}$ should be taken as the smallest value for the three possible failure modes 1, 2 and 3.

(7) In cases where prying forces may not develop, see Table 6.2, the design tension resistance of a T-stub flange $F_{T,Rd}$ should be taken as the smallest value for the two possible failure modes 1-2 and 3.

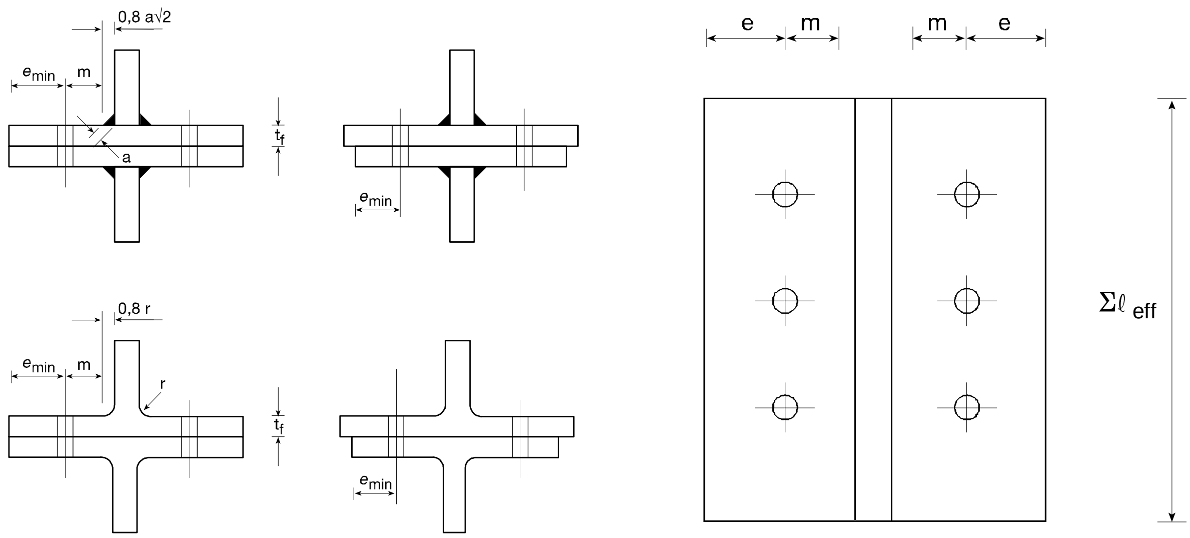


Figure 6.2: Dimensions of an equivalent T-stub flange

Table 6.2: Design Resistance of a T-stub flange

	Prying forces may develop, i.e. $L_b \le L_b^*$		No prying forces
Mode 1	Method 1	Method 2 (alternative method)	$F_{T,1\text{-}2,Rd} = \dfrac{2M_{p\ell,1,Rd}}{m}$
without backing plates	$F_{T,1,Rd} = \dfrac{4M_{p\ell,1,Rd}}{m}$	$F_{T,1,Rd} = \dfrac{(8n-2e_w)\,M_{p\ell,1,Rd}}{2mn-e_w\,(m+n)}$	
with backing plates	$F_{T,1,Rd} = \dfrac{4M_{p\ell,1,Rd}+2M_{bp,Rd}}{m}$	$F_{T,1,Rd} = \dfrac{(8n-2e_w)M_{p\ell,1,Rd}+4nM_{bp,Rd}}{2mn-e_w(m+n)}$	
Mode 2	$F_{T,2,Rd} = \dfrac{2M_{p\ell,2,Rd}+n\Sigma F_{t,Rd}}{m+n}$		
Mode 3	$F_{T,3,Rd} = \Sigma F_{t,Rd}$		

Mode 1: Complete yielding of the flange
Mode 2: Bolt failure with yielding of the flange
Mode 3: Bolt failure

L_b is
- the bolt elongation length, taken equal to the grip length (total thickness of material and washers), plus half the sum of the height of the bolt head and the height of the nut or
- the anchor bolt elongation length, taken equal to the sum of 8 times the nominal bolt diameter, the grout layer, the plate thickness, the washer and half the height of the nut

$L_b^* = \dfrac{8{,}8m^3A_s}{\Sigma\ell_{eff,1}t_f^{\,3}}$

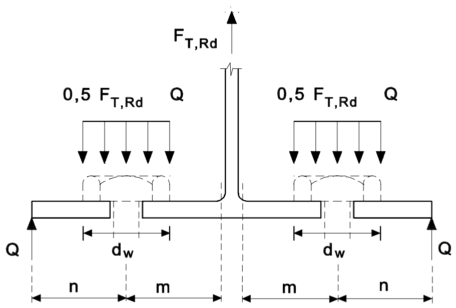
$F_{T,Rd}$ is the design tension resistance of a T-stub flange
Q is the prying force

$M_{p\ell,1,Rd} = 0{,}25\Sigma\ell_{eff,1}t_f^{\,2}f_y/\gamma_{M0}$

$M_{p\ell,2,Rd} = 0{,}25\Sigma\ell_{eff,2}\,t_f^{\,2}f_y/\gamma_{M0}$

$M_{bp,Rd} = 0{,}25\Sigma\ell_{eff,1}t_{bp}^{\,2}f_{y,bp}/\gamma_{M0}$

$n = e_{min}$ but $n \le 1{,}25m$
$F_{t,Rd}$ is the design tension resistance of a bolt, see Table 3.4;
$\sum F_{t,Rd}$ is the total value of $F_{t,Rd}$ for all the bolts in the T-stub;
$\sum \ell_{eff,1}$ is the value of $\sum \ell_{eff}$ for mode 1;
$\sum \ell_{eff,2}$ is the value of $\sum \ell_{eff}$ for mode 2;
e_{min}, m and t_f are as indicated in Figure 6.2.
$f_{y,bp}$ is the yield strength of the backing plates;
t_{bp} is the thickness of the backing plates;
$e_w = d_w/4$;
d_w is the diameter of the washer, or the width across points of the bolt head or nut, as relevant.



NOTE 1: In bolted beam-to-column joints or beam splices it may be assumed that prying forces will develop.
NOTE 2: In method 2, the force applied to the T-stub flange by a bolt is assumed to be uniformly distributed under the washer, the bolt head or the nut, as appropriate, see figure, instead of concentrated at the centre-line of the bolt. This assumption leads to a higher value for mode 1, but leaves the values for $F_{T,1\text{-}2,Rd}$ and modes 2 and 3 unchanged.

6.2.4.2 Individual bolt-rows, bolt-groups and groups of bolt-rows

(1) Although in an actual T-stub flange the forces at each bolt-row are generally equal, when an equivalent T-stub flange is used to model a basic component listed in 6.2.4.1(1), allowance should be made for the different in forces at each bolt-row.

(2) When using the equivalent T-stub approach to model a group of bolt rows it may be necessary to divide the group in to separate bolt-rows and use an equivalent T-stub to model each separate bolt-row.

(3) When using the T-stub approach to model a group of bolt rows the following conditions should be satisfied:

a) the force at each bolt-row should not exceed the design resistance determined considering only that individual bolt-row;

b) the total force on each group of bolt-rows, comprising two or more adjacent bolt-rows within the same bolt-group, should not exceed the design resistance of that group of bolt-rows.

(4) When determining the design tension resistance of a basic component represented by an equivalent T-stub flange, the following parameters should be calculated:

a) the maximum design resistance of an individual bolt-row, determined considering only that bolt-row;

b) the contribution of each bolt-row to the maximum design resistance of two or more adjacent bolt-rows within a bolt-group, determined considering only those bolt-rows.

(5) In the case of an individual bolt-row $\sum \ell_{eff}$ should be taken as equal to the effective length ℓ_{eff} tabulated in 6.2.6 for that bolt-row taken as an individual bolt-row.

(6) In the case of a group of bolt-rows $\sum \ell_{eff}$ should be taken as the sum of the effective lengths ℓ_{eff} tabulated in 6.2.6 for each relevant bolt-row taken as part of a bolt-group.

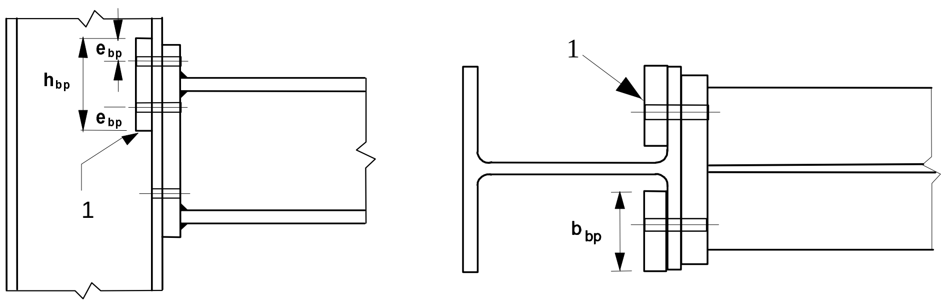
6.2.4.3 Backing plates

(1) Backing plates may be used to reinforce a column flange in bending as indicated in Figure 6.3.

(2) Each backing plate should extend at least to the edge of the column flange, and to within 3 mm of the toe of the root radius or of the weld.

(3) The backing plate should extend beyond the furthermost bolt rows active in tension as defined in Figure 6.3.

(4) Where backing plates are used, the design resistance of the T-stub $F_{T,Rd}$ should be determined using the method given in Table 6.2.



$h_{bp} \geq \sum \ell_{eff,1}$

$e_{bp} \geq 2d$

1 Backing plate

Figure 6.3: Column flange with backing plates

6.2.5 Equivalent T-stub in compression

(1) In steel- to-concrete joints, the flange of an equivalent T-stub in compression may be used to model the design resistances for the combination of the following basic components:

– the steel base plate in bending under the bearing pressure on the foundation,

– the concrete and/or grout joint material in bearing.

(2) The total length l_{eff} and the total width b_{eff} of an equivalent T-stub should be such that the design compression resistance of the T-stub is equivalent to that of the basic joint component it represents.

NOTE: The effective length and the effective width of an equivalent T-stub are notional lengths and may be smaller than or equal to the physical dimensions of the basic joint component it represents.

(3) The design compression resistance of a T-stub flange $F_{C,Rd}$ should be determined as follows:

$$F_{C,Rd} = f_{jd}\, b_{eff}\, l_{eff} \qquad \text{... (6.4)}$$

where:

b_{eff} is the effective width of the T-stub flange, see 6.2.5(5) and 6.2.5(6)

l_{eff} is the effective length of the T-stub flange, see 6.2.5(5) and 6.2.5(6)

f_{jd} is the design bearing strength of the joint, see 6.2.5(7)

(4) The forces transferred through a T-stub should be assumed to spread uniformly as shown in Figure 6.4(a) and (b). The pressure on the resulting bearing area should not exceed the design bearing strength f_j and the additional bearing width, c, should not exceed:

$$c = t\,[f_y / (3\, f_j\, \gamma_{M0})]^{0.5} \qquad \text{... (6.5)}$$

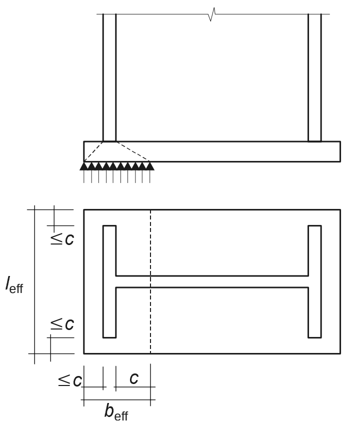
where:

t is the thickness of the T-stub flange;

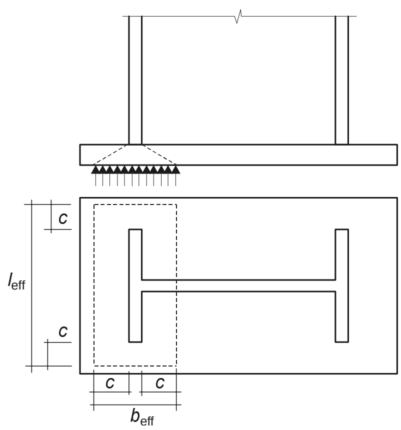
f_y is the yield strength of the T-stub flange.

(5) Where the projection of the physical length of the basic joint component represented by the T-stub is less than c, the effective area should be taken as indicated in Figure 6.4(a)

(6) Where the projection of the physical length of the basic joint component represented by the T-stub exceeds c on any side, the part of the additional projection beyond the width c should be neglected, see Figure 6.4(b).



(a) Short projection



(b) Large projection

Figure 6.4: Area of equivalent T-Stub in compression

(7) The design bearing strength of the joint f_{jd} should be determined from:

$$f_{jd} = \beta_j F_{Rdu} / (b_{eff} l_{eff}) \quad \text{... (6.6)}$$

where:

β_j is the foundation joint material coefficient, which may be taken as 2/3 provided that the characteristic strength of the grout is not less than 0,2 times the characteristic strength of the concrete foundation and the thickness of the grout is not greater than 0,2 times the smallest width of the steel base plate. In cases where the thickness of the grout is more than 50 mm, the characteristic strength of the grout should be at least the same as that of the concrete foundation.

F_{Rdu} is the concentrated design resistance force given in EN 1992, where A_{c0} is to be taken as (b_{eff} l_{eff}).

6.2.6 Design Resistance of basic components

6.2.6.1 Column web panel in shear

(1) The design methods given in 6.2.6.1(2) to 6.2.6.1(14) are valid provided the column web slenderness satisfies the condition $d/t_w \leq 69\varepsilon$.

(2) For a single-sided joint, or for a double-sided joint in which the beam depths are similar, the design shear resistance $V_{wp,Rd}$ of an unstiffened column web panel, subject to a design shear force $V_{wp,Ed}$, see 5.3(3), should be obtained using:

$$V_{wp,Rd} = \frac{0,9 f_{y,wc} A_{vc}}{\sqrt{3}\,\gamma_{M0}} \quad \text{... (6.7)}$$

where:

A_{vc} is the shear area of the column, see EN 1993-1-1.

(3) The design shear resistance may be increased by the use of stiffeners or supplementary web plates.

(4) Where transverse web stiffeners are used in both the compression zone and the tension zone, the design plastic shear resistance of the column web panel $V_{wp,Rd}$ may be increased by $V_{wp,add,Rd}$ given by:

$$V_{wp,add,Rd} = \frac{4 M_{p\ell,fc,Rd}}{d_s} \quad \text{but} \quad V_{wp,add,Rd} \leq \frac{2 M_{p\ell,fc,Rd} + 2 M_{p\ell,st,Rd}}{d_s} \quad \text{... (6.8)}$$

where:

d_s is the distance between the centrelines of the stiffeners;

$M_{p\ell,fc,Rd}$ is the design plastic moment resistance of a column flange

$M_{p\ell,st,Rd}$ is the design plastic moment resistance of a stiffener.

NOTE: In welded joints, the transverse stiffeners should be aligned with the corresponding beam flange.

(5) When diagonal web stiffeners are used the design shear resistance of a column web should be determined according to EN 1993-1-1.

NOTE: In double-sided beam-to-column joint configurations without diagonal stiffeners on the column webs, the two beams are assumed to have similar depths.

(6) Where a column web is reinforced by adding a supplementary web plate, see Figure 6.5, the shear area A_{vc} may be increased by $b_s t_{wc}$. If a further supplementary web plate is added on the other side of the web, no further increase of the shear area should be made.

(7) Supplementary web plates may also be used to increase the rotational stiffness of a joint by increasing the stiffness of the column web in shear, compression or tension, see 6.3.2(1).

(8) The steel grade of the supplementary web plate should be similar to that of the column.

(9) The width b_s should be such that the supplementary web plate extends at least to the toe of the root radius.

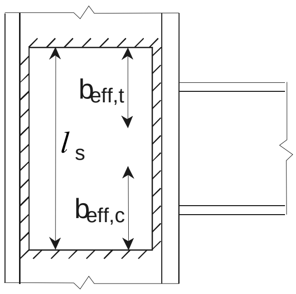
(10) The length ℓ_s should be such that the supplementary web plate extends throughout the effective width of the web in tension and compression, see Figure 6.5.

(11) The thickness t_s of the supplementary web plate should be not less than the column web thickness t_{wc}.

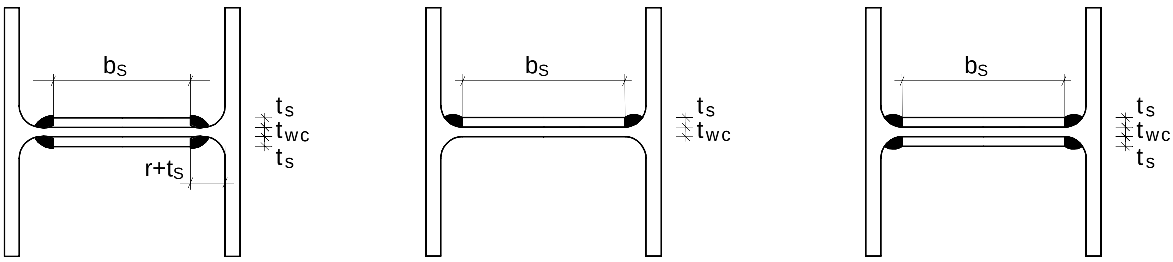
(12) The welds between the supplementary web plate and profile should be designed to resist the applied design forces.

(13) The width b_s of a supplementary web plate should be less than $40\varepsilon\, t_s$.

(14) Discontinuous welds may be used in non corrosive environments.



a) Layout



NOTE: Weldability at the corner should be taken into account.

b) Examples of cross-section with longitudinal welds

Figure 6.5: Examples of supplementary web plates

6.2.6.2 Column web in transverse compression

(1) The design resistance of an unstiffened column web subject to transverse compression should be determined from:

$$F_{c,wc,Rd} = \frac{\omega\, k_{wc}\, b_{eff,c,wc}\, t_{wc}\, f_{y,wc}}{\gamma_{M0}} \quad \text{but} \quad F_{c,wc,Rd} \le \frac{\omega\, k_{wc}\, \rho\, b_{eff,c,wc}\, t_{wc}\, f_{y,wc}}{\gamma_{M1}} \qquad \text{... (6.9)}$$

where:

ω is a reduction factor to allow for the possible effects of interaction with shear in the column web panel according to Table 6.3;

$b_{\mathrm{eff,c,wc}}$ is the effective width of column web in compression

– for a welded connection:

$$b_{\mathrm{eff,c,wc}} = t_{fb} + 2\sqrt{2}\,a_b + 5(t_{fc} + s) \quad \text{... (6.10)}$$

a_{c}, r_{c} and a_{b} are as indicated in Figure 6.6.

– for bolted end-plate connection:

$$b_{\mathrm{eff,c,wc}} = t_{fb} + 2\sqrt{2}\,a_p + 5(t_{fc} + s) + s_p \quad \text{... (6.11)}$$

s_{p} is the length obtained by dispersion at 45° through the end-plate (at least t_{p} and, provided that the length of end-plate below the flange is sufficient, up to $2t_{\mathrm{p}}$).

– for bolted connection with angle flange cleats:

$$b_{\mathrm{eff,c,wc}} = 2t_a + 0{,}6r_a + 5(t_{fc} + s) \quad \text{... (6.12)}$$

– for a rolled I or H section column: $s = r_{\mathrm{c}}$

– for a welded I or H section column: $s = \sqrt{2}a_c$

ρ is the reduction factor for plate buckling:

– if $\bar{\lambda}_p \leq 0{,}72$: $\rho = 1{,}0$ ... (6.13a)

– if $\bar{\lambda}_p > 0{,}72$: $\rho = (\bar{\lambda}_p - 0{,}2)/\bar{\lambda}_p^{\,2}$ ... (6.13b)

$\bar{\lambda}_p$ is the plate slenderness:

$$\bar{\lambda}_p = 0{,}932\sqrt{\frac{b_{eff,c,wc}\,d_{wc}\,f_{y,wc}}{E t_{wc}^{\,2}}} \quad \text{... (6.13c)}$$

– for a rolled I or H section column: $d_{\mathrm{wc}} = h_{\mathrm{c}} - 2(t_{\mathrm{fc}} + r_{\mathrm{c}})$

– for a welded I or H section column: $d_{\mathrm{wc}} = h_{\mathrm{c}} - 2(t_{\mathrm{fc}} + \sqrt{2}a)$

k_{wc} is a reduction factor and is given in 6.2.6.2(2).

Table 6.3: Reduction factor ω for interaction with shear

Transformation parameter β	Reduction factor ω
$0 \leq \beta \leq 0{,}5$	$\omega = 1$
$0{,}5 < \beta < 1$	$\omega = \omega_1 + 2(1-\beta)(1-\omega_1)$
$\beta = 1$	$\omega = \omega_1$
$1 < \beta < 2$	$\omega = \omega_1 + (\beta - 1)(\omega_2 - \omega_1)$
$\beta = 2$	$\omega = \omega_2$
$\omega_1 = \dfrac{1}{\sqrt{1+1{,}3(b_{eff,c,wc}\,t_{wc}/A_{vc})^2}}$	$\omega_2 = \dfrac{1}{\sqrt{1+5{,}2(b_{eff,c,wc}\,t_{wc}/A_{vc})^2}}$

A_{vc} is the shear area of the column, see 6.2.6.1;

β is the transformation parameter, see 5.3(7).

(2) Where the maximum longitudinal compressive stress $\sigma_{com,Ed}$ due to axial force and bending moment in the column exceeds $0{,}7 f_{y,wc}$ in the web (adjacent to the root radius for a rolled section or the toe of the weld for a welded section), its effect on the design resistance of the column web in compression should be allowed for by multiplying the value of $F_{c,wc,Rd}$ given by expression (6.9) by a reduction factor k_{wc} as follows:

– when $\sigma_{com,Ed} \leq 0{,}7 f_{y,wc}$: $k_{wc} = 1$

– when $\sigma_{com,Ed} > 0{,}7 f_{y,wc}$: $k_{wc} = 1{,}7 - \sigma_{com,Ed} / f_{y,wc}$ ... (6.14)

NOTE: Generally the reduction factor k_{wc} is 1,0 and no reduction is necessary. It can therefore be omitted in preliminary calculations when the longitudinal stress is unknown and checked later.

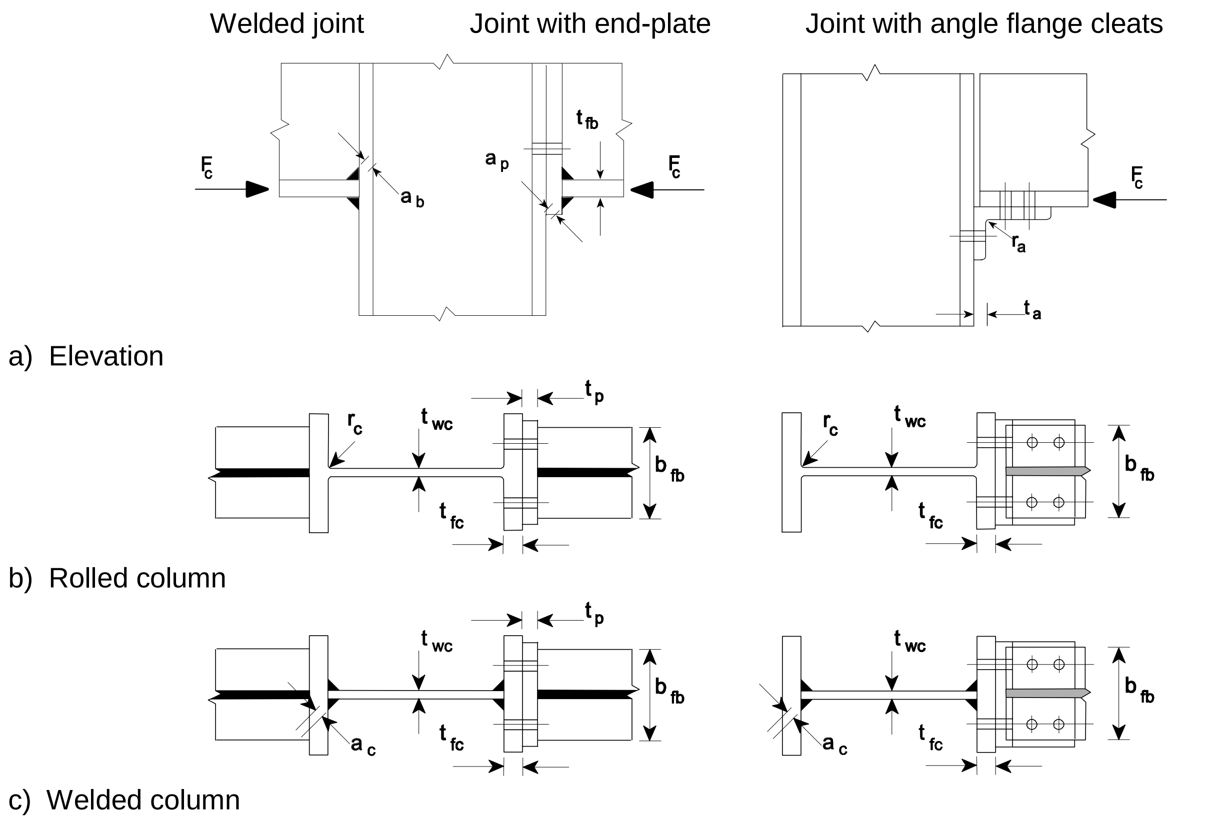


Figure 6.6: Transverse compression on an unstiffened column

(3) The ‘column-sway' buckling mode of an unstiffened column web in compression illustrated in Figure 6.7 should normally be prevented by constructional restraints.

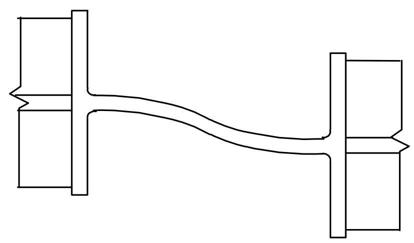


Figure 6.7: ‘Column-sway’ buckling mode of an unstiffened web

(4) Stiffeners or supplementary web plates may be used to increase the design resistance of a column web in transverse compression.

(5) Transverse stiffeners or appropriate arrangements of diagonal stiffeners may be used in association with or as an alternative to, transverse stiffeners in order to increase the design resistance of the column web in compression.

NOTE: In welded joints, the transverse stiffeners should be aligned with the corresponding beam flange. In bolted joints, the stiffener in the compression zone should be aligned with the centre of compression as defined Figure 6.15.

(6) Where an unstiffened column web is reinforced by adding a supplementary web plate conforming with 6.2.6.1, the effective thickness of the web may be taken as $1{,}5\, t_{wc}$ if one supplementary web plate is added, or $2{,}0\, t_{wc}$ if supplementary web plates are added to both sides of the web. In calculating the reduction factor ω for the possible effects of shear stress, the shear area A_{vc} of the web may be increased only to the extent permitted when determining its design shear resistance, see 6.2.6.1(6).

6.2.6.3 Column web in transverse tension

(1) The design resistance of an unstiffened column web subject to transverse tension should be determined from:

$$F_{t,wc,Rd} = \frac{\omega\, b_{eff,t,wc}\, t_{wc}\, f_{y,wc}}{\gamma_{M0}} \qquad \text{... (6.15)}$$

where:

ω is a reduction factor to allow for the interaction with shear in the column web panel.

(2) For a welded connection, the effective width $b_{eff,t,wc}$ of a column web in tension should be obtained using:

$$b_{eff,t,wc} = t_{fb} + 2\sqrt{2}\, a_b + 5(t_{fc} + s) \qquad \text{... (6.16)}$$

where:

– for a rolled I or H section column: $s = r_c$

– for a welded I or H section column: $s = \sqrt{2}\, a_c$

where:

a_c and r_c are as indicated in Figure 6.8 and a_b is as indicated in Figure 6.6.

(3) For a bolted connection, the effective width $b_{eff,t,wc}$ of column web in tension should be taken as equal to the effective length of equivalent T-stub representing the column flange, see 6.2.6.4.

(4) The reduction factor ω to allow for the possible effects of shear in the column web panel should be determined from Table 6.3, using the value of $b_{eff,t,wc}$ given in 6.2.6.3(2) or 6.2.6.3(3) as appropriate.

(5) Stiffeners or supplementary web plates may be used to increase the design tension resistance of a column web.

(6) Transverse stiffeners and/or appropriate arrangements of diagonal stiffeners may be used to increase the design resistance of the column web in tension.

NOTE: In welded joints, the transverse stiffeners should be aligned with the corresponding beam flange. In bolted joints, the stiffener in the compression zone should be aligned with the centre of compression as defined in Figure 6.15.

(7) The welds connecting diagonal stiffeners to the column flange should be fill-in welds with a sealing run providing a combined throat thickness equal to the thickness of the stiffeners.

(8) Where an unstiffened column web is reinforced by adding supplementary web plates conforming with 6.2.6.1, the design tension resistance depends on the throat thickness of the longitudinal welds connecting the supplementary web plates. The effective thickness of the web $t_{w,eff}$ should be taken as follows:

- when the longitudinal welds are full penetration butt welds with a throat thickness $a \geq t_s$ then:
  - for one supplementary web plate: $t_{w,eff} = 1{,}5\, t_{wc}$ ... (6.17)
  - for supplementary web plates both sides: $t_{w,eff} = 2{,}0\, t_{wc}$ ... (6.18)
- when the longitudinal welds are fillet welds with a throat thickness $a \geq t_s / \sqrt{2}$ then for either one or two supplementary web plates:
  - for steel grades S 235, S 275 or S 355: $t_{w,eff} = 1{,}4\, t_{wc}$ ... (6.19a)
  - for steel grades S 420 or S 460: $t_{w,eff} = 1{,}3\, t_{wc}$ ... (6.19b)

(9) In calculating the reduction factor ω for the possible effects of shear stress, the shear area A_{vc} of a column web reinforced by adding supplementary web plates may be increased only to the extent permitted when determining its design shear resistance, see 6.2.6.1(6).

6.2.6.4 Column flange in tranverse bending

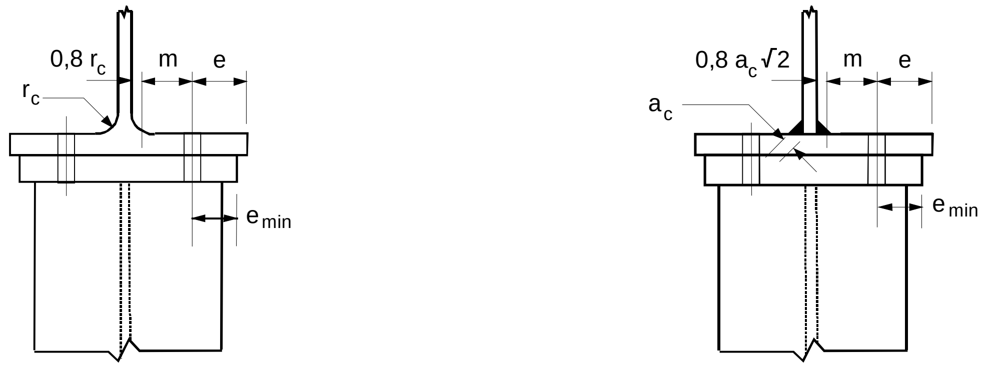
6.2.6.4.1 Unstiffened column flange, bolted connection

(1) The design resistance and failure mode of an unstiffened column flange in tranverse bending, together with the associated bolts in tension, should be taken as similar to those of an equivalent T-stub flange, see 6.2.4, for both:

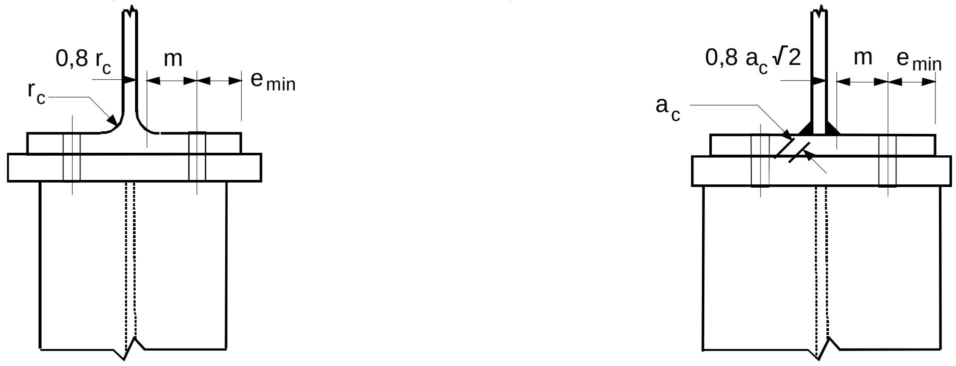
- each individual bolt-row required to resist tension;
- each group of bolt-rows required to resist tension.

(2) The dimensions e_{min} and m for use in 6.2.4 should be determined from Figure 6.8.

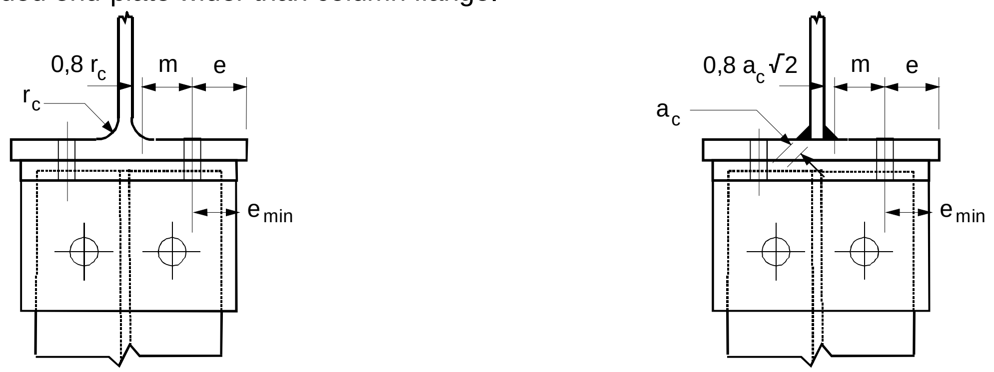
(3) The effective length of equivalent T-stub flange should be determined for the individual bolt-rows and the bolt-group in accordance with 6.2.4.2 from the values given for each bolt-row in Table 6.4.



a) Welded end-plate narrower than column flange.



b) Welded end-plate wider than column flange.



c) Angle flange cleats.

Figure 6.8: Definitions of e, e_{min}, r_c and m

Table 6.4: Effective lengths for an unstiffened column flange

Bolt-row Location	Bolt-row considered individually		Bolt-row considered as part of a group of bolt-rows	
	Circular patterns $\ell_{eff,cp}$	Non-circular patterns $\ell_{eff,nc}$	Circular patterns $\ell_{eff,cp}$	Non-circular patterns $\ell_{eff,nc}$
Inner bolt-row	$2\pi m$	$4m + 1{,}25e$	$2p$	p
End bolt-row	The smaller of: $2\pi m$ $\pi m + 2e_1$	The smaller of: $4m + 1{,}25e$ $2m + 0{,}625e + e_1$	The smaller of: $\pi m + p$ $2e_1 + p$	The smaller of: $2m + 0{,}625e + 0{,}5p$ $e_1 + 0{,}5p$
Mode 1:	$\ell_{eff,1} = \ell_{eff,nc}$ but $\ell_{eff,1} \leq \ell_{eff,cp}$		$\sum\ell_{eff,1} = \sum\ell_{eff,nc}$ but $\sum\ell_{eff,1} \leq \sum\ell_{eff,cp}$	
Mode 2:	$\ell_{eff,2} = \ell_{eff,nc}$		$\sum\ell_{eff,2} = \sum\ell_{eff,nc}$	

6.2.6.4.2 Stiffened column flange, joint with bolted end-plate or flange cleats

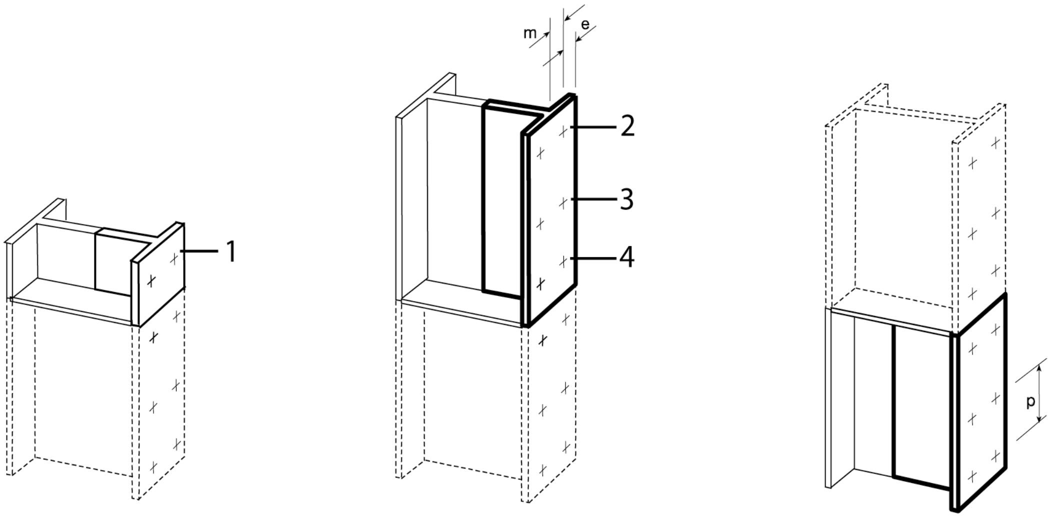
(1) Transverse stiffeners and/or appropriate arrangements of diagonal stiffeners may be used to increase the design resistance of the column flange in bending.

(2) The design resistance and failure mode of a stiffened column flange in transverse bending, together with the associated bolts in tension, should be taken as similar to those of an equivalent T-stub flange, see 6.2.4, for both:

– each individual bolt-row required to resist tension;

– each group of bolt-rows required to resist tension.

(3) The groups of bolt-rows either side of a stiffener should be modelled as separate equivalent T-stub flanges, see Figure 6.9. The design resistance and failure mode should be determined separately for each equivalent T-stub.



1 End bolt row adjacent to a stiffener
2 End bolt row
3 Inner bolt row
4 Bolt row adjacent to a stiffener

Figure 6.9: Modelling a stiffened column flange as separate T-stubs

(4) The dimensions e_{min} and m for use in 6.2.4 should be determined from Figure 6.8.

(5) The effective lengths of an equivalent T-stub flange ℓ_{eff} should be determined in accordance with 6.2.4.2 using the values for each bolt-row given in Table 6.5. The value of α for use in Table 6.5 should be obtained from Figure 6.11.

(6) The stiffeners should meet the requirements specified in 6.2.6.1.

Table 6.5: Effective lengths for a stiffened column flange

Bolt-row Location	Bolt-row considered individually		Bolt-row considered as part of a group of bolt-rows	
	Circular patterns $\ell_{eff,cp}$	Non-circular patterns $\ell_{eff,nc}$	Circular patterns $\ell_{eff,cp}$	Non-circular patterns $\ell_{eff,nc}$
Bolt-row adjacent to a stiffener	$2\pi m$	αm	$\pi m + p$	$0{,}5p + \alpha m - (2m + 0{,}625e)$
Other inner bolt-row	$2\pi m$	$4m + 1{,}25e$	$2p$	p
Other end bolt-row	The smaller of: $2\pi m$ $\pi m + 2e_1$	The smaller of: $4m + 1{,}25e$ $2m + 0{,}625e + e_1$	The smaller of: $\pi m + p$ $2e_1 + p$	The smaller of: $2m + 0{,}625e + 0{,}5p$ $e_1 + 0{,}5p$
End bolt-row adjacent to a stiffener	The smaller of: $2\pi m$ $\pi m + 2e_1$	$e_1 + \alpha m - (2m + 0{,}625e)$	not relevant	not relevant
For Mode 1:	$\ell_{eff,1} = \ell_{eff,nc}$ but $\ell_{eff,1} \le \ell_{eff,cp}$		$\sum\ell_{eff,1} = \sum\ell_{eff,nc}$ but $\sum\ell_{eff,1} \le \sum\ell_{eff,cp}$	
For Mode 2:	$\ell_{eff,2} = \ell_{eff,nc}$		$\sum\ell_{eff,2} = \sum\ell_{eff,nc}$	
α should be obtained from Figure 6.11.				

6.2.6.4.3 Unstiffened column flange, welded connection

(1) In a welded joint, the design resistance $F_{fc,Rd}$ of an unstiffened column flange in bending, due to tension or compression from a beam flange, should be obtained using:

$$F_{fc,Rd} = b_{eff,b,fc}\, t_{fb}\, f_{y,fb} / \gamma_{M0} \qquad \text{... (6.20)}$$

where:

$b_{eff,b,fc}$ is the effective breath b_{eff} defined in 4.10 where the beam flange is considered as a plate.

NOTE: The requirements specified in 4.10(4) and 4.10(6) should be satisfied.

6.2.6.5 End-plate in bending

(1) The design resistance and failure mode of an end-plate in bending, together with the associated bolts in tension, should be taken as similar to those of an equivalent T-stub flange, see 6.2.4 for both:

- each individual bolt-row required to resist tension;
- each group of bolt-rows required to resist tension.

(2) The groups of bolt-rows either side of any stiffener connected to the end-plate should be treated as separate equivalent T-stubs. In extended end-plates, the bolt-row in the extended part should also be treated as a separate equivalent T-stub, see Figure 6.10. The design resistance and failure mode should be determined separately for each equivalent T-stub.

(3) The dimension e_{min} required for use in 6.2.4 should be obtained from Figure 6.8 for that part of the end-plate located between the beam flanges. For the end-plate extension e_{min} should be taken as equal to e_x, see Figure 6.10.

(4) The effective length of an equivalent T-stub flange ℓ_{eff} should be determined in accordance with 6.2.4.2 using the values for each bolt-row given in Table 6.6.

(5) The values of m and m_x for use in Table 6.6 should be obtained from Figure 6.10.

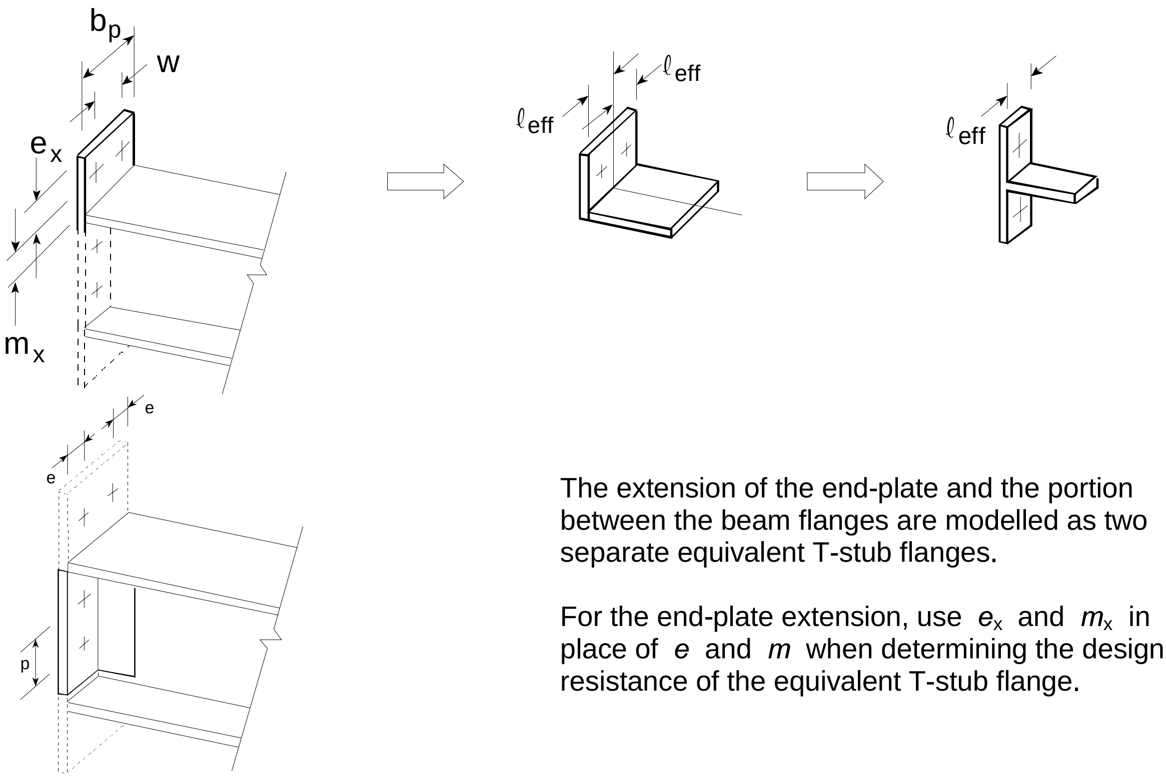
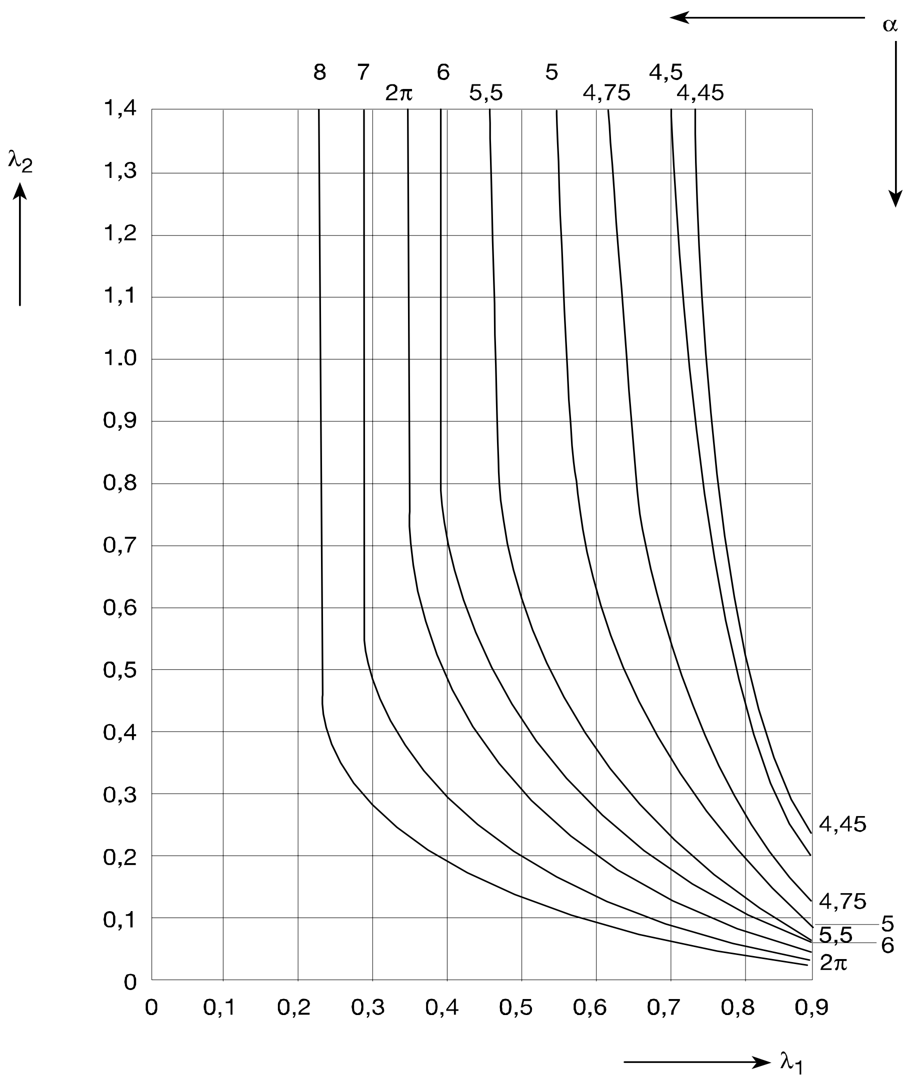


Figure 6.10: Modelling an extended end-plate as separate T-stubs

Table 6.6: Effective lengths for an end-plate

Bolt-row location	Bolt-row considered individually		Bolt-row considered as part of a group of bolt-rows	
	Circular patterns $\ell_{eff,cp}$	Non-circular patterns $\ell_{eff,nc}$	Circular patterns $\ell_{eff,cp}$	Non-circular patterns $\ell_{eff,nc}$
Bolt-row outside tension flange of beam	Smallest of: $2\pi m_x$ $\pi m_x + w$ $\pi m_x + 2e$	Smallest of: $4m_x + 1,25e_x$ $e+2m_x+0,625e_x$ $0,5b_p$ $0,5w+2m_x+0,625e_x$	—	—
First bolt-row below tension flange of beam	$2\pi m$	αm	πm + p	$0,5p + \alpha m - (2m + 0,625e)$
Other inner bolt-row	$2\pi m$	$4m + 1,25\ e$	$2p$	p
Other end bolt-row	$2\pi m$	$4m + 1,25\ e$	$\pi m + p$	$2m+0,625e+0,5p$
Mode 1:	$\ell_{eff,1} = \ell_{eff,nc}$ but $\ell_{eff,1} \le \ell_{eff,cp}$		$\sum\ell_{eff,1} = \sum\ell_{eff,nc}$ but $\sum\ell_{eff,1} \le \sum\ell_{eff,cp}$	
Mode 2:	$\ell_{eff,2} = \ell_{eff,nc}$		$\sum\ell_{eff,2} = \sum\ell_{eff,nc}$	
α should be obtained from Figure 6.11.				



$$\lambda_1 = \frac{m}{m + e}$$

$$\lambda_2 = \frac{m_2}{m + e}$$

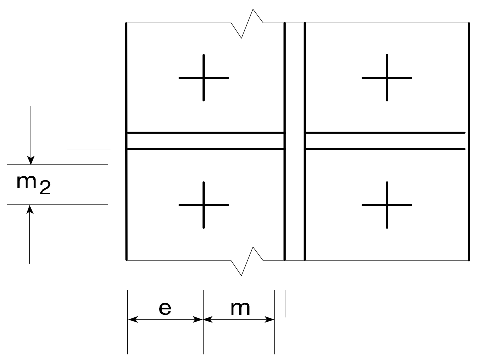


Figure 6.11: Values of α for stiffened column flanges and end-plates

6.2.6.6 Flange cleat in bending

(1) The design resistance and failure mode of a bolted angle flange cleat in bending, together with the associated bolts in tension, should be taken as similar to those of an equivalent T-stub flange, see 6.2.4.

(2) The effective length ℓ_{eff} of the equivalent T-stub flange should be taken as $0{,}5b_a$ where b_a is the length of the angle cleat, see Figure 6.12.

(3) The dimensions e_{min} and m for use in 6.2.4 should be determined from Figure 6.13.

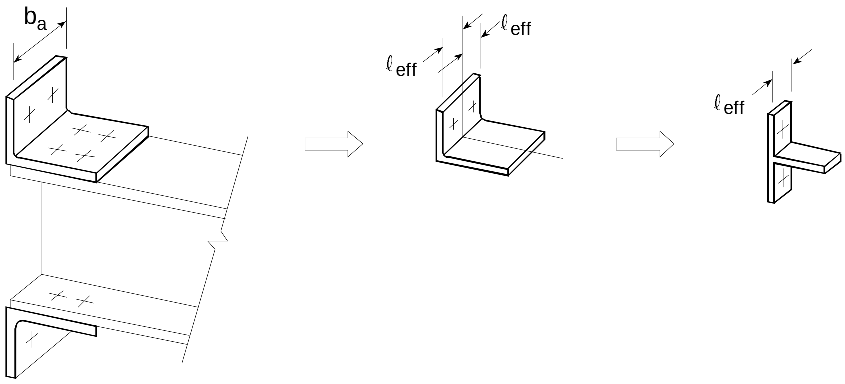
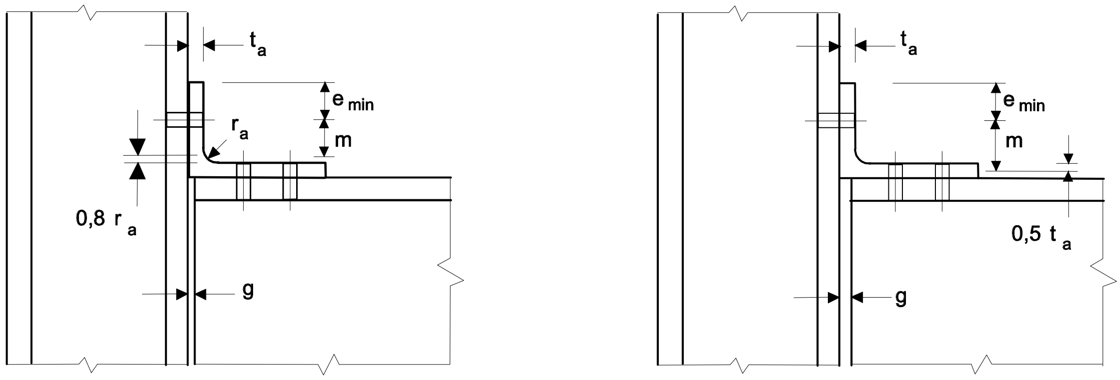


Figure 6.12: Effective length ℓ_{eff} of an angle flange cleat



a) Gap $g \leq 0{,}4\,t_a$ b) Gap $g > 0{,}4\,t_a$

Notes:

- The number of bolt-rows connecting the cleat to the column flange is limited to one;
- The number of bolt-rows connecting the cleat to the beam flange is not limited;
- The length b_a of the cleat may be different from both the width of the beam flange and the width of the column flange.

Figure 6.13: Dimensions e_{min} and m for a bolted angle cleat

6.2.6.7 Beam flange and web in compression

(1) The design compression resistance of a beam flange and the adjacent compression zone of the beam web, may be assumed to act at the level of the centre of compression, see 6.2.7. The design compression resistance of the combined beam flange and web is given by the following expression:

$$F_{c,fb,Rd} = M_{c,Rd}/(h - t_{fb}) \quad \text{... (6.21)}$$

where:

h is the depth of the connected beam;

$M_{c,Rd}$ is the design moment resistance of the beam cross-section, reduced if necessary to allow for shear, see EN 1993-1-1. For a haunched beam $M_{c,Rd}$ may be calculated neglecting the intermediate flange.

t_{fb} is the flange thickness of the connected beam.

If the height of the beam including the haunch exceeds 600 mm the contribution of the beam web to the design compression resistance should be limited to 20%.

(2) If a beam is reinforced with haunches they should be arranged such that:

- the steel grade of the haunch should match that of the member;
- the flange size and the web thickness of the haunch should not be less than that of the member;
- the angle of the haunch flange to the flange of the member should not be greater than 45°;
- the length of stiff bearing s_s should be taken as equal to the thickness of the haunch flange parallel to the beam.

(3) If a beam is reinforced with haunches, the design resistance of beam web in compression should be determined according to 6.2.6.2.

6.2.6.8 Beam web in tension

(1) In a bolted end-plate connection, the design tension resistance of the beam web should be obtained from:

$$F_{\mathrm{t,wb,Rd}} = b_{eff,t,wb}\, t_{wb}\, f_{y,wb} / \gamma_{M0} \qquad \text{... (6.22)}$$

(2) The effective width $b_{\mathrm{eff,t,wb}}$ of the beam web in tension should be taken as equal to the effective length of the equivalent T-stub representing the end-plate in bending, obtained from 6.2.6.5 for an individual bolt-row or a bolt-group.

6.2.6.9 Concrete in compression including grout

(1) The design bearing strength of the joint between the base plate and its concrete support should be determined taking account of the material properties and dimensions of both the grout and the concrete support. The concrete support should be designed according to EN 1992.

(2) The design resistance of concrete in compression, including grout, together with the associated base plate in bending $F_{\mathrm{c,pl,Rd}}$, should be taken as similar to those of an equivalent T-stub, see 6.2.5.

6.2.6.10 Base plate in bending under compression

(1) The design resistance of a base plate in bending under compression, together with concrete slab on which the column base is placed $F_{\mathrm{c,pl,Rd}}$, should be taken as similar to those of an equivalent T-stub, see 6.2.5.

6.2.6.11 Base plate in bending under tension

(1) The design resistance and failure mode of a base plate in bending under tension, together with the associated anchor bolts in tension $F_{\mathrm{t,pl,Rd}}$, may be determined using the rules given in 6.2.6.5.

(2) In the case of base plates prying forces which may develop should not be taken into consideration.

6.2.6.12 Anchor bolt in tension

(1) Anchor bolts should be designed to resist the effects of the design loads. They should provide design resistance to tension due to uplift forces and bending moments where appropriate.

(2) When calculating the tension forces in the anchor bolts due to bending moments, the lever arm should not be taken as more than the distance between the centroid of the bearing area on the compression side and the centroid of the bolt group on the tension side.

NOTE: Tolerances on the positions of the anchor bolts should be taken into account if the influence of tolerances is significant.

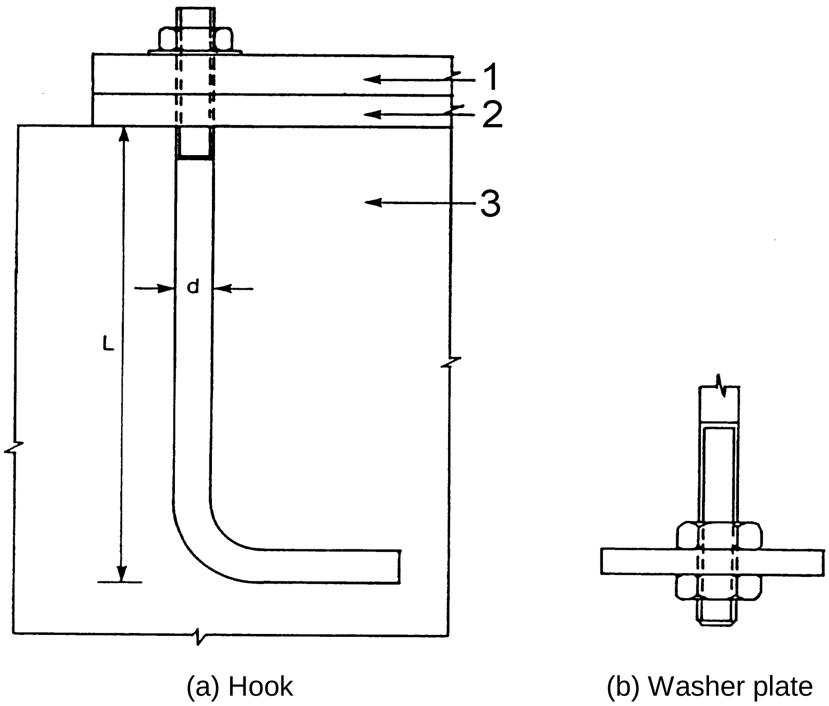
(3) The design resistance of the anchor bolts should be taken as the smaller of the design tension resistance of the anchor bolt, see 3.6, and the design bond resistance of the concrete on the anchor bolt according to EN 1992-1-1.

(4) One of the following methods should be used to secure anchor bolts into the foundation:

- a hook (Figure 6.14(a)),
- a washer plate (Figure 6.14(b)),
- some other appropriate load distributing member embedded in the concrete,
- some other fixing which has been adequately tested and approved.

(5) When the bolts are provided with a hook, the anchorage length should be such as to prevent bond failure before yielding of the bolt. The anchorage length should be calculated in accordance with EN 1992-1-1. This type of anchorage should not be used for bolts with a yield strength f_{yb} higher than 300 N/mm^2.

(6) When the anchor bolts are provided with a washer plate or other load distributing member, no account should be taken of the contribution of bond. The whole of the force should be transferred through the load distributing device.



1 Base plate
2 Grout
3 Concrete foundation

Figure 6.14: Fixing of anchor bolts

6.2.7 Design Moment resistance of beam-to-column joints and splices

6.2.7.1 General

(1) The applied design moment $M_{j,Ed}$ shall satisfy:

$$\frac{M_{j,Ed}}{M_{j,Rd}} \leq 1,0 \qquad \text{... (6.23)}$$

(2) The methods given in 6.2.7 for determining the design moment resistance of a joint $M_{j,Rd}$ do not take account of any co-existing axial force N_{Ed} in the connected member. They should not be used if the axial force in the connected member exceeds 5% of the design plastic resistance $N_{p\ell,Rd}$ of its cross-section.

(3) If the axial force N_{Ed} in the connected beam exceeds 5% of the design resistance, $N_{pl,Rd}$, the following conservative method may be used:

$$\frac{M_{j,Ed}}{M_{j,Rd}} + \frac{N_{j,Ed}}{N_{j,Rd}} \leq 1,0 \qquad \text{... (6.24)}$$

where:

$M_{j.Rd}$ is the design moment resistance of the joint, assuming no axial force;

$N_{j.Rd}$ is the axial design resistance of the joint, assuming no applied moment.

(4) The design moment resistance of a welded joint should be determined as indicated in Figure 6.15(a).

(5) The design moment resistance of a bolted joint with a flush end-plate that has only one bolt-row in tension (or in which only one bolt-row in tension is considered, see 6.2.3(6)) should be determined as indicated in Figure 6.15(b).

(6) The design moment resistance of a bolted joint with angle flange cleats should be determined as indicated in Figure 6.15(c).

(7) The design moment resistance of a bolted end-plate joint with more than one row of bolts in tension should generally be determined as specified in 6.2.7.2.

(8) As a conservative simplification, the design moment resistance of an extended end-plate joint with only two rows of bolts in tension may be approximated as indicated in Figure 6.16, provided that the total design resistance F_{Rd} does not exceed $3,8F_{t,Rd}$, where $F_{t,Rd}$ is given in Table 6.2. In this case the whole tension region of the end-plate may be treated as a single basic component. Provided that the two bolt-rows are approximately equidistant either side of the beam flange, this part of the end-plate may be treated as a T-stub to determine the bolt-row force $F_{1,Rd}$. The value of $F_{2,Rd}$ may then be assumed to be equal to $F_{1,Rd}$, and so F_{Rd} may be taken as equal to $2F_{1,Rd}$.

(9) The centre of compression should be taken as the centre of the stress block of the compression forces. As a simplification the centre of compression may be taken as given in Figure 6.15.

(10) A splice in a member or part subject to tension shall be designed to transmit all the moments and forces to which the member or part is subjected at that point.

(11) Splices shall be designed to hold the connected members in place. Friction forces between contact surfaces may not be relied upon to hold connected members in place in a bearing splice.

(12) Wherever practicable the members should be arranged so that the centroidal axis of any splice material coincides with the centroidal axis of the member. If eccentricity is present then the resulting forces should be taken into account.

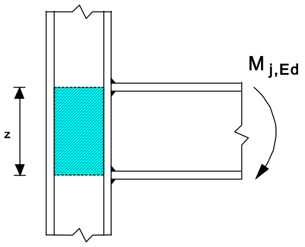
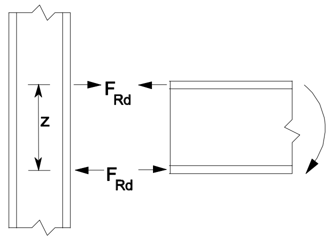
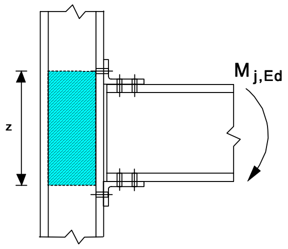
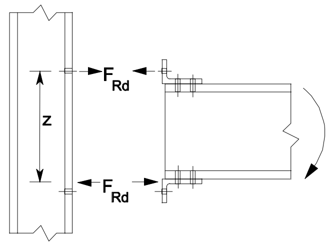
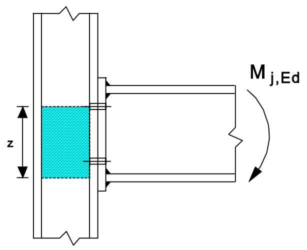
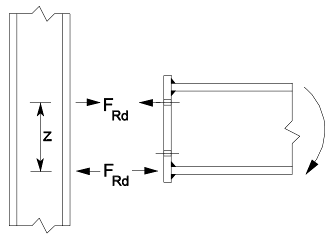
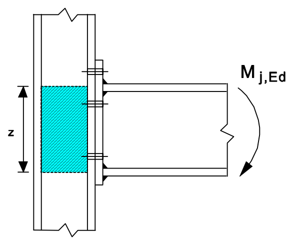
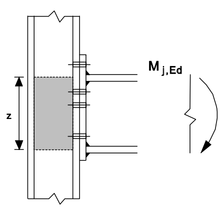
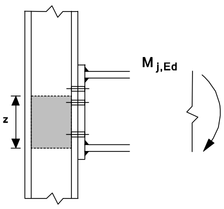
Type of connection	Centre of compression	Lever arm	Force distributions
a) Welded connection 	In line with the mid thickness of the compression flange	$z = h - t_{fb}$ h is the depth of the connected beam t_{fb} is the thickness of the beam flange	
b) Bolted connection with angle flange cleats 	In line with the mid-thickness of the leg of the angle cleat on the compression flange	Distance from the centre of compression to the bolt-row in tension	
c) Bolted end-plate connection with only one bolt-row active in tension 	In line with the mid-thickness of the compression flange	Distance from the centre of compression to the bolt-row in tension	
d) Bolted extended end-plate connection with only two bolt-rows active in tension 	In line with the mid-thickness of the compression flange	Conservatively z may be taken as the distance from the centre of compression to a point midway between these two bolt-rows	
e) Other bolted end-plate connections with two or more bolt-rows in tension  	In line with the mid-thickness of the compression flange	An approximate value may be obtained by taking the distance from the centre of compression to a point midway between the farthest two bolt-rows in tension	A more accurate value may be determined by taking the lever arm z as equal to z_{eq} obtained using the method given in 6.3.3.1.

Figure 6.15: Centre of compression, lever arm z and force distributions for deriving the design moment resistance $M_{j,Rd}$

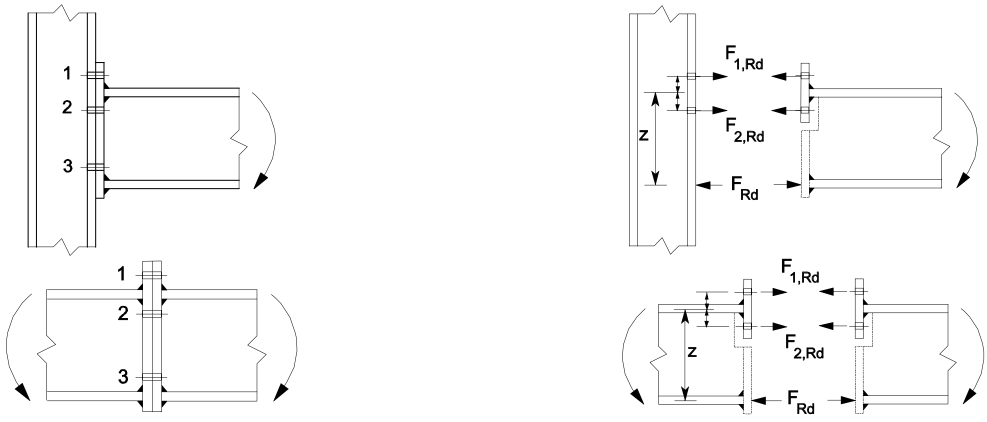


Figure 6.16: Simplified models for bolted joints with extended end-plates

(13) Where the members are not prepared for full contact in bearing, splice material should be provided to transmit the internal forces and moments in the member at the spliced section, including the moments due to applied eccentricity, initial imperfections and second-order deformations. The internal forces and moments should be taken as not less than a moment equal to 25% of the moment capacity of the weaker section about both axes and a shear force equal to 2.5% of the normal force capacity of the weaker section in the directions of both axes.

(14) Where the members are prepared for full contact in bearing, splice material should be provided to transmit 25% of the maximum compressive force in the column.

(15) The alignment of the abutting ends of members subjected to compression should be maintained by cover plates or other means. The splice material and its fastenings should be proportioned to carry forces at the abutting ends, acting in any direction perpendicular to the axis of the member. In the design of splices the second order effects should also be taken into account.

(16) Splices in flexural members should comply with the following:

a) Compression flanges should be treated as compression members;

b) Tension flanges should be treated as tension members;

c) Parts subjected to shear should be designed to transmit the following effects acting together:

- the shear force at the splice;
- the moment resulting from the eccentricity, if any, of the centroids of the groups of fasteners on each side of the splice;
- the proportion of moment, deformation or rotations carried by the web or part, irrespective of any shedding of stresses into adjoining parts assumed in the design of the member or part.

6.2.7.2 Beam-to-column joints with bolted end-plate connections

(1) The design moment resistance $M_{j,Rd}$ of a beam-to-column joint with a bolted end-plate connection may be determined from:

$$M_{j,Rd} = \sum_r h_r F_{tr,Rd} \qquad \text{... (6.25)}$$

where:

$F_{tr,Rd}$ is the effective design tension resistance of bolt-row r ;

h_r is the distance from bolt-row r to the centre of compression;

r is the bolt-row number.

NOTE: In a bolted connection with more than one bolt-row in tension, the bolt-rows are numbered starting from the bolt-row farthest from the centre of compression.

(2) For bolted end-plate connections, the centre of compression should be assumed to be in line with the centre of the compression flange of the connected member.

(3) The effective design tension resistance $F_{tr,Rd}$ for each bolt-row should be determined in sequence, starting from bolt-row 1, the bolt-row farthest from the centre of compression, then progressing to bolt-row 2, etc.

(4) When determining the value of $F_{tr,Rd}$ for bolt-row r the effective design tension resistance of all other bolt-rows closer to the centre of compression should be ignored.

(5) The effective design tension resistance $F_{tr,Rd}$ of bolt-row r should be taken as its design tension resistance $F_{t,Rd}$ as an individual bolt-row determined from 6.2.7.2(6), reduced if necessary to satisfy the conditions specified in 6.2.7.2(7), (8) and (9).

(6) The effective design tension resistance $F_{tr,Rd}$ of bolt-row r ,taken as an individual bolt-row, should be taken as the smallest value of the design tension resistance for an individual bolt-row of the following basic components:

– the column web in tension $F_{t,wc,Rd}$ - see 6.2.6.3;
– the column flange in bending $F_{t,fc,Rd}$ - see 6.2.6.4;
– the end-plate in bending $F_{t,ep,Rd}$ - see 6.2.6.5;
– the beam web in tension $F_{t,wb,Rd}$ - see 6.2.6.8.

(7) The effective design tension resistance $F_{tr,Rd}$ of bolt-row r should, if necessary, be reduced below the value of $F_{t,Rd}$ given by 6.2.7.2(6) to ensure that, when account is taken of all bolt-rows up to and including bolt-row r the following conditions are satisfied:

– the total design resistance $\sum F_{t,Rd} \leq V_{wp,Rd}/\beta$ - with β from 5.3(7) - see 6.2.6.1;
– the total design resistance $\sum F_{t,Rd}$ does not exceed the smaller of:
  – the design resistance of the column web in compression $F_{c,wc,Rd}$ - see 6.2.6.2;
  – the design resistance of the beam flange and web in compression $F_{c,fb,Rd}$ - see 6.2.6.7.

(8) The effective design tension resistance $F_{tr,Rd}$ of bolt-row r should, if necessary, be reduced below the value of $F_{t,Rd}$ given by 6.2.7.2(6), to ensure that the sum of the design resistances taken for the bolt-rows up to and including bolt-row r that form part of the same group of bolt-rows, does not exceed the design resistance of that group as a whole. This should be checked for the following basic components:

– the column web in tension $F_{t,wc,Rd}$ - see 6.2.6.3;
– the column flange in bending $F_{t,fc,Rd}$ - see 6.2.6.4;
– the end-plate in bending $F_{t,ep,Rd}$ - see 6.2.6.5;
– the beam web in tension $F_{t,wb,Rd}$ - see 6.2.6.8.

(9) Where the effective design tension resistance $F_{tx,Rd}$ of one of the previous bolt-rows x is greater than $1,9 F_{t,Rd}$, then the effective design tension resistance $F_{tr,Rd}$ for bolt-row r should be reduced, if necessary, in order to ensure that:

$$F_{tr,Rd} \leq F_{tx,Rd} h_r / h_x \qquad \text{... (6.26)}$$

where:

h_x is the distance from bolt-row x to the centre of compression;

x is the bolt-row farthest from the centre of compression that has a design tension resistance greater than $1{,}9\,F_{t,Rd}$.

NOTE: The National Annex may give other situations where equation (6.26) is relevant.

(10) The method described in 6.2.7.2(1) to 6.2.7.2(9) may be applied to a bolted beam splice with welded end-plates, see Figure 6.17, by omitting the items relating to the column.

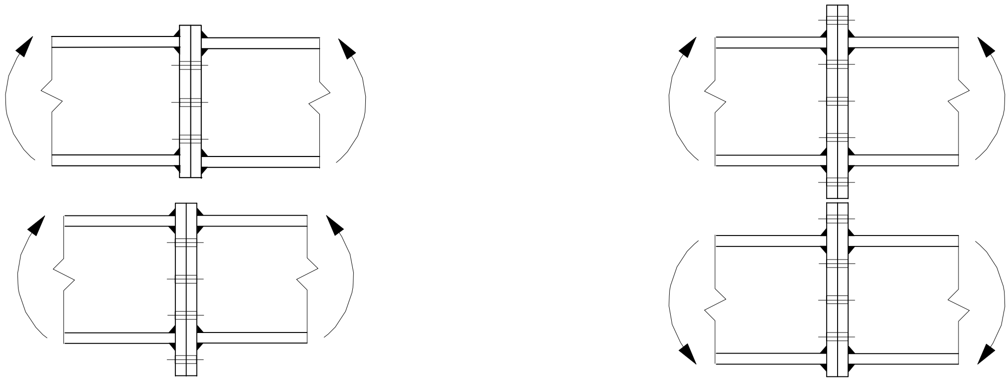


Figure 6.17: Bolted beam splices with welded end-plates

6.2.8 Design Resistance of column bases with base plates

6.2.8.1 General

(1) Column bases should be of sufficient size, stiffness and strength to transmit the axial forces, bending moments and shear forces in columns to their foundations or other supports without exceeding the load carrying capacity of these supports.

(2) The design bearing strength between the base plate and its support may be determined on the basis of a uniform distribution of compressive force over the bearing area. For concrete foundations the bearing strength should not exceed the design bearing strength, f_{jd} , given in 6.2.5(7).

(3) For a column base subject to combined axial force and bending the forces between the base plate and its support can take one of the following distribution depending on the relative magnitude of the applied axial force and bending moment:

- In the case of a dominant compressive axial force, full compression may develop under both column flanges as shown in Figure 6.18(a).
- In the case of a dominant tensile force, full tension may develop under both flanges as shown in Figure 6.18(b).
- In the case of a dominant bending moment compression may develop under one column flange and tension under the other as shown in Figure 6.18(c) and Figure 6.18(d).

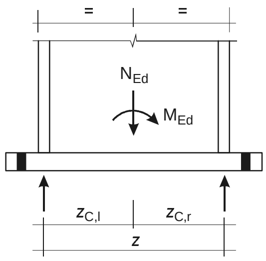
(4) Base plates should be designed using the appropriate methods given in 6.2.8.2 and 6.2.8.3.

(5) One of the following methods should be used to resist the shear force between the base plate and its support:

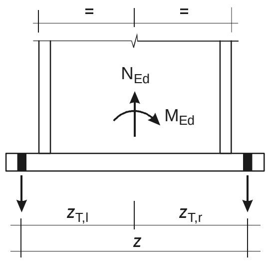
- Frictional design resistance at the joint between the base plate and its support.
- The design shear resistance of the anchor bolts.
- The design shear resistance of the surrounding part of the foundation.

If anchor bolts are used to resist the shear forces between the base plate and its support, rupture of the concrete in bearing should also be checked, according to EN 1992.

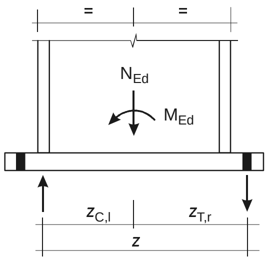
Where the above methods are inadequate special elements such as blocks or bar shear connectors should be used to transfer the shear forces between the base plate and its support.



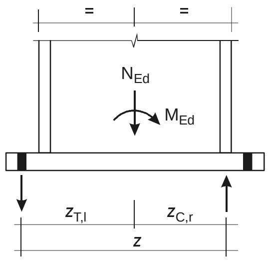
a) Column base connection in case of a dominant compressive normal force



b) Column base connection in case of a dominant tensile normal force



c) Column base connection in case of a dominant bending moment

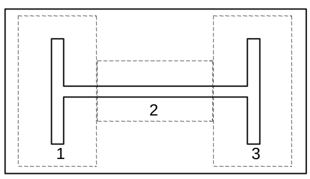


d) Column base connection in case of a dominant bending moment

Figure 6.18: Determination of the lever arm *z* for column base connections

6.2.8.2 Column bases only subjected to axial forces

(1) The design resistance, $N_{j,Rd}$,of a symmetric column base plate subject to an axial compressive force applied concentrically may be determined by adding together the individual design resistance $F_{C,Rd}$ of the three T-stubs shown in Figure 6.19 (Two T-stubs under the column flanges and one T-stub under the column web.) The three T-stubs should not be overlapping, see Figure 6.19. The design resistance of each of these T-stubs should be calculated using the method given in 6.2.5.



1 T-stub 1
2 T-stub 2
3 T-stub 3

Figure 6.19: Non overlapping T-stubs

6.2.8.3 Column bases subjected to axial forces and bending moments

(1) The design moment resistance $M_{j,Rd}$ of a column base subject to combined axial force and moment should be determined using the method given in Table 6.7 where the contribution of the concrete portion just under the column web (T-stub 2 of Figure 6.19) to the compressive capacity is omitted. The following parameters are used in this method:

- $F_{T,l,Rd}$ is the design tension resistance of the left hand side of the joint - see 6.2.8.3(2)
- $F_{T,r,Rd}$ is the design tension resistance of the right hand side of the joint - see 6.2.8.3(3)
- $F_{C,l,Rd}$ is the design compressive resistance of the left hand side of the joint - see 6.2.8.3(4)
- $F_{C,r,Rd}$ is the design compressive resistance of the right hand side of the joint - see 6.2.8.3(5)

(2) The design tension resistance $F_{T,l,Rd}$ of the left side of the joint should be taken as the smallest values of the design resistance of following basic components:

– the column web in tension under the left column flange $F_{t,wc,Rd}$ - see 6.2.6.3;

– the base plate in bending under the left column flange $F_{t,pl,Rd}$ - see 6.2.6.11.

(3) The design tension resistance $F_{T,r,Rd}$ of the right side of the joint should be taken as the smallest values of the design resistance of following basic components:

– the column web in tension under the right column flange $F_{t,wc,Rd}$ - see 6.2.6.3;

– the base plate in bending under the right column flange $F_{t,pl,Rd}$ - see 6.2.6.11.

(4) The design compressive resistance $F_{C,l,Rd}$ of the left side of the joint should be taken as the smallest values of the design resistance of following basic components:

– the concrete in compression under the left column flange $F_{c,pl,Rd}$ - see 6.2.6.9;

– the left column flange and web in compression $F_{c,fc,Rd}$ - see 6.2.6.7.

(5) The design compressive resistance $F_{C,r,Rd}$ of the right side of the joint should be taken as the smallest values of the design resistance of following basic components:

– the concrete in compression under the right column flange $F_{c,pl,Rd}$ - see 6.2.6.9;

– the right column flange and web in compression $F_{c,fc,Rd}$ - see 6.2.6.7.

(6) For the calculation of $z_{T,l}$, $z_{C,l}$, $z_{T,r}$, $z_{C,r}$ see 6.2.8.1.

Table 6.7: Design moment resistance $M_{j,Rd}$ of column bases

Loading	Lever arm z	Design moment resistance $M_{j,Rd}$	
Left side in tension Right side in compression	$z = z_{T,l} + z_{C,r}$	$N_{Ed} > 0$ and $e > z_{T,l}$	$N_{Ed} \leq 0$ and $e \leq -z_{C,r}$
		The smaller of $\frac{F_{T,1,Rd}\, z}{z_{C,r}/e+1}$ and $\frac{-F_{C,r,Rd}\, z}{z_{T,1}/e-1}$	
Left side in tension Right side in tension	$z = z_{T,l} + z_{T,r}$	$N_{Ed} > 0$ and $0 < e < z_{T,l}$	$N_{Ed} > 0$ and $-z_{T,r} < e \leq 0$
		The smaller of $\frac{F_{T,1,Rd}\, z}{z_{T,r}/e+1}$ and $\frac{F_{T,r,Rd}\, z}{z_{T,1}/e-1}$	The smaller of $\frac{F_{T,1,Rd}\, z}{z_{T,r}/e+1}$ and $\frac{F_{T,1,Rd}\, z}{z_{T,1}/e-1}$
Left side in compression Right side in tension	$z = z_{C,l} + z_{T,r}$	$N_{Ed} > 0$ and $e \leq -z_{T,r}$	$N_{Ed} \leq 0$ and $e > z_{C,l}$
		The smaller of $\frac{-F_{C,1,Rd}\, z}{z_{T,r}/e+1}$ and $\frac{F_{T,r,Rd}\, z}{z_{C,1}/e-1}$	
Left side in compression Right side in compression	$z = z_{C,l} + z_{C,r}$	$N_{Ed} \leq 0$ and $0 < e < z_{C,l}$	$N_{Ed} \leq 0$ and $-z_{C,r} < e \leq 0$
		The smaller of $\frac{-F_{C,1,Rd}\, z}{z_{C,r}/e+1}$ and $\frac{-F_{C,r,Rd}\, z}{z_{C,1}/e-1}$	The smaller of $\frac{-F_{C,1,Rd}\, z}{z_{C,r}/e+1}$ and $\frac{-F_{C,r,Rd}\, z}{z_{C,1}/e-1}$
$M_{Ed} > 0$ is clockwise, $N_{Ed} > 0$ is tension $e = \frac{M_{Ed}}{N_{Ed}} = \frac{M_{Rd}}{N_{Rd}}$			

6.3 Rotational stiffness

6.3.1 Basic model

(1) The rotational stiffness of a joint should be determined from the flexibilities of its basic components, each represented by an elastic stiffness coefficient k_i obtained from 6.3.2.

NOTE: These elastic stiffness coefficients are for general application.

(2) For bolted end-plate connections with more than one row of bolts in tension, the stiffness coefficients k_i for the related basic components should be combined. For beam-to-column joints and beam splices a method is given in 6.3.3 and for column bases a method is given in 6.3.4.

(3) In a bolted connection with more than one bolt-row in tension, as a simplification the contribution of any bolt-row may be neglected, provided that the contributions of all other bolt-rows closer to the centre of compression are also neglected. The number of bolt-rows retained need not necessarily be the same as for the determination of the design moment resistance.

(4) Provided that the axial force N_{Ed} in the connected member does not exceed 5% of the design resistance $N_{\mathrm{p}\ell,\mathrm{Rd}}$ of its cross-section, the rotational stiffness S_{j} of a beam-to-column joint or beam splice, for a moment $M_{\mathrm{j,Ed}}$ less than the design moment resistance $M_{\mathrm{j,Rd}}$ of the joint, may be obtained with sufficient accuracy from:

$$S_{\mathrm{j}} = \frac{Ez^2}{\mu \sum_i \frac{1}{k_i}} \qquad \text{... (6.27)}$$

where:

k_i is the stiffness coefficient for basic joint component i;

z is the lever arm, see 6.2.7;

μ is the stiffness ratio $S_{\mathrm{j,ini}}/S_{\mathrm{j}}$, see 6.3.1(6);

NOTE: The initial rotational stiffness $S_{\mathrm{j,ini}}$ of the joint is given by expression (6.27) with $\mu = 1{,}0$.

(5) The rotational stiffness S_{j} of a column base, for a moment $M_{\mathrm{j,Ed}}$ less than the design moment resistance $M_{\mathrm{j,Rd}}$ of the joint, may be obtained with sufficient accuracy from 6.3.4.

(6) The stiffness ratio μ should be determined from the following:

– if $M_{\mathrm{j,Ed}} \leq 2/3\ M_{\mathrm{j,Rd}}$:

$$\mu = 1 \qquad \text{... (6.28a)}$$

– if $2/3\ M_{\mathrm{j,Rd}} < M_{\mathrm{j,Ed}} \leq M_{\mathrm{j,Rd}}$:

$$\mu = (1{,}5 M_{j,Ed} / M_{j,Rd})^{\Psi} \qquad \text{... (6.28b)}$$

in which the coefficient ψ is obtained from Table 6.8.

Table 6.8: Value of the coefficient ψ

Type of connection	ψ
Welded	2,7
Bolted end-plate	2,7
Bolted angle flange cleats	3,1
Base plate connections	2,7

(7) The basic components that should be taken into account when calculating the stiffness of a welded beam-to-column connection and a bolted angle flange cleat are given in Table 6.9. Similarly, the basic components for a bolted end-plate connection and a base plate are given in Table 6.10. In both of these tables the stiffness coefficients, k_i ,for the basic components are defined in Table 6.11.

(8) For beam-to-column end plate joints the following procedure should be used for obtaining the joint stiffness. The equivalent stiffness coefficient, k_{eq}, and the equivalent lever arm, z_{eq}, of the connection should be obtained from 6.3.3. The stiffness of the joint should then be obtained from 6.3.1(4) based on the stiffness coefficients, k_{eq} (for the connection), k_1 (for the column web in shear), and with the lever arm, z, taken equal to the equivalent lever arm of the connection, z_{eq}.

Table 6.9: Joints with welded connections or bolted angle flange cleat connections

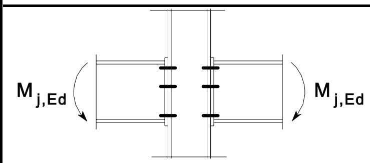
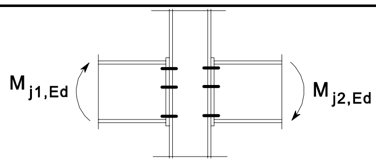
Beam-to-column joint with welded connections	Stiffness coefficients k_i to be taken into account
Single-sided	k_1; k_2; k_3
Double-sided – Moments equal and opposite	k_2; k_3
Double-sided – Moments unequal	k_1; k_2; k_3
Beam-to-column joint with Bolted angle flange cleat connections	Stiffness coefficients k_i to be taken into account
Single-sided	k_1; k_2; k_3; k_4; k_6; k_{10}; k_{11} *); k_{12} **)
Double-sided – Moments equal and opposite	k_2; k_3; k_4; k_6; k_{10}; k_{11} *); k_{12} **)
Double-sided – Moments unequal	k_1; k_2; k_3; k_4; k_6; k_{10}; k_{11} *); k_{12} **)
 Moments equal and opposite  Moments unequal	*) Two k_{11} coefficients, one for each flange; **) Four k_{12} coefficients, one for each flange and one for each cleat.

Table 6.10: Joints with bolted end-plate connections and base plate connections

Beam-to-column joint with bolted end-plate connections	Number of bolt-rows in tension	Stiffness coefficients k_i to be taken into account
Single-sided	One	k_1; k_2; k_3; k_4; k_5; k_{10}
	Two or more	k_1; k_2; k_{eq}
Double sided – Moments equal and opposite	One	k_2; k_3; k_4; k_5; k_{10}
	Two or more	k_2; k_{eq}
Double sided – Moments unequal	One	k_1; k_2; k_3; k_4; k_5; k_{10}
	Two or more	k_1; k_2; k_{eq}
Beam splice with bolted end-plates	Number of bolt-rows in tension	Stiffness coefficients k_i to be taken into account
Double sided - Moments equal and opposite	One	k_5[left]; k_5[right]; k_{10}
	Two or more	k_{eq}
Base plate connections	Number of bolt-rows in tension	Stiffness coefficients k_i to be taken into account
Base plate connections	One	k_{13}; k_{15}; k_{16}
	Two or more	k_{13}; k_{15} and k_{16} for each bolt row

6.3.2 Stiffness coefficients for basic joint components

(1) The stiffness coefficients for basic joint component should be determined using the expressions given in Table 6.11.

Table 6.11: Stiffness coefficients for basic joint components

<table>
<tr><td>Component</td><td colspan="2">Stiffness coefficient k_i</td></tr>
<tr><td rowspan="3">Column web panel in shear</td><td>Unstiffened,
single-sided joint, or a double-sided joint in which the beam depths are similar</td><td>stiffened</td></tr>
<tr><td>$k_1 = \dfrac{0{,}38 A_{VC}}{\beta z}$</td><td>$k_1 = \infty$</td></tr>
<tr><td colspan="2">z is the lever arm from Figure 6.15;
β is the transformation parameter from 5.3(7).</td></tr>
<tr><td rowspan="3">Column web in compression</td><td>unstiffened</td><td>stiffened</td></tr>
<tr><td>$k_2 = \dfrac{0{,}7\, b_{eff,c,wc}\, t_{wc}}{d_c}$</td><td>$k_2 = \infty$</td></tr>
<tr><td colspan="2">$b_{\mathrm{eff,c,wc}}$ is the effective width from 6.2.6.2</td></tr>
<tr><td rowspan="3">Column web in tension</td><td>stiffened or unstiffened bolted connection with a single bolt-row in tension or unstiffened welded connection</td><td>stiffened welded connection</td></tr>
<tr><td>$k_3 = \dfrac{0{,}7 b_{eff,t,wc}\, t_{wc}}{d_c}$</td><td>$k_3 = \infty$</td></tr>
<tr><td colspan="2">$b_{\mathrm{eff,t,wc}}$ is the effective width of the column web in tension from 6.2.6.3. For a joint with a single bolt-row in tension, $b_{\mathrm{eff,t,wc}}$ should be taken as equal to the smallest of the effective lengths ℓ_{eff} (individually or as part of a group of bolt-rows) given for this bolt-row in Table 6.4 (for an unstiffened column flange) or Table 6.5 (for a stiffened column flange).</td></tr>
<tr><td>Column flange in bending
(for a single bolt-row in tension)</td><td colspan="2">$k_4 = \dfrac{0{,}9 \ell_{eff}\, t_{fc}^{\;3}}{m^3}$
ℓ_{eff} is the smallest of the effective lengths (individually or as part of a bolt group) for this bolt-row given in Table 6.4 for an unstiffened column flange or Table 6.5 for a stiffened column flange;
m is as defined in Figure 6.8;</td></tr>
<tr><td>End-plate in bending
(for a single bolt-row in tension)</td><td colspan="2">$k_5 = \dfrac{0{,}9 \ell_{eff}\, t_p^{\;3}}{m^3}$
ℓ_{eff} is the smallest of the effective lengths (individually or as part of a group of bolt-rows) given for this bolt-row in Table 6.6;
m is generally as defined in Figure 6.11, but for a bolt-row located in the extended part of an extended end-plate $m = m_{\mathrm{x}}$, where m_{x} is as defined in Figure 6.10.</td></tr>
<tr><td>Flange cleat in bending</td><td colspan="2">$k_6 = \dfrac{0{,}9 \ell_{eff}\, t_a^{\;3}}{m^3}$
ℓ_{eff} is the effective length of the flange cleat from Figure 6.12;
m is as defined in Figure 6.13.</td></tr>
</table>

Component	Stiffness coefficient k_i
Bolts in tension (for a single bolt-row)	$k_{10} = 1{,}6A_s / L_b$ preloaded or non-preloaded L_b is the bolt elongation length, taken as equal to the grip length (total thickness of material and washers), plus half the sum of the height of the bolt head and the height of the nut.
Bolts in shear	non-preloaded: k_{11} (or k_{17}) $= \dfrac{16 n_b d^2 f_{ub}}{E d_{M16}}$ preloaded [*)]: $k_{11} = \infty$ d_{M16} is the nominal diameter of an M16 bolt; n_b is the number of bolt-rows in shear.
Bolts in bearing (for each component j on which the bolts bear)	non-preloaded: k_{12} (or k_{18}) $= \dfrac{24 n_b k_b k_t d f_u}{E}$ preloaded [*)]: $k_{12} = \infty$ $k_b = k_{b1}$ but $k_b \leq k_{b2}$ $k_{b1} = 0{,}25\, e_b / d + 0{,}5$ but $k_{b1} \leq 1{,}25$ $k_{b2} = 0{,}25\, p_b / d + 0{,}375$ but $k_{b2} \leq 1{,}25$ $k_t = 1{,}5\, t_j / d_{M16}$ but $k_t \leq 2{,}5$ e_b is the distance from the bolt-row to the free edge of the plate in the direction of load transfer; f_u is the ultimate tensile strength of the steel on which the bolt bears; p_b is the spacing of the bolt-rows in the direction of load transfer; t_j is the thickness of that component.
Concrete in compression (including grout)	$k_{13} = \dfrac{E_c \sqrt{b_{eff} l_{eff}}}{1{,}275\, E}$ b_{eff} is the effective width of the T-stub flange, see 6.2.5(3); l_{eff} is the effective length of the T-stub flange, see 6.2.5(3).
Plate in bending under compression	$k_{14} = \infty$ This coefficient is already taken into consideration in the calculation of the stiffness coefficient k_{13}
Base plate in bending under tension (for a single bolt row in tension)	with prying forces [**)]: $k_{15} = \dfrac{0{,}85 \ell_{eff} t_p^3}{m^3}$ without prying forces [**)]: $k_{15} = \dfrac{0{,}425 \ell_{eff} t_p^3}{m^3}$ l_{eff} is the effective length of the T-stub flange, see 6.2.5(3); t_p is the thickness of the base plate; m is the distance according to Figure 6.8.
Anchor bolts in tension	with prying forces [**)]: $k_{16} = 1{,}6A_s / L_b$ without prying forces [**)]: $k_{16} = 2{,}0A_s / L_b$ L_b is the anchor bolt elongation length, taken as equal to the sum of 8 times the nominal bolt diameter, the grout layer, the plate thickness, the washer and half of the height of the nut.

*) provided that the bolts have been designed not to slip into bearing at the load level concerned

**) prying forces may develop, if $L_b \leq \dfrac{8{,}8 m^3 A_s}{l_{eff} t^3}$

NOTE 1: When calculating b_{eff} and l_{eff} the distance c should be taken as 1,25 times the base plate thickness.

NOTE 2: Backing plates should be assumed not to affect the rotational stiffness S_j of the joint.

NOTE 3: For *welds* (k_{19}) the stiffness coefficient should be taken as equal to infinity. This component need not be taken into account when calculating the rotational stiffness S_j.

NOTE 4: For *beam flange and web in compression* (k_7), *beam web in tension* (k_8), *plate in tension or compression* (k_9), *haunched beams* (k_{20}), the stiffness coefficients should be taken as equal to infinity. These components need not be taken into account when calculating the rotational stiffness S_j.

NOTE 5: Where a *supplementary web plate* is used, the stiffness coefficients for the relevant basic joint components k_1 to k_3 should be increased as follows:

- k_1 for the column web panel in shear should be based on the increased shear area A_{vc} from 6.2.6.1(6);
- k_2 for the column web in compression should be based on the effective thickness of the web from 6.2.6.2(6);
- k_3 for the column web in tension, should be based on the effective thickness of the web from 6.2.6.3(8).

6.3.3 End-plate connections with two or more bolt-rows in tension

6.3.3.1 General method

(1) For end-plate connections with two or more bolt-rows in tension, the basic components related to all of these bolt-rows should be represented by a single equivalent stiffness coefficient k_{eq} determined from:

$$k_{eq} = \frac{\sum_r k_{eff,r} h_r}{z_{eq}} \quad \text{... (6.29)}$$

where:

h_r is the distance between bolt-row r and the centre of compression;

$k_{eff,r}$ is the effective stiffness coefficient for bolt-row r taking into account the stiffness coefficients k_i for the basic components listed in 6.3.3.1(4) or 6.3.3.1(5) as appropriate;

z_{eq} is the equivalent lever arm, see 6.3.3.1(3).

(2) The effective stiffness coefficient $k_{eff,r}$ for bolt-row r should be determined from:

$$k_{eff,r} = \frac{1}{\sum_i \frac{1}{k_{i,r}}} \quad \text{... (6.30)}$$

where:

$k_{i,r}$ is the stiffness coefficient representing component i relative to bolt-row r.

(3) The equivalent lever arm z_{eq} should be determined from:

$$z_{eq} = \frac{\sum_r k_{eff,r}\, h_r^2}{\sum_r k_{eff,r}\, h_r} \qquad \text{... (6.31)}$$

(4) In the case of a beam-to-column joint with an end-plate connection, k_{eq} should be based upon (and replace) the stiffness coefficients k_i for:

- the column web in tension (k_3);
- the column flange in bending (k_4);
- the end-plate in bending (k_5);
- the bolts in tension (k_{10}).

(5) In the case of a beam splice with bolted end-plates, k_{eq} should be based upon (and replace) the stiffness coefficients k_i for:

- the end-plates in bending (k_5);
- the bolts in tension (k_{10}).

6.3.3.2 Simplified method for extended end-plates with two bolt-rows in tension

(1) For extended end-plate connections with two bolt-rows in tension, (one in the extended part of the end-plate and one between the flanges of the beam, see Figure 6.20), a set of modified values may be used for the stiffness coefficients of the related basic components to allow for the combined contribution of both bolt-rows. Each of these modified values should be taken as twice the corresponding value for a single bolt-row in the extended part of the end-plate.

NOTE: This approximation leads to a slightly lower estimate of the rotational stiffness.

(2) When using this simplified method, the lever arm z should be taken as equal to the distance from the centre of compression to a point midway between the two bolt-rows in tension, see Figure 6.20.

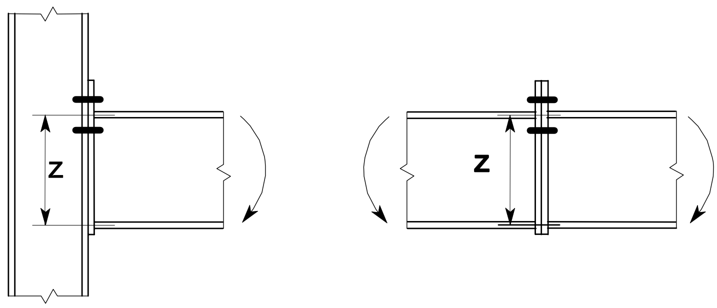


Figure 6.20: Lever arm z for simplified method

6.3.4 Column bases

(1) The rotational stiffness, S_j, of a column base subject to combined axial force and bending moment should be calculated using the method given in Table 6.12. This method uses the following stiffness coefficients:

$k_{T,l}$ is the tension stiffness coefficient of the left hand side of the joint and should be taken as equal to the sum of the stiffness coefficients k_{15} and k_{16} (given in Table 6.11) acting on the left hand side of the joint.

$k_{T,r}$ is the tension stiffness coefficient of the right hand side of the joint and should be taken as equal to the sum of the stiffness coefficients k_{15} and k_{16} (given in Table 6.11) acting on the right hand side of the joint.

$k_{C,l}$ is the compression stiffness coefficient of the left hand side of the joint and should be taken as equal to the stiffness coefficient k_{13} (given in Table 6.11) acting on the left hand side of the joint.

$k_{C,r}$ is the compression stiffness coefficient of the right hand side of the joint and should be taken as equal to the stiffness coefficient k_{13} (given in Table 6.11) acting on the right hand side of the joint.

(2) For the calculation of $z_{T,l}$, $z_{C,l}$, $z_{T,r}$, $z_{C,r}$ see 6.2.8.1.

Table 6.12: Rotational stiffness S_j of column bases

Loading	Lever arm z	Rotational stiffness $S_{j,ini}$	
Left side in tension Right side in compression	$z = z_{T,l} + z_{C,r}$	$N_{Ed} > 0$ and $e > z_{T,l}$	$N_{Ed} \leq 0$ and $e \leq -z_{C,r}$
		$\dfrac{E z^2}{\mu(1/k_{T,l} + 1/k_{C,r})} \dfrac{e}{e + e_k}$ where $e_k = \dfrac{z_{C,r}k_{C,r} - z_{T,l}k_{T,l}}{k_{T,l} + k_{C,r}}$	
Left side in tension Right side in tension	$z = z_{T,l} + z_{T,r}$	$N_{Ed} > 0$ and $0 < e < z_{T,l}$	$N_{Ed} > 0$ and $-z_{T,r} < e \leq 0$
		$\dfrac{E z^2}{\mu(1/k_{T,l} + 1/k_{T,r})} \dfrac{e}{e + e_k}$ where $e_k = \dfrac{z_{T,r}k_{T,r} - z_{T,l}k_{T,l}}{k_{T,l} + k_{T,r}}$	
Left side in compression Right side in tension	$z = z_{C,l} + z_{T,r}$	$N_{Ed} > 0$ and $e \leq -z_{T,r}$	$N_{Ed} \leq 0$ and $e > z_{C,l}$
		$\dfrac{E z^2}{\mu(1/k_{C,l} + 1/k_{T,r})} \dfrac{e}{e + e_k}$ where $e_k = \dfrac{z_{T,r}k_{T,r} - z_{C,l}k_{C,l}}{k_{C,l} + k_{T,r}}$	
Left side in compression Right side in compression	$z = z_{C,l} + z_{C,r}$	$N_{Ed} \leq 0$ and $0 < e < z_{C,l}$	$N_{Ed} \leq 0$ and $-z_{C,r} < e \leq 0$
		$\dfrac{E z^2}{\mu(1/k_{C,l} + 1/k_{C,r})} \dfrac{e}{e + e}$ where $e_k = \dfrac{z_{C,r}k_{C,r} - z_{C,l}k_{C,l}}{k_{C,l} + k_{C,r}}$	
$M_{Ed} > 0$ is clockwise, $N_{Ed} > 0$ is tension, μ see 6.3.1(6). $e = \dfrac{M_{Ed}}{N_{Ed}} = \dfrac{M_{Rd}}{N_{Rd}}$			

6.4 Rotation capacity

6.4.1 General

(1) In the case of rigid plastic global analysis, a joint at a plastic hinge location should have sufficient rotation capacity.

(2) The rotation capacity of a bolted or welded joint should be determined using the provisions given in 6.4.2 or 6.4.3. The design methods given in these clauses are only valid for S235, S275 and S355 steel grades and for joints in which the axial force N_{Ed} in the connected member does not exceed 5% of the design plastic resistance $N_{p\ell,Rd}$ of its cross-section.

(3) As an alternative to 6.4.2 and 6.4.3 the rotation capacity of a joint need not be checked provided that the design moment resistance $M_{j,Rd}$ of the joint is at least 1.2 times the design plastic moment resistance $M_{pl,Rd}$ of the connected member.

(4) In cases not covered by 6.4.2 and 6.4.3 the rotation capacity may be determined by testing in accordance with EN 1990, Annex D. Alternatively, appropriate calculation models may be used, provided that they are based on the results of tests in accordance with EN1990.

6.4.2 Bolted joints

(1) A beam-to-column joint in which the design moment resistance of the joint $M_{j,Rd}$ is governed by the design resistance of the column web panel in shear, may be assumed to have adequate rotation capacity for plastic global analysis, provided that $d/t_w \leq 69\varepsilon$.

(2) A joint with either a bolted end-plate or angle flange cleat connection may be assumed to have sufficient rotation capacity for plastic analysis, provided that both of the following conditions are satisfied:

a) the design moment resistance of the joint is governed by the design resistance of either:

- the column flange in bending or
- the beam end-plate or tension flange cleat in bending.

b) the thickness t of either the column flange or the beam end-plate or tension flange cleat (not necessarily the same basic component as in (a)) satisfies:

$$t \leq 0{,}36\, d \sqrt{f_{ub} / f_y} \qquad \text{... (6.32)}$$

where:

f_y is the yield strength of the relevant basic component.

(3) A joint with a bolted connection in which the design moment resistance $M_{j,Rd}$ is governed by the design resistance of its bolts in shear, should not be assumed to have sufficient rotation capacity for plastic global analysis.

6.4.3 Welded Joints

(1) The rotation capacity ϕ_{Cd} of a welded beam-to-column connection may be assumed to be not less that the value given by the following expression provided that its column web is stiffened in compression but unstiffened in tension, and its design moment resistance is not governed by the design shear resistance of the column web panel, see 6.4.2(1):

$$\phi_{Cd} = 0{,}025\, h_c / h_b \qquad \text{... (6.33)}$$

where:

h_b is the depth of the beam;

h_c is the depth of the column.

(2) An unstiffened welded beam-to-column joint designed in conformity with the provisions of this section, may be assumed to have a rotation capacity ϕ_{Cd} of at least 0,015 radians.

7 Hollow section joints

7.1 General

7.1.1 Scope

(1) This section gives detailed application rules to determine the static design resistances of uniplanar and multiplanar joints in lattice structures composed of circular, square or rectangular hollow sections, and of uniplanar joints in lattice structures composed of combinations of hollow sections with open sections.

(2) The static design resistances of the joints are expressed in terms of maximum design axial and/or moment resistances for the brace members.

(3) These application rules are valid both for hot finished hollow sections to EN 10210 and for cold formed hollow sections to EN 10219, if the dimensions of the structural hollow sections fulfil the requirements of this section.

(4) For hot finished hollow sections and cold formed hollow sections the nominal yield strength of the end product should not exceed 460 N/mm^2. For end products with a nominal yield strength higher than 355 N/mm^2 , the static design resistances given in this section should be reduced by a factor 0,9.

(5) The nominal wall thickness of hollow sections should not be less than 2,5 mm.

(6) The nominal wall thickness of a hollow section chord should not be greater than 25 mm unless special measures have been taken to ensure that the through thickness properties of the material will be adequate.

(7) For fatigue assessment see EN 1993-1-9.

(8) The types of joints covered are indicated in Figure 7.1.

7.1.2 Field of application

(1) The application rules for hollow section joints may be used only where all of the conditions given in 7.1.2(2) to 7.1.2(8) are satisfied.

(2) The compression elements of the members should satisfy the requirements for Class 1 or Class 2 given in EN 1993-1-1 for the condition of pure bending.

(3) The angles θ_i between the chords and the brace members, and between adjacent brace members, should satisfy:

$$\theta_i \geq 30°$$

(4) The ends of members that meet at a joint should be prepared in such a way that their cross-sectional shape is not modified. Flattened end connections and cropped end connections are not covered in this section.

(5) In gap type joints, in order to ensure that the clearance is adequate for forming satisfactory welds, the gap between the brace members should not be less than $(t_1 + t_2)$.

(6) In overlap type joints, the overlap should be large enough to ensure that the interconnection of the brace members is sufficient for adequate shear transfer from one brace to the other. In any case the overlap should be at least 25%.

(7) Where overlapping brace members have different thicknesses and/or different strength grades, the member with the lowest $t_i f_{yi}$ value should overlap the other member.

(8) Where overlapping brace members are of different widths, the narrower member should overlap the wider one.

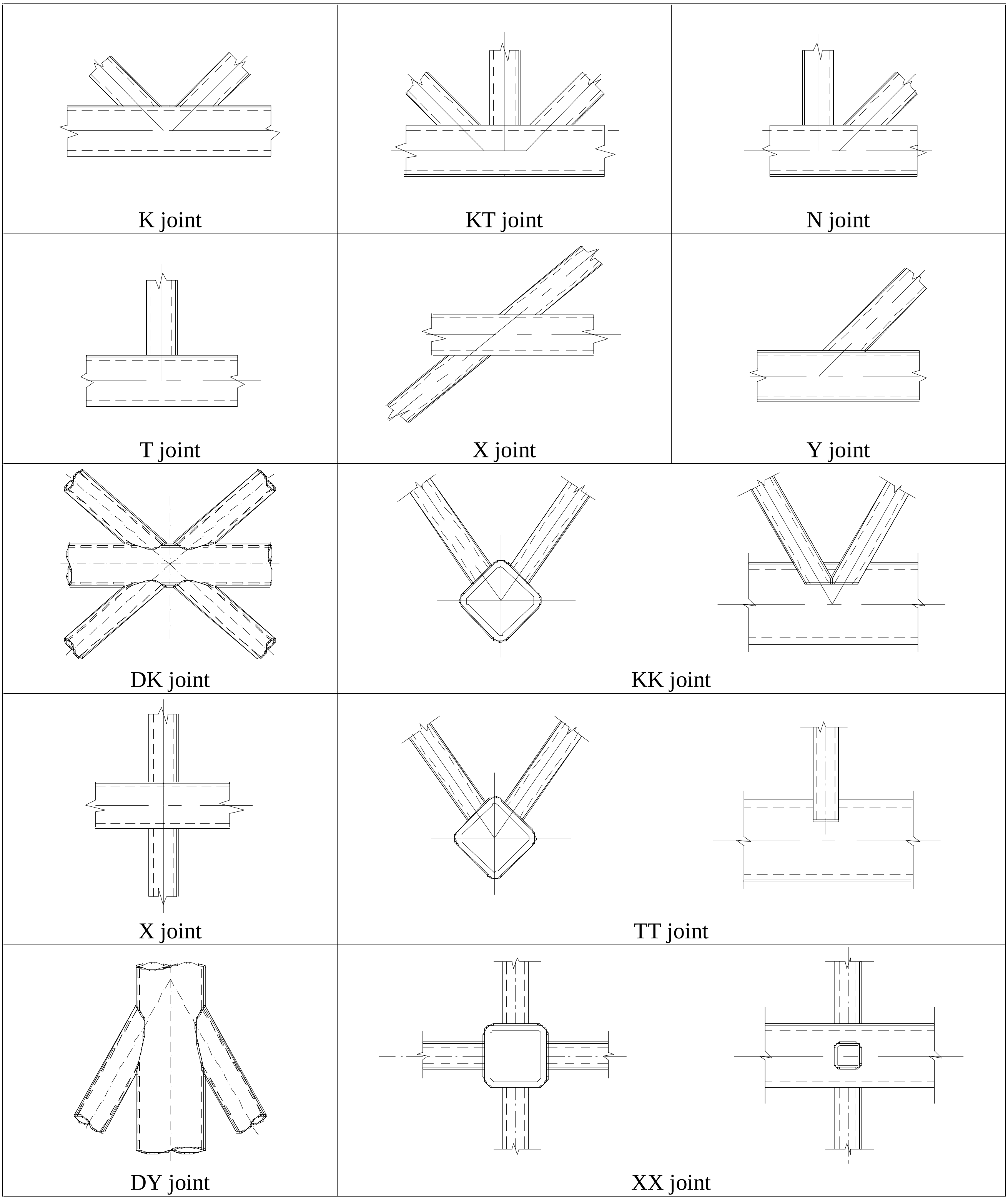


Figure 7.1: Types of joints in hollow section lattice girders

7.2 Design

7.2.1 General

(1) The design values of the internal axial forces both in the brace members and in the chords at the ultimate limit state should not exceed the design resistances of the members determined from EN 1993-1-1.

(2) The design values of the internal axial forces in the brace members at the ultimate limit state should also not exceed the design resistances of the joints given in 7.4, 7.5 or 7.6 as appropriate.

(3) The stresses $\sigma_{0,Ed}$ or $\sigma_{p,Ed}$ in the chord at a joint should be determined from:

$$\sigma_{0,Ed} = \frac{N_{0,Ed}}{A_0} + \frac{M_{0,Ed}}{W_{e(,0}} \quad \text{... (7.1)}$$

$$\sigma_{p,Ed} = \frac{N_{p,Ed}}{A_0} + \frac{M_{0,Ed}}{W_{e(,0}} \quad \text{... (7.2)}$$

where:

$$N_{p,Ed} = N_{0,Ed} - \sum_{i>0} N_{i,Ed} \cos\theta_i$$

7.2.2 Failure modes for hollow section connections

(1) The design joint resistances of connections between hollow sections and of connections between hollow sections and open sections, should be based on the following failure modes as applicable:

a) **Chord face failure** (plastic failure of the chord face) or chord plastification (plastic failure of the chord cross-section);

b) **Chord side wall failure** (or **chord web failure**) by yielding, crushing or instability (crippling or buckling of the chord side wall or chord web) under the compression brace member;

c) **Chord shear failure**;

d) **Punching shear** failure of a hollow section chord wall (crack initiation leading to rupture of the brace members from the chord member);

e) **Brace failure** with reduced effective width (cracking in the welds or in the brace members);

f) **Local buckling** failure of a brace member or of a hollow section chord member at the joint location.

NOTE: The phrases printed in boldface type in this list are used to describe the various failure modes in the tables of design resistances given in 7.4 to 7.6.

(2) Figure 7.2 illustrates failure modes (a) to (f) for joints between CHS brace and chord members.

(3) Figure 7.3 illustrates failure modes (a) to (f) for joints between RHS brace and chord members.

(4) Figure 7.4 illustrates failure modes (a) to (f) for joints between CHS or RHS brace members and I or H section chord members.

(5) Although the resistance of a joint with properly formed welds is generally higher under tension than under compression, the design resistance of the joint is generally based on the resistance of the brace in compression to avoid the possible excessive local deformation or reduced rotation capacity or deformation capacity which might otherwise occur.

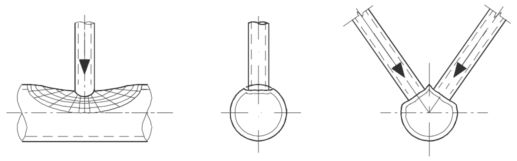
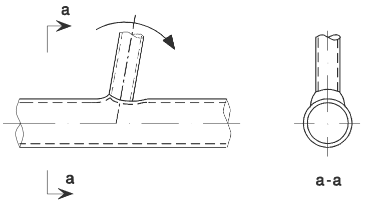
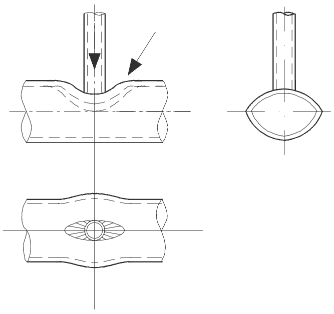
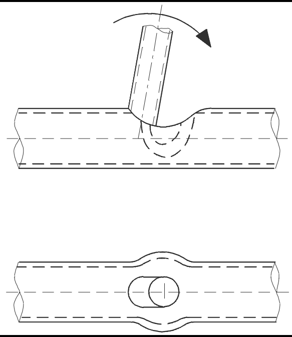
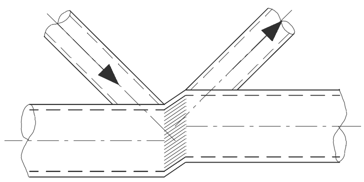
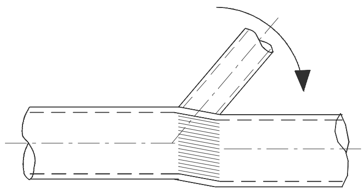
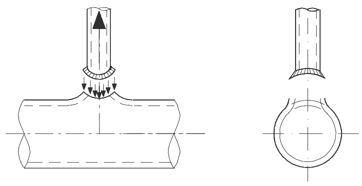
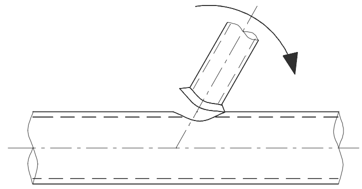
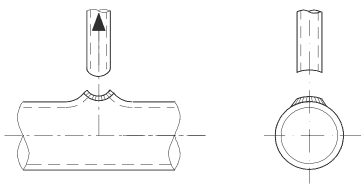
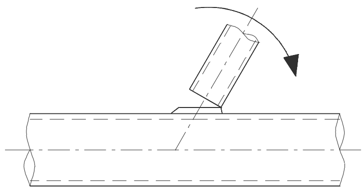
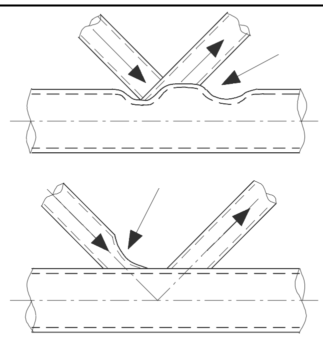
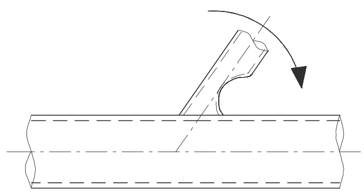
Mode	Axial loading	Bending moment
a		
b		
c		
d		
e		
f		

Figure 7.2: Failure modes for joints between CHS members

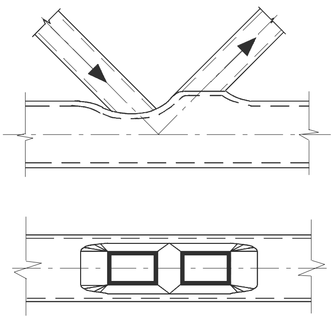
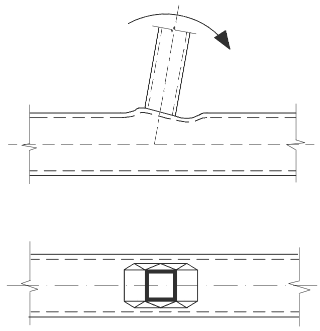
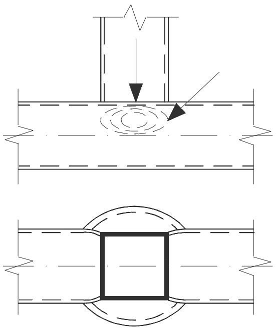
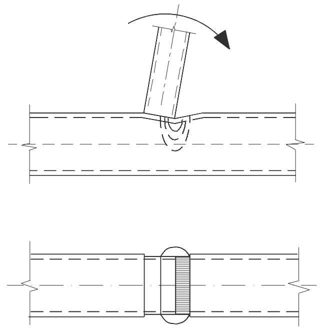
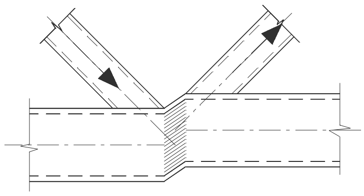
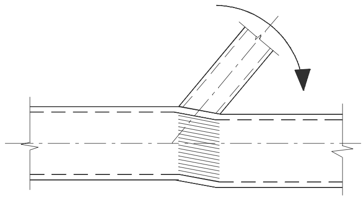
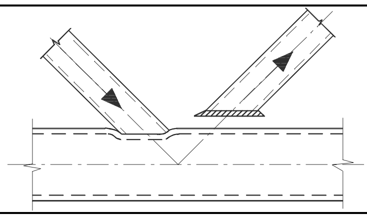
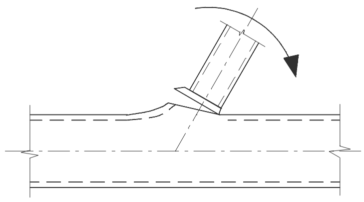
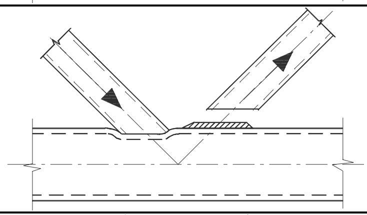
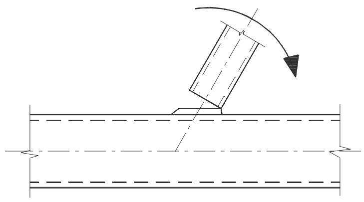
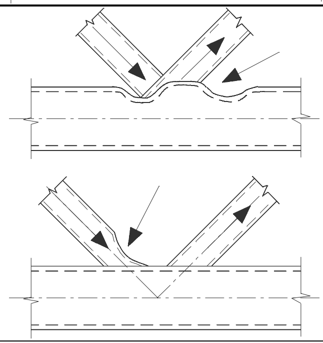
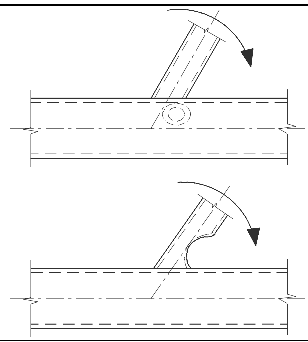
Mode	Axial loading	Bending moment
a		
b		
c		
d		
e		
f		

Figure 7.3: Failure modes for joints between RHS brace members and RHS chord members

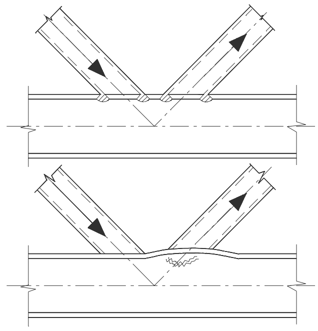
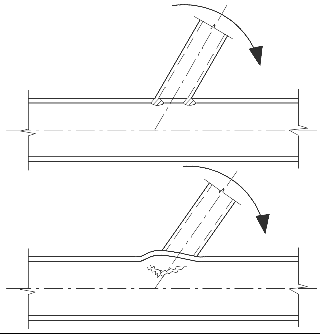
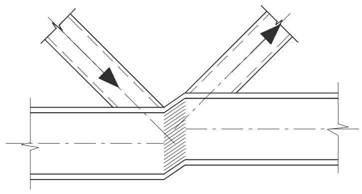
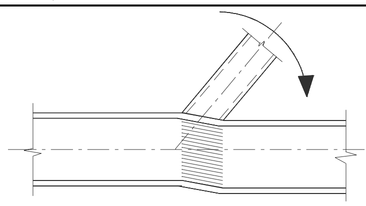
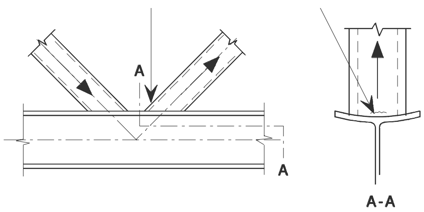
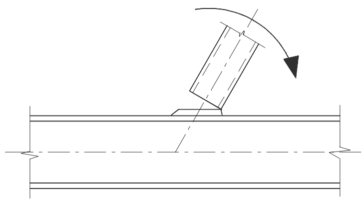
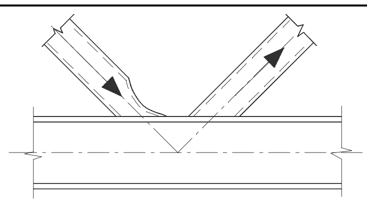
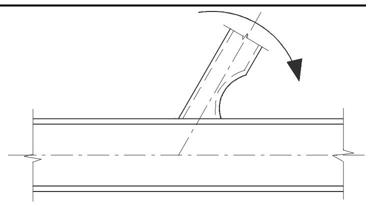
Mode	Axial loading	Bending moment
a	–	–
b		
c		
d	–	–
e		
f		

Figure 7.4: Failure modes for joints between CHS or RHS brace members and I or H section chord members

7.3 Welds

7.3.1 Design resistance

(1) The welds connecting the brace members to the chords should be designed to have sufficient resistance to allow for non-uniform stress-distributions and sufficient deformation capacity to allow for redistribution of bending moments.

(2) In welded joints, the connection should normally be formed around the entire perimeter of the hollow section by means of a butt weld, a fillet weld, or combinations of the two. However in partially overlapping joints the hidden part of the connection need not be welded, provided that the axial forces in the brace members are such that their components perpendicular to the axis of the chord do not differ by more than 20%.

(3) Typical weld details are indicated in 2.8 Reference Standards: Group 7.

(4) The design resistance of the weld, per unit length of perimeter of a brace member, should not normally be less than the design resistance of the cross-section of that member per unit length of perimeter.

(5) The required throat thickness should be determined from section 4.

(6) The criterion given in 7.3.1(4) may be waived where a smaller weld size can be justified both with regard to resistance and with regard to deformation capacity and rotation capacity, taking account of the possibility that only part of its length is effective.

(7) For rectangular structural hollow sections the design throat thickness of flare groove welds is defined in Figure 7.5.

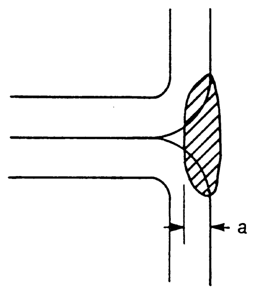


Figure 7.5: Design throat thickness of flare groove welds in rectangular structural hollow section

(8) For welding in cold-formed zones, see 4.14.

7.4 Welded joints between CHS members

7.4.1 General

(1) Provided that the geometry of the joints is within the range of validity given in Table 7.1, the design resistances of welded joints between circular hollow section members should be determined using 7.4.2 and 7.4.3.

(2) For joints within the range of validity given in Table 7.1, only chord face failure and punching shear need be considered. The design resistance of a connection should be taken as the minimum value for these two criteria.

(3) For joints outside the range of validity given in Table 7.1, all the criteria given in 7.2.2 should be considered. In addition, the secondary moments in the joints caused by their rotational stiffness should be taken into account.

Table 7.1: Range of validity for welded joints between CHS brace members and CHS chords

$0,2 \leq d_i/d_0 \leq 1,0$
Class 2 and $10 \leq d_0/t_0 \leq 50$ generally but $10 \leq d_0/t_0 \leq 40$ for X joints
Class 2 and $10 \leq d_i / t_i \leq 50$
$\lambda_{ov} \geq 25\%$
$g \geq t_1 + t_2$

7.4.2 Uniplanar joints

(1) In brace member connections subject only to axial forces, the design internal axial force $N_{i,Ed}$ should not exceed the design axial resistance of the welded joint $N_{i,Rd}$ obtained from Table 7.2, Table 7.3 or Table 7.4 as appropriate.

(2) Brace member connections subject to combined bending and axial force should satisfy:

$$\frac{N_{i,Ed}}{N_{i,Rd}} + \left[\frac{|M_{ip,i,Ed}|}{M_{ip,i,Rd}}\right]^2 + \frac{M_{op,i,Ed}}{M_{op,i,Rd}} \leq 1,0 \qquad \text{... (7.3)}$$

where:

$M_{ip,i,Rd}$ is the design in-plane moment resistance;

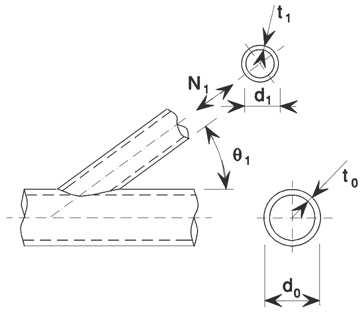
$M_{ip,i,Ed}$ is the design in-plane internal moment;

$M_{op,i,Rd}$ is the design out-of-plane moment resistance;

$M_{op,i,Ed}$ is the design out-of-plane internal moment.

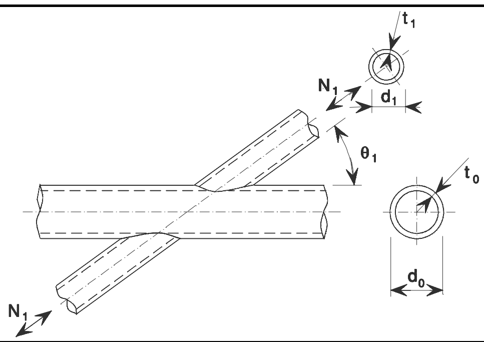
Table 7.2: Design axial resistances of welded joints between CHS brace members and CHS chords

Chord face failure - T and Y joints



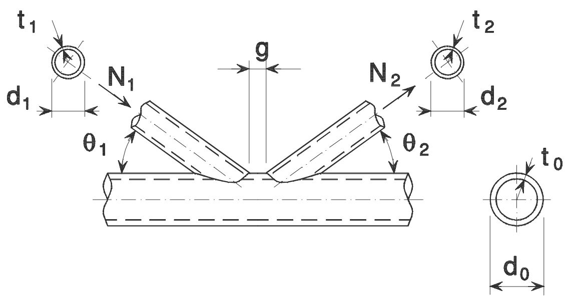
$$N_{1,\mathrm{Rd}} = \frac{\gamma^{0,2} k_p f_{y0} t_0^2}{\sin\theta_1}(2{,}8 + 14{,}2\beta^2)/\gamma_{M5}$$

Chord face failure - X joints



$$N_{1,\mathrm{Rd}} = \frac{k_p f_{y0} t_0^2}{\sin\theta_1}\frac{5{,}2}{(1-0{,}81\beta)}/\gamma_{M5}$$

Chord face failure - K and N gap or overlap joints



$$N_{1,\mathrm{Rd}} = \frac{k_g k_p f_{y0} t_0^2}{\sin\theta_1}\left(1{,}8 + 10{,}2\frac{d_1}{d_0}\right)/\gamma_{M5}$$

$$N_{2,\mathrm{Rd}} = \frac{\sin\theta_1}{\sin\theta_2} N_{1,Rd}$$

Punching shear failure - K, N and KT gap joints and all T, Y and X joints [i = 1, 2 or 3]

When $d_i \le d_0 - 2t_0$: $N_{i,\mathrm{Rd}} = \dfrac{f_{y0}}{\sqrt{3}} t_0 \pi d_i \dfrac{1+\sin\theta_i}{2\sin^2\theta_i}/\gamma_{M5}$

Factors k_g and k_p

$$k_g = \gamma^{0,2}\left(1 + \frac{0{,}024\gamma^{1,2}}{1+\exp(0{,}5g/t_0 - 1{,}33)}\right)$$ (see Figure 7.6)

For $n_p > 0$ (compression): $k_p = 1 - 0{,}3\, n_p(1 + n_p)$ but $k_p \le 1{,}0$

For $n_p \le 0$ (tension): $k_p = 1{,}0$

Table 7.3: Design resistances of welded joints connecting gusset plates to CHS members

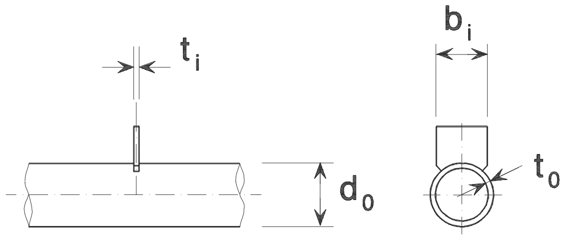
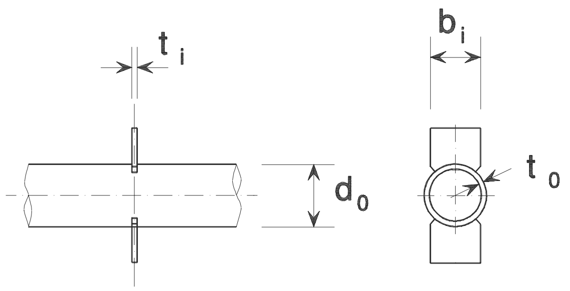
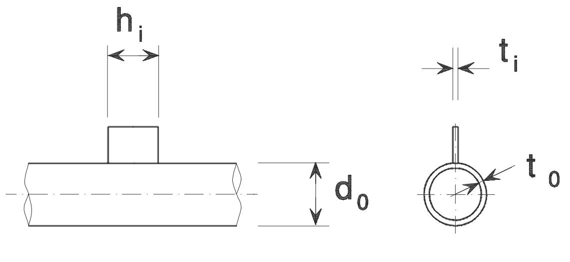
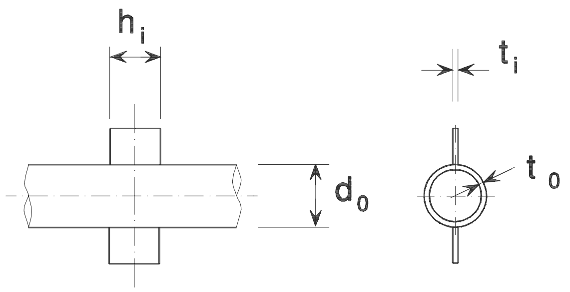
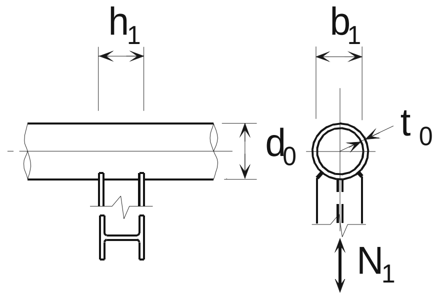
<table>
<tr><td colspan="2">Chord face failure</td></tr>
<tr><td>
</td><td>$N_{\mathrm{i,Rd}} = k_p f_{y0} t_0^2 (4 + 20\beta^2) / \gamma_{M5}$
$M_{\mathrm{ip,i,Rd}} = 0$
$M_{\mathrm{op,i,Rd}} = 0{,}5\; b_{\mathrm{i}}\, N_{\mathrm{i,Rd}}$</td></tr>
<tr><td>
</td><td>$N_{\mathrm{i,Rd}} = \dfrac{5 k_p f_{y0} t_0^2}{1 - 0{,}81\beta} / \gamma_{M5}$
$M_{\mathrm{ip,i,Rd}} = 0$
$M_{\mathrm{op,i,Rd}} = 0{,}5\; b_{\mathrm{i}}\, N_{\mathrm{i,Rd}}$</td></tr>
<tr><td>
</td><td>$N_{\mathrm{i,Rd}} = 5 k_p f_{y0} t_0^2 (1 + 0{,}25\eta) / \gamma_{M5}$
$M_{\mathrm{ip,i,Rd}} = h_{\mathrm{i}}\, N_{\mathrm{i,Rd}}$
$M_{\mathrm{op,i,Rd}} = 0$</td></tr>
<tr><td>
</td><td>$N_{\mathrm{i,Rd}} = 5 k_p f_{y0} t_0^2 (1 + 0{,}25\eta) / \gamma_{M5}$
$M_{\mathrm{ip,i,Rd}} = h_{\mathrm{i}}\, N_{\mathrm{i,Rd}}$
$M_{\mathrm{op,i,Rd}} = 0$</td></tr>
<tr><td colspan="2">Punching shear failure</td></tr>
<tr><td colspan="2">$\sigma_{\max} t_i = (N_{Ed} / A + M_{Ed} / W_{el}) t_i \le 2 t_0 (f_{y0} / \sqrt{3}) / \gamma_{M5}$</td></tr>
<tr><td>Range of validity</td><td>Factor k_{p}</td></tr>
<tr><td>In addition to the limits given in Table 7.1:
$\beta \ge 0{,}4$ and $\eta \le 4$
where $\beta = b_{\mathrm{i}}/d_0$ and $\eta = h_{\mathrm{i}}/d_0$</td><td>For $n_{\mathrm{p}} > 0$ (compression):
$k_{\mathrm{p}} = 1 - 0{,}3\, n_{\mathrm{p}}(1 + n_{\mathrm{p}})$ but $k_{\mathrm{p}} \le 1{,}0$
For $n_{\mathrm{p}} \le 0$ (tension): $k_{\mathrm{p}} = 1{,}0$</td></tr>
</table>

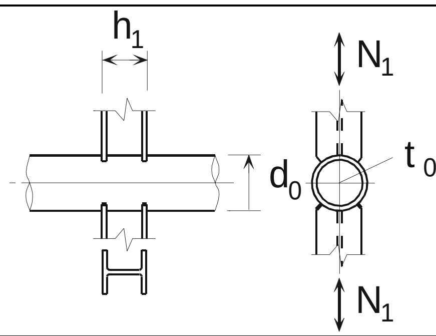
Table 7.4: Design resistances of welded joints connecting I, H or RHS sections to CHS members

<table>
<tr><td colspan="2">Chord face failure</td></tr>
<tr><td>
</td><td>

$N_{1,\mathrm{Rd}} = k_p f_{y0}\, t_0^{\,2}\,(4 + 20\beta^2)(1 + 0{,}25\eta) / \gamma_{M5}$

$M_{ip,1,\mathrm{Rd}} = h_1\, N_{1,\mathrm{Rd}}/(1 + 0{,}25\eta)$

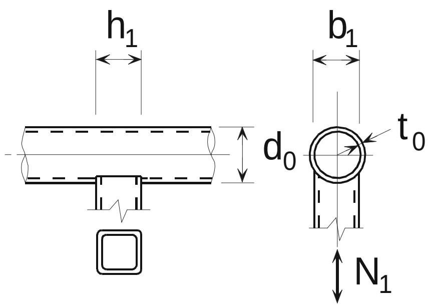
$M_{op,1,\mathrm{Rd}} = 0{,}5\, b_1\, N_{1,\mathrm{Rd}}$

</td></tr>
<tr><td>
</td><td>

$N_{1,\mathrm{Rd}} = \dfrac{5 k_p f_{y0} t_0^{\,2}}{1 - 0{,}81\beta}\left(1 + 0{,}25\eta\right) / \gamma_{M5}$

$M_{ip,1,\mathrm{Rd}} = h_1\, N_{1,\mathrm{Rd}}/(1 + 0{,}25\eta)$

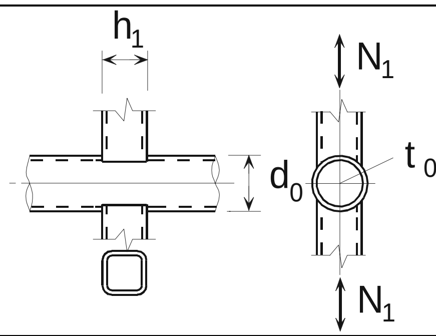
$M_{op,1,\mathrm{Rd}} = 0{,}5\, b_1\, N_{1,\mathrm{Rd}}$

</td></tr>
<tr><td>
</td><td>

$N_{1,\mathrm{Rd}} = k_p f_{y0}\, t_0^{\,2}\,(4 + 20\beta^2)(1 + 0{,}25\eta) / \gamma_{M5}$

$M_{ip,1,\mathrm{Rd}} = h_1\, N_{1,\mathrm{Rd}}$

$M_{op,1,\mathrm{Rd}} = 0{,}5\, b_1\, N_{1,\mathrm{Rd}}$

</td></tr>
<tr><td>
</td><td>

$N_{1,\mathrm{Rd}} = \dfrac{5 k_p f_{y0} t_0^{\,2}}{1 - 0{,}81\beta}\left(1 + 0{,}25\eta\right) / \gamma_{M5}$

$M_{ip,1,\mathrm{Rd}} = h_1\, N_{1,\mathrm{Rd}}$

$M_{op,1,\mathrm{Rd}} = 0{,}5\, b_1\, N_{1,\mathrm{Rd}}$

</td></tr>
<tr><td colspan="2">Punching shear failure</td></tr>
<tr><td colspan="2">

I or H sections: $\sigma_{max} t_1 = \left(N_{Ed}/A + M_{Ed}/W_{e(}\right) t_1 \le 2 t_0 (f_{y0}/\sqrt{3}) / \gamma_{M5}$

RHS sections: $\sigma_{max} t_1 = \left(N_{Ed}/A + M_{Ed}/W_{e(}\right) t_1 \le t_0 (f_{y0}/\sqrt{3}) / \gamma_{M5}$

</td></tr>
<tr><td>Range of validity</td><td>Factor k_p</td></tr>
<tr><td>

In addition to the limits given in Table 7.1:

$\beta \ge 0{,}4$ and $\eta \le 4$

where $\beta = b_1 / d_0$ and $\eta = h_1 / d_0$

</td><td>

For $n_p > 0$ (compression):

$k_p = 1 - 0{,}3\, n_p\, (1 + n_p)$ but $k_p \le 1{,}0$

For $n_p \le 0$ (tension): $k_p = 1{,}0$

</td></tr>
</table>

(3) The design internal moment $M_{i,Ed}$ may be taken as the value at the point where the centreline of the brace member meets the face of the chord member.

(4) The design in-plane moment resistance and the design out-of-plane moment resistance $M_{i,Rd}$ should be obtained from Table 7.3, Table 7.4 or Table 7.5 as appropriate.

(5) The special types of welded joints indicated in Table 7.6 should satisfy the appropriate design criteria specified for each type in that table.

(6) Values of the factor k_g which is used in Table 7.2 for K, N and KT joints are given in Figure 7.6. The factor k_g is used to cover both gap type and overlap type joints by adopting g for both the gap and the overlap and using negative values of g to represent the overlap q as defined in Figure 1.3(b).

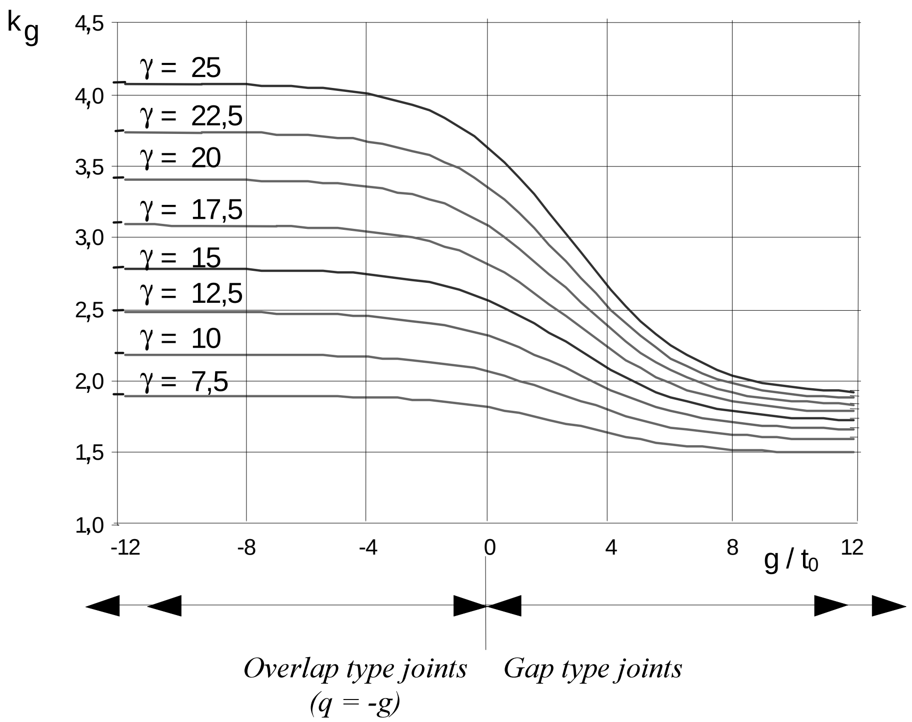


Figure 7.6: Values of the factor k_g for use in Table 7.2

Table 7.5: Design resistance moments of welded joints between CHS brace members and CHS chords

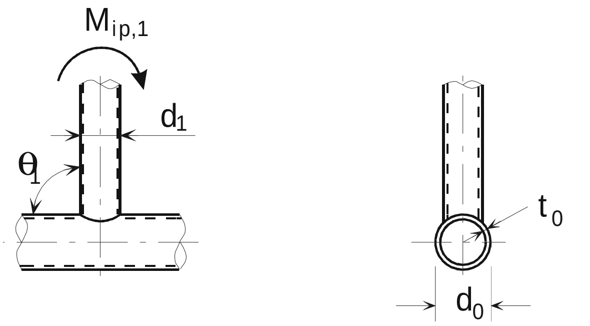
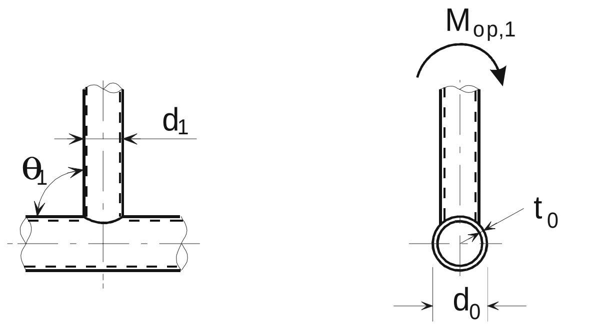
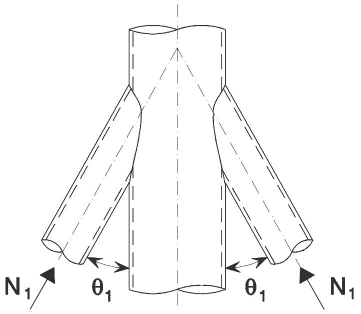
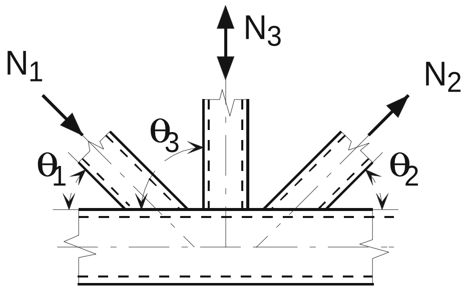
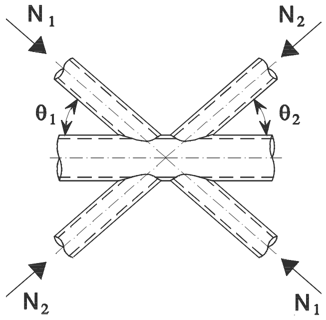
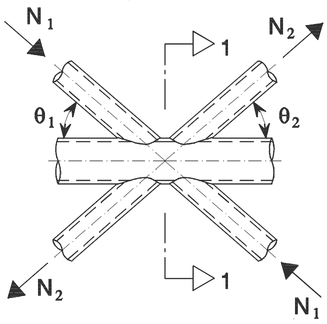
Chord face failure - T, X, and Y joints	
	$M_{\text{ip,1,Rd}} = 4{,}85 \dfrac{f_{y0} t_0^2 d_1}{\sin\theta_1} \sqrt{\gamma}\,\beta k_p / \gamma_{M5}$
Chord face failure - K, N, T, X and Y joints	
	$M_{\text{op,1,Rd}} = \dfrac{f_{y0} t_0^2 d_1}{\sin\theta_1} \dfrac{2{,}7}{1-0{,}81\beta} k_p / \gamma_{M5}$
Punching shear failure - K and N gap joints and all T, X and Y joints	
When $d_1 \le d_0 - 2t_0$: $M_{\text{ip,1,Rd}} = \dfrac{f_{y0} t_0 d_1^2}{\sqrt{3}} \dfrac{1+3\sin\theta_1}{4\sin^2\theta_1} / \gamma_{M5}$ $M_{\text{op,1,Rd}} = \dfrac{f_{y0} t_0 d_1^2}{\sqrt{3}} \dfrac{3+\sin\theta_1}{4\sin^2\theta_1} / \gamma_{M5}$	
Factor k_p	
For $n_p > 0$ (compression): $k_p = 1 - 0{,}3\, n_p (1 + n_p)$ but $k_p \le 1{,}0$ For $n_p \le 0$ (tension): $k_p = 1{,}0$	

Table 7.6: Design criteria for special types of welded joints between CHS brace members and CHS chords

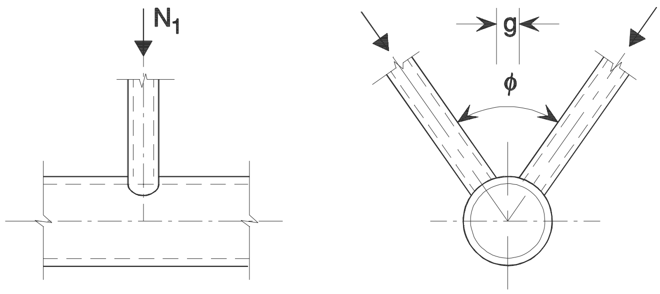
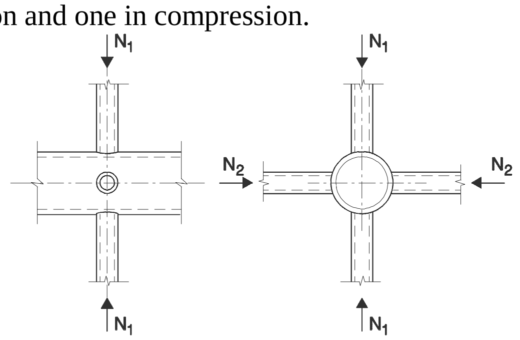
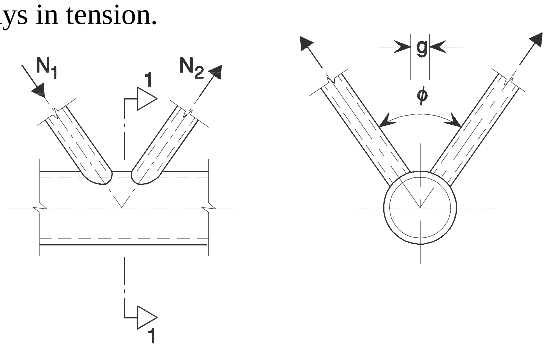
Type of joint	Design criteria
The forces may be either tension or compression but shall act in the same direction for both members. 	$N_{1,\mathrm{Ed}} \leq N_{1,\mathrm{Rd}}$ where $N_{1,\mathrm{Rd}}$ is the value of $N_{1,\mathrm{Rd}}$ for an X joint from Table 7.2.
Member 1 is always in compression and member 2 is always in tension. 	$N_{1,\mathrm{Ed}} \sin\theta_1 + N_{3,\mathrm{Ed}} \sin\theta_3 \leq N_{1,\mathrm{Rd}} \sin\theta_1$ $N_{2,\mathrm{Ed}} \sin\theta_2 \leq N_{1,\mathrm{Rd}} \sin\theta_1$ where $N_{1,\mathrm{Rd}}$ is the value of $N_{1,\mathrm{Rd}}$ for a K joint from Table 7.2 but with $\dfrac{d_1}{d_0}$ replaced by: $\dfrac{d_1 + d_2 + d_3}{3d_0}$
All bracing members shall always be in either compression or tension. 	$N_{1,\mathrm{Ed}} \sin\theta_1 + N_{2,\mathrm{Ed}} \sin\theta_2 \leq N_{\mathrm{x,Rd}} \sin\theta_{\mathrm{x}}$ where $N_{\mathrm{x,Rd}}$ is the value of $N_{\mathrm{x,Rd}}$ for an X joint from Table 7.2, where $N_{\mathrm{x,Rd}} \sin\theta_{\mathrm{x}}$ is the larger of: $\lvert N_{1,\mathrm{Rd}} \sin\theta_1 \rvert$ and $\lvert N_{2,\mathrm{Rd}} \sin\theta_2 \rvert$
Member 1 is always in compression and member 2 is always in tension. 	$N_{\mathrm{i,Ed}} \leq N_{\mathrm{i,Rd}}$ where $N_{\mathrm{i,Rd}}$ is the value of $N_{\mathrm{i,Rd}}$ for a K joint from Table 7.2, provided that, in a gap-type joint, at section 1-1 the chord satisfies: $\left[\dfrac{N_{0,Ed}}{N_{0,p(,Rd}}\right]^2 + \left[\dfrac{V_{0,Ed}}{V_{0,p(,Rd}}\right]^2 \leq 1{,}0$

7.4.3 Multiplanar joints

(1) In each relevant plane of a multiplanar joint, the design criteria given in 7.4.2 should be satisfied using the reduced design resistances obtained from 7.4.3(2).

(2) The design resistances for each relevant plane of a multiplanar joint should be determined by applying the appropriate reduction factor μ given in Table 7.7 to the resistance of the corresponding uniplanar joint calculated according to 7.4.2 by using the appropriate chord force for k_p .

Table 7.7: Reduction factors for multiplanar joints

Type of joint	Reduction factor μ
TT joint	$60° \leq \varphi \leq 90°$
Member 1 may be either tension or compression.  	$\mu = 1{,}0$
XX joint	
Members 1 and 2 can be either in compression or tension. $N_{2,Ed}/N_{1,Ed}$ is negative if one member is in tension and one in compression.  	$\mu = 1 + 0{,}33 N_{2,Ed} / N_{1,Ed}$ taking account of the sign of $N_{1,Ed}$ and $N_{2,Ed}$ where $\lvert N_{2,Ed} \rvert \leq \lvert N_{1,Ed} \rvert$
KK joint	$60° \leq \varphi \leq 90°$
Member 1 is always in compression and member 2 is always in tension.  	$\mu = 0{,}9$ provided that, in a gap-type joint, at section 1-1 the chord satisfies: $\left[\frac{N_{0,Ed}}{N_{p(,0,Rd}}\right]^2 + \left[\frac{V_{0,Ed}}{V_{p(,0,Rd}}\right]^2 \leq 1{,}0$

7.5 Welded joints between CHS or RHS brace members and RHS chord members

7.5.1 General

(1) Provided that the geometry of the joints is within the range of validity given in Table 7.8, the design resistances of welded joints between hollow section brace members and rectangular or square hollow section chord members may be determined using 7.5.2 and 7.5.3.

(2) For joints within the range of validity given in Table 7.8, only the design criteria covered in the appropriate table need be considered. The design resistance of a connection should be taken as the minimum value for all applicable criteria.

(3) For joints outside the range of validity given in Table 7.8, all the criteria given in 7.2.2 should be considered. In addition, the secondary moments in the joints caused by their rotational stiffness should be taken into account.

Table 7.8: Range of validity for welded joints between CHS or RHS brace members and RHS chord members

<table>
<tr><th rowspan="3">Type of joint</th><th colspan="6">Joint parameters [i = 1 or 2, j = overlapped brace]</th></tr>
<tr><th rowspan="2">b_i/b_0
or
d_i/b_0</th><th colspan="2">b_i/t_i and h_i/t_i or d_i/t_i</th><th rowspan="2">h_0/b_0
and
h_i/b_i</th><th rowspan="2">b_0/t_0
and
h_0/t_0</th><th rowspan="2">Gap or overlap

b_i/b_j</th></tr>
<tr><th>Compression</th><th>Tension</th></tr>
<tr><td>T, Y or X</td><td>$b_i/b_0 \geq 0{,}25$</td><td rowspan="2">$b_i/t_i \leq 35$
and
$h_i/t_i \leq 35$
and
Class 2</td><td rowspan="3">b_i/t_i
≤ 35
and
h_i/t_i
≤ 35</td><td rowspan="3">$\geq 0{,}5$
but
$\leq 2{,}0$</td><td>≤ 35
and
Class 2</td><td>–</td></tr>
<tr><td>K gap

N gap</td><td>$b_i/b_0 \geq 0{,}35$
and
$\geq 0{,}1 + 0{,}01\, b_0/t_0$</td><td>$\leq 35$
and
Class 2</td><td>$g/b_0 \geq 0{,}5(1-\beta)$
but $\leq 1{,}5(1-\beta)$ 1)

and as a minimum
$g \geq t_1 + t_2$</td></tr>
<tr><td>K overlap

N overlap</td><td>$b_i/b_0 \geq 0{,}25$</td><td>Class 1</td><td>Class 2</td><td>$\lambda_{ov} \geq 25\%$
but $\lambda_{ov} \leq 100\%$ 2)

and $b_i/b_j \geq 0{,}75$</td></tr>
<tr><td>Circular brace member</td><td>$d_i/b_0 \geq 0{,}4$

but $\leq 0{,}8$</td><td>Class 1</td><td>d_i/t_i
≤ 50</td><td colspan="3">As above but with d_i replacing b_i and d_j replacing b_j.</td></tr>
<tr><td colspan="7">1) If $g/b_0 > 1{,}5(1-\beta)$ and $g/b_0 > t_1 + t_2$ treat the joint as two separate T or Y joints.
2) The overlap may be increased to enable the toe of the overlapped brace to be welded to the chord.</td></tr>
</table>

7.5.2 Uniplanar joints

7.5.2.1 Unreinforced joints

(1) In brace member connections subject only to axial forces, the design internal axial force $N_{i,Ed}$ should not exceed the design axial resistance of the welded joint $N_{i,Rd}$, determined from 7.5.2.1(2) or 7.5.2.1(4) as appropriate.

(2) For welded joints between square or circular hollow section brace members and square hollow section chord members only, where the geometry of the joints is within the range of validity given in Table 7.8 and also satisfies the additional conditions given in Table 7.9, the design axial resistances may be determined from the expressions given in Table 7.10.

(3) For joints within the range of validity of Table 7.9, the only design criteria that need be considered are chord face failure and brace failure with reduced effective width. The design axial resistance should be taken as the minimum value for these two criteria.

NOTE: The design axial resistances for joints of hollow section brace members to square hollow section chords given in Table 7.10 have been simplified by omitting design criteria that are never critical within the range of validity of Table 7.9.

(4) The design axial resistances of any unreinforced welded joint between CHS or RHS brace members and RHS chords, within the range of validity of Table 7.8, may be determined using the expressions given in Table 7.11, Table 7.12 or Table 7.13 as appropriate. For reinforced joints see 7.5.2.2.

Table 7.9: Additional conditions for the use of Table 7.10

Type of brace	Type of joint	Joint parameters	
Square hollow section	T, Y or X	$b_i/b_0 \leq 0{,}85$	$b_0/t_0 \geq 10$
	K gap or N gap	$0{,}6 \leq \dfrac{b_1 + b_2}{2b_1} \leq 1{,}3$	$b_0/t_0 \geq 15$
Circular hollow section	T, Y or X		$b_0/t_0 \geq 10$
	K gap or N gap	$0{,}6 \leq \dfrac{d_1 + d_2}{2d_1} \leq 1{,}3$	$b_0/t_0 \geq 15$

Table 7.10: Design axial resistances of welded joints between square or circular hollow section

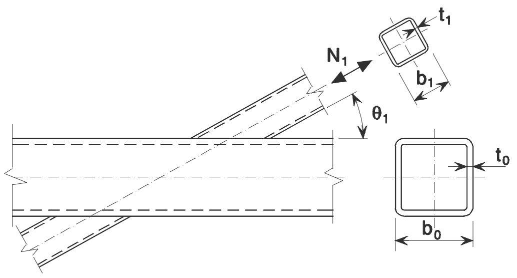
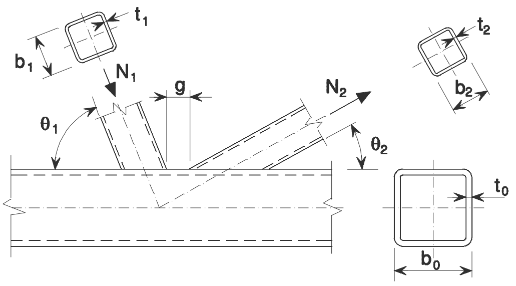
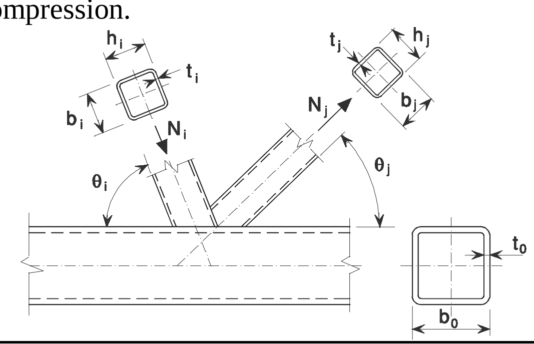
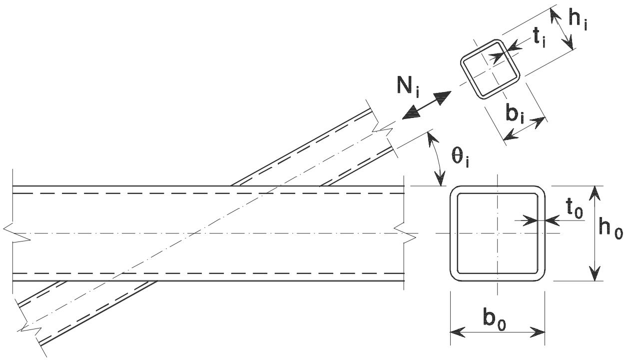
Type of joint	Design resistance [i = 1 or 2, j = overlapped brace]
T, Y and X joints	Chord face failure $\beta \leq 0{,}85$
	$N_{1,\mathrm{Rd}} = \dfrac{k_n f_{y0} t_0^{\ 2}}{(1-\beta)\sin\theta_1}\left(\dfrac{2\beta}{\sin\theta_1} + 4\sqrt{1-\beta}\right) / \gamma_{M5}$
K and N gap joints	Chord face failure $\beta \leq 1{,}0$
	$N_{i,\mathrm{Rd}} = \dfrac{8{,}9\gamma^{0,5} k_n f_{y0} t_0^2}{\sin\theta_i}\left(\dfrac{b_1 + b_2}{2b_0}\right) / \gamma_{M5}$
K and N overlap joints *)	Brace failure $25\% \leq \lambda_{ov} < 50\%$
Member i or member j may be either tension or compression but one shall be tension and the other compression.	$N_{i,\mathrm{Rd}} = f_{yi} t_i \left(b_{eff} + b_{e,ov} + \dfrac{\lambda_{ov}}{50}(2h_i - 4t_i)\right) / \gamma_{M5}$
	Brace failure $50\% \leq \lambda_{ov} < 80\%$
	$N_{i,\mathrm{Rd}} = f_{yi} t_i \left[b_{eff} + b_{e,ov} + 2h_i - 4t_i\right] / \gamma_{M5}$
	Brace failure $\lambda_{ov} \geq 80\%$
	$N_{i,\mathrm{Rd}} = f_{yi} t_i \left[b_i + b_{e,ov} + 2h_i - 4t_i\right] / \gamma_{M5}$
Parameters b_{eff}, $b_{\mathrm{e,ov}}$ and k_{n}	
$b_{\mathrm{eff}} = \dfrac{10}{b_0 / t_0}\dfrac{f_{y0} t_0}{f_{yi} t_i} b_i$ but $b_{\mathrm{eff}} \leq b_{\mathrm{i}}$	For $n > 0$ (compression): $k_{\mathrm{n}} = 1{,}3 - \dfrac{0{,}4n}{\beta}$ but $k_{\mathrm{n}} \leq 1{,}0$
$b_{\mathrm{e,ov}} = \dfrac{10}{b_j / t_j}\dfrac{f_{yj} t_j}{f_{ji} t_i} b_i$ but $b_{\mathrm{e,ov}} \leq b_{\mathrm{i}}$	For $n \leq 0$ (tension): $k_{\mathrm{n}} = 1{,}0$
For circular braces, multiply the above resistances by $\pi/4$, replace b_1 and h_1 by d_1 and replace b_2 and h_2 by d_2.	
*) Only the overlapping brace member i need be checked. The brace member efficiency (i.e. the design resistance of the joint divided by the design plastic resistance of the brace member) of the overlapped brace member j should be taken as equal to that of the overlapping brace member.	

Table 7.11: Design axial resistances of welded T, X and Y joints between RHS or CHS braces and RHS chords

Type of joint	Design resistance $[i = 1]$
	Chord face failure $\beta \leq 0{,}85$
	$N_{\text{i,Rd}} = \dfrac{k_n f_{y0} t_0^2}{(1-\beta)\sin\theta_1}\left(\dfrac{2\eta}{\sin\theta_1} + 4\sqrt{1-\beta}\right) / \gamma_{M5}$
	Chord side wall buckling [1)] $\beta = 1{,}0$ [2)]
	$N_{\text{i,Rd}} = \dfrac{f_b t_0}{\sin\theta_i}\left(\dfrac{2h_i}{\sin\theta_1} + 10t_0\right) / \gamma_{M5}$
	Brace failure $\beta \geq 0{,}85$
	$N_{\text{i,Rd}} = f_{yi} t_i (2h_i - 4t_i + 2b_{eff}) / \gamma_{M5}$
	Punching shear $0{,}85 \leq \beta \leq (1 - 1/\gamma)$
	$N_{\text{i,Rd}} = \dfrac{f_{y0} t_0}{\sqrt{3}\sin\theta_1}\left(\dfrac{2h_i}{\sin\theta_1} + 2b_{e,p}\right) / \gamma_{M5}$

1) For X joints with $\theta < 90°$ use the smaller of this value and the design shear resistance of the chord side walls given for K and N gap joints in Table 7.12.

2) For $0{,}85 \leq \beta \leq 1{,}0$ use linear interpolation between the value for chord face failure at $\beta = 0{,}85$ and the governing value for chord side wall failure at $\beta = 1{,}0$ (side wall buckling or chord shear).

For circular braces, multiply the above resistances by $\pi/4$, replace b_1 and h_1 by d_1 and replace b_2 and h_2 by d_2.

For tension: $f_b = f_{y0}$	$b_{\text{eff}} = \dfrac{10}{b_0/t_0}\dfrac{f_{y0}t_0}{f_{yi}t_i} b_i$ but $b_{\text{eff}} \leq b_i$
For compression: $f_b = \chi f_{y0}$ (T and Y joints); $f_b = 0{,}8\,\chi f_{y0}\sin\theta_i$ (X joints)	$b_{\text{e,p}} = \dfrac{10}{b_0 t_0} b_i$ but $b_{\text{e.p}} \leq b_i$
where χ is the reduction factor for flexural buckling obtained from EN 1993-1-1 using the relevant buckling curve and a normalized slenderness $\bar{\lambda}$ determined from: $\bar{\lambda} = 3{,}46\dfrac{\left(\dfrac{h_0}{t_0} - 2\right)\sqrt{\dfrac{1}{\sin\theta_i}}}{\pi\sqrt{\dfrac{E}{f_{y0}}}}$	For $n > 0$ (compression): $k_n = 1{,}3 - \dfrac{0{,}4n}{\beta}$ but $k_n \leq 1{,}0$; For $n \leq 0$ (tension): $k_n = 1{,}0$

Table 7.12: Design axial resistances of welded K and N joints between RHS or CHS braces and RHS chords

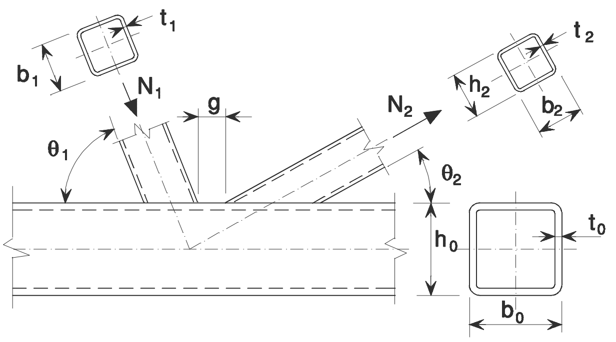
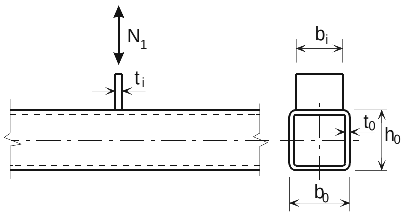
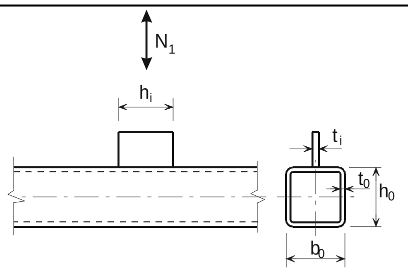
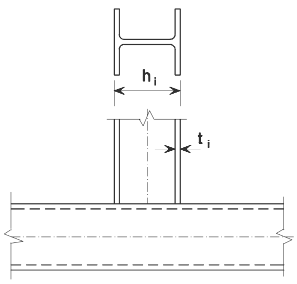
Type of joint	Design resistance $[i = 1 \text{ or } 2]$
K and N gap joints	Chord face failure
	$N_{\mathrm{i,Rd}} = \dfrac{8{,}9 k_n f_{y0} t_0^2 \sqrt{\gamma}}{\sin\theta_i}\left(\dfrac{b_1 + b_2 + h_1 + h_2}{4b_0}\right) / \gamma_{M5}$
	Chord shear
	$N_{\mathrm{i,Rd}} = \dfrac{f_{y0} A_v}{\sqrt{3}\sin\theta_i} / \gamma_{M5}$ $N_{\mathrm{0,Rd}} = \left[(A_0 - A_v) f_{y0} + A_v f_{y0}\sqrt{1 - (V_{Sd} / V_{pl,Rd})^2}\right] / \gamma_{M5}$
	Brace failure
	$N_{\mathrm{i,Rd}} = f_{yi} t_i (2h_i - 4t_i + b_i + b_{eff}) / \gamma_{M5}$
	Punching shear $\quad \beta \leq (1 - 1/\gamma)$
	$N_{\mathrm{i,Rd}} = \dfrac{f_{y0} t_0}{\sqrt{3}\sin\theta_i}\left(\dfrac{2h_i}{\sin\theta_i} + b_i + b_{e,p}\right) / \gamma_{M5}$
K and N overlap joints	As in Table 7.10.
For circular braces, multiply the above resistances by $\pi/4$, replace b_1 and h_1 by d_1 and replace b_2 and h_2 by d_2.	
$A_v = (2h_0 + \alpha b_0) t_0$ For a square or rectangular brace member: $\alpha = \sqrt{\dfrac{1}{1 + \dfrac{4g^2}{3t_0^2}}}$ where g is the gap, see Figure 1.3(a). For a circular brace member: $\alpha = 0$	$b_{\mathrm{eff}} = \dfrac{10}{b_0 / t_0} \dfrac{f_{y0} t_0}{f_{yi} t_i} b_i \quad$ but $b_{\mathrm{eff}} \leq b_{\mathrm{i}}$ $b_{\mathrm{e,p}} = \dfrac{10}{b_0 t_0} b_i \quad$ but $b_{\mathrm{e.p}} \leq b_{\mathrm{i}}$ For $n > 0$ (compression): $k_{\mathrm{n}} = 1{,}3 - \dfrac{0{,}4n}{\beta}$ but $k_{\mathrm{n}} \leq 1{,}0$ For $n \leq 0$ (tension): $k_{\mathrm{n}} = 1{,}0$

Table 7.13: Design resistances of welded joints connecting gusset plates or I or H sections to RHS members

Transverse plate	Brace failure $[i = 1]$
	$N_{1,\mathrm{Rd}} = f_{y1} t_1 b_{eff} / \gamma_{M5}$ *) Chord side wall crushing when $b_1 \geq b_0 - 2t_0$ $N_{1,\mathrm{Rd}} = f_{y0} t_0 (2t_1 + 10t_0) / \gamma_{M5}$ Punching shear when $b_1 \leq b_0 - 2t_0$ $N_{1,\mathrm{Rd}} = \dfrac{f_{y0} t_0}{\sqrt{3}} \left(2t_1 + 2b_{e,p}\right) / \gamma_{M5}$
Longitudinal plate	Chord face failure
 $t_1/b_0 \leq 0{,}2$	$N_{1,\mathrm{Rd}} = \dfrac{k_m f_{y0} t_0^2}{1 - t_1 / b_0} \left(2h_1 / b_0 + 4\sqrt{1 - t_1 / b_0}\right) / \gamma_{M5}$
I or H section	
	As a conservative approximation, if $\eta \geq 2\sqrt{1-\beta}$, $N_{1,\mathrm{Rd}}$ for an I or H section may be assumed to be equal to the design resistance of two transverse plates of similar dimensions to the flanges of the I or H section, determined as specified above. If $\eta < 2\sqrt{1-\beta}$, a linear interpolation between one and two plates should be made. $M_{ip,1,\mathrm{Rd}} = N_{1,\mathrm{Rd}} (h_1 - t_1)$
Range of validity	
In addition to the limits given in Table 7.8: $0{,}5 \leq \beta \leq 1{,}0$ $b_0/t_0 \leq 30$	
Parameters b_{eff}, $b_{\mathrm{e,p}}$ and k_{m}	
$b_{\mathrm{eff}} = \dfrac{10}{b_0 / t_0} \dfrac{f_{y0} t_0}{f_{y1} t_1} b_1$ but $b_{\mathrm{eff}} \leq b_{\mathrm{i}}$	For $n > 0$ (compression): $k_{\mathrm{m}} = 1{,}3(1 - n)$ but $k_{\mathrm{m}} \leq 1{,}0$
$b_{\mathrm{e,p}} = \dfrac{10}{b_0 / t_0} b_1$ but $b_{\mathrm{e.p}} \leq b_{\mathrm{i}}$	For $n \leq 0$ (tension): $k_{\mathrm{m}} = 1{,}0$
*) Fillet welded connections should be designed in accordance with 4.10.	

(5) Brace member connections subjected to combined bending and axial force should satisfy the following requirement:

$$\frac{N_{i,Ed}}{N_{i,Rd}} + \frac{M_{ip,i,Ed}}{M_{ip,i,Rd}} + \frac{M_{op,i,Ed}}{M_{op,i,Rd}} \leq 1,0 \qquad \text{... (7.4)}$$

where:

$M_{\mathrm{ip,i,Rd}}$ is the design in-plane moment resistance

$M_{\mathrm{ip,i,Ed}}$ is the design in-plane internal moment

$M_{\mathrm{op,i,Rd}}$ is the design out-of-plane moment resistance

$M_{\mathrm{op,i,Ed}}$ is the design out-of-plane internal moment

(6) The design internal moment $M_{\mathrm{i,Ed}}$ may be taken as the value at the point where the centreline of the brace member meets the face of the chord member.

(7) For unreinforced joints, the design in-plane moment resistance and design out-of-plane moment resistance $M_{\mathrm{i,Rd}}$ should be obtained from Table 7.13 or Table 7.14 as appropriate. For reinforced joints see 7.5.2.2.

(8) The special types of welded joints indicated in Table 7.15 and Table 7.16 should satisfy the appropriate design criteria specified for each type in that table.

7.5.2.2 Reinforced joints

(1) Various types of joint reinforcement may be used. The appropriate type depends upon the failure mode that, in the absence of reinforcement, governs the design resistance of the joint.

(2) Flange reinforcing plates may be used to increase the resistance of the joint to chord face failure, punching shear failure or brace failure with reduced effective width.

(3) A pair of side plates may be used to reinforce a joint against chord side wall failure or chord shear failure.

(4) In order to avoid partial overlapping of brace members in a K or N joint, the brace members may be welded to a vertical stiffener.

(5) Any combinations of these types of joint reinforcement may also be used.

(6) The grade of steel used for the reinforcement should not be lower than that of the chord member.

(7) The design resistances of reinforced joints should be determined using Table 7.17 and Table 7.18.

Table 7.14: Design resistance moments of welded joints between RHS brace members and RHS chords

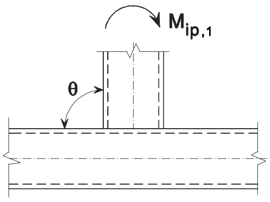
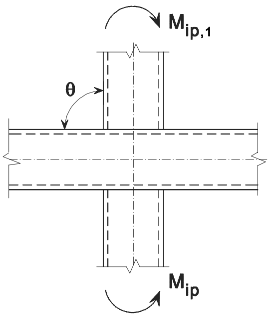
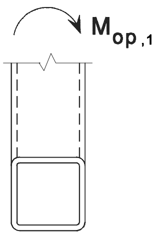
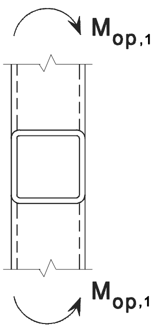
T and X joints	Design resistance
In-plane moments ($\theta = 90°$)	Chord face failure $\beta \leq 0{,}85$
	$M_{ip,1,Rd} = k_n f_{y0} t_0^2 h_1 \left(\frac{1}{2\eta} + \frac{2}{\sqrt{1-\beta}} + \frac{\eta}{1-\beta} \right) / \gamma_{M5}$
	Chord side wall crushing $0{,}85 \leq \beta \leq 1{,}0$
	$M_{ip,1,Rd} = 0{,}5 f_{yk} t_0 (h_1 + 5t_0)^2 / \gamma_{M5}$ $f_{yk} = f_{y0}$ for T joints $f_{yk} = 0{,}8 f_{y0}$ for X joints
	Brace failure $0{,}85 \leq \beta \leq 1{,}0$
	$M_{ip,1,Rd} = f_{y1} \left(W_{pl,1} - (1 - b_{eff}/b_1) b_1 h_1 t_1 \right) / \gamma_{M5}$
Out-of-plane moments ($\theta = 90°$)	Chord face failure $\beta \leq 0{,}85$
	$M_{op,1,Rd} = k_n f_{y0} t_0^2 \left(\frac{h_1(1+\beta)}{2(1-\beta)} + \sqrt{\frac{2b_0 b_1 (1+\beta)}{1-\beta}} \right) / \gamma_{M5}$
	Chord side wall crushing $0{,}85 \leq \beta \leq 1{,}0$
	$M_{op,1,Rd} = f_{yk} t_0 (b_0 - t_0)(h_1 + 5t_0) / \gamma_{M5}$ $f_{yk} = f_{y0}$ for T joints $f_{yk} = 0{,}8 f_{y0}$ for X joints
	Chord distortional failure (T joints only) *)
	$M_{op,1,Rd} = 2 f_{y0} t_0 \left(h_1 t_0 + \sqrt{b_0 h_0 t_0 (b_0 + h_0)} \right) / \gamma_{M5}$
	Brace failure $0{,}85 \leq \beta \leq 1{,}0$
	$M_{op,1,Rd} = f_{y1} \left(W_{pl,1} - 0{,}5(1 - b_{eff}/b_1)^2 b_1^2 t_1 \right) / \gamma_{M5}$
Parameters b_{eff} and k_n	
$b_{eff} = \frac{10}{b_0/t_0} \frac{f_{y0} t_0}{f_{y1} t_1} b_1$ but $b_{eff} \leq b_1$	For $n > 0$ (compression): $k_n = 1{,}3 - \frac{0{,}4n}{\beta}$ but $k_n \leq 1{,}0$ For $n \leq 0$ (tension): $k_n = 1{,}0$
*) This criterion does not apply where chord distortional failure is prevented by other means.	

Table 7.15: Design criteria for special types of welded joints between RHS brace members and RHS chords

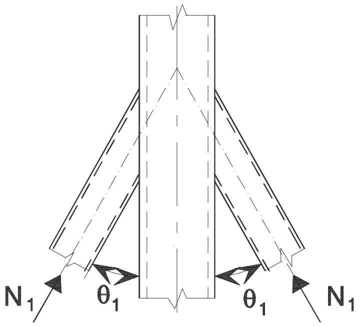
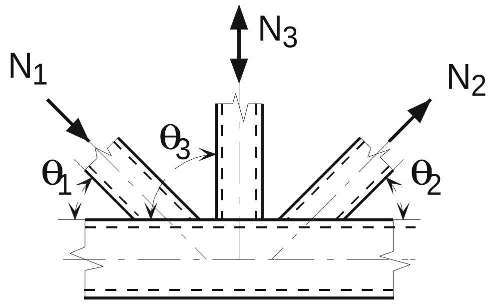
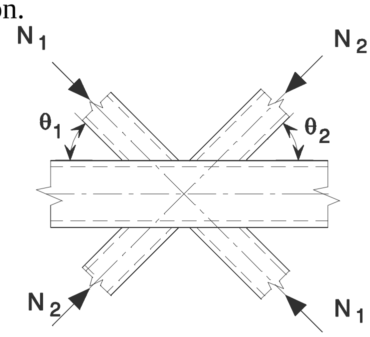
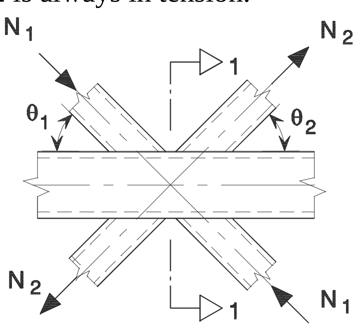
Type of joint	Design criteria
The members may be in either tension or compression and shall act as in the same direction for both members. 	$N_{1,\mathrm{Ed}} \le N_{1,\mathrm{Rd}}$ where $N_{1,\mathrm{Rd}}$ is the value of $N_{1,\mathrm{Rd}}$ for an X joint from Table 7.11.
The member 1 is always in compression and member 2 is always in tension. 	$N_{1,\mathrm{Ed}} \sin\theta_1 + N_{3,\mathrm{Ed}} \sin\theta_3 \le N_{1,\mathrm{Rd}} \sin\theta_1$ $N_{2,\mathrm{Ed}} \sin\theta_2 \le N_{1,\mathrm{Rd}} \sin\theta_1$ where $N_{1,\mathrm{Rd}}$ is the value of $N_{1,\mathrm{Rd}}$ for a K joint from Table 7.12, but with $\dfrac{b_1 + b_2 + h_1 + h_2}{4b_0}$ replaced by: $\dfrac{b_1 + b_2 + b_3 + h_1 + h_2 + h_3}{6b_0}$
All bracing members shall be either compression or tension. 	$N_{1,\mathrm{Ed}} \sin\theta_1 + N_{2,\mathrm{Ed}} \sin\theta_2 \le N_{\mathrm{x,Rd}} \sin\theta_{\mathrm{x}}$ where $N_{\mathrm{x,Rd}}$ is the value of $N_{\mathrm{x,Rd}}$ for an X joint from Table 7.11, and $N_{\mathrm{x,Rd}} \sin\theta_{\mathrm{x}}$ is the larger of: $\lvert N_{1,\mathrm{Rd}} \sin\theta_1 \rvert$ and $\lvert N_{2,\mathrm{Rd}} \sin\theta_2 \rvert$
Member 1 is always in compression and member 2 is always in tension. 	$N_{\mathrm{i,Ed}} \le N_{\mathrm{i,Rd}}$ where $N_{\mathrm{i,Rd}}$ is the value of $N_{\mathrm{i,Rd}}$ for a K joint from Table 7.12, provided that, in a gap-type joint, at section 1-1 the chord satisfies: $\left[\dfrac{N_{0,Ed}}{N_{0,p(,Rd}}\right]^2 + \left[\dfrac{V_{0,Ed}}{V_{0,p(,Rd}}\right]^2 \le 1{,}0$

Table 7.16: Design criteria for welded knee joints and cranked-chord joints in RHS members

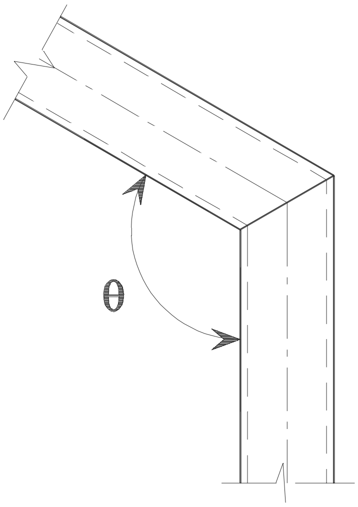
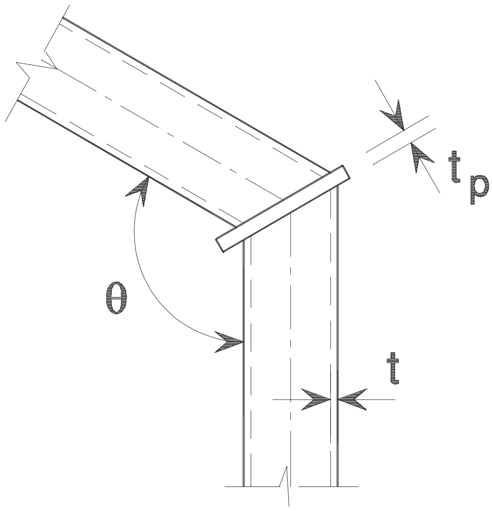
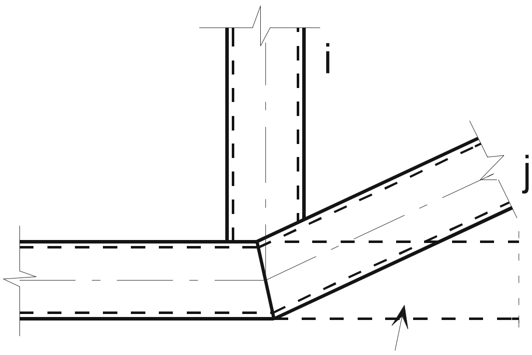
Type of joint	Criteria
Welded knee joints	
	The cross-section should be Class 1 for pure bending, see EN 1993-1-1. $N_{Ed} \le 0{,}2 N_{p\ell,Rd}$ and $\dfrac{N_{Ed}}{N_{p\ell,Rd}} + \dfrac{M_{Ed}}{M_{p\ell,Rd}} \le \kappa$ If $\theta \le 90°$: $\kappa = \dfrac{3\sqrt{b_0 / h_0}}{[b_0 / t_0]^{0,8}} + \dfrac{1}{1 + 2b_0 / h_0}$ If $90° < \theta \le 180°$: $\kappa = 1 - \left(\sqrt{2}\cos(\theta / 2)\right)(1 - \kappa_{90})$ where κ_{90} is the value of κ for $\theta = 90°$.
	$t_p \ge 1{,}5t$ and $\ge$ 10 mm $\dfrac{N_{Ed}}{N_{p\ell,Rd}} + \dfrac{M_{Ed}}{M_{p\ell,Rd}} \le 1{,}0$
Cranked-chord	
 Imaginary extension of chord	$N_{i,Ed} \le N_{i,Rd}$ where $N_{i,Rd}$ is the value of $N_{i,Rd}$ for a K or N overlap joint from Table 7.12.

Table 7.17: Design resistances of reinforced welded T, Y and X joints between RHS or CHS brace members and RHS chords

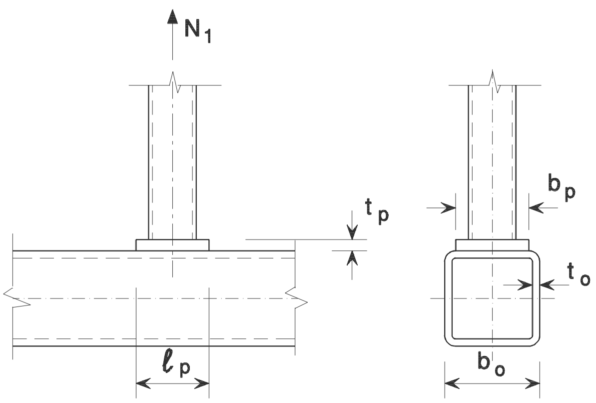
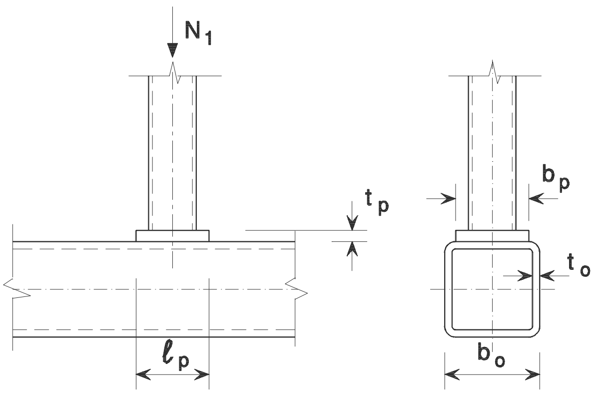
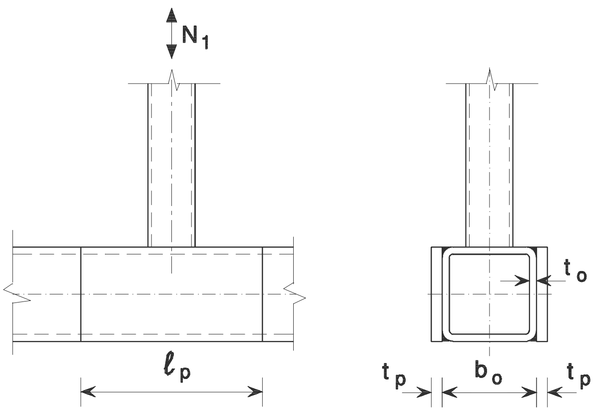
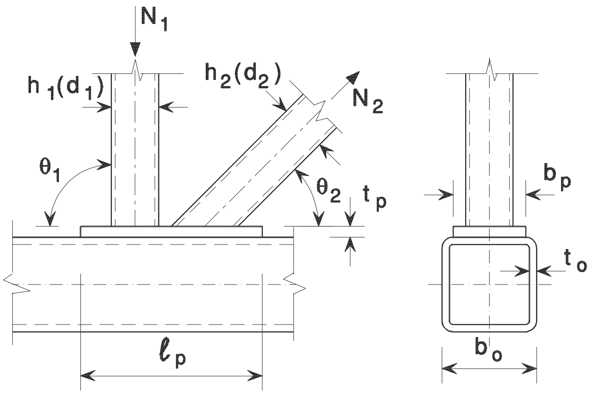
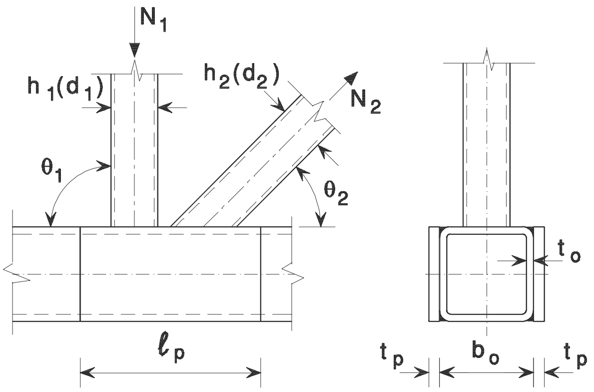
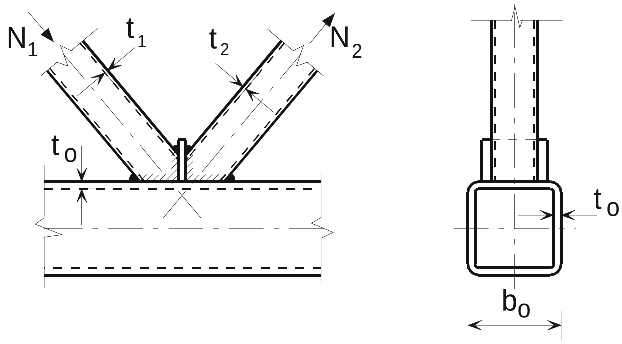
Type of joint	Design resistance [i = 1]
Reinforced with flange plates to avoid chord face failure, brace failure or punching shear.	
Tension loading	$\beta_p \le 0{,}85$
	$\ell_p \ge \dfrac{h_i}{\sin\theta_i} + \sqrt{b_p(b_p - b_i)}$ $\ge 1{,}5h_i / \sin\theta_i$ and $b_p \ge b_0 - 2t_0$ $N_{i,Rd} = \dfrac{f_{yp} t_p^2}{(1 - b_i / b_p)\sin\theta_i} x$ $x\left(\dfrac{2h_i / b_p}{\sin\theta_i} + 4\sqrt{1 - b_i / b_p}\right) / \gamma_{M5}$
Compression loading	$\beta_p \le 0{,}85$
	$\ell_p \ge \dfrac{h_i}{\sin\theta_i} + \sqrt{b_p(b_p - b_i)}$ $\ge 1{,}5h_i / \sin\theta_i$ and $b_p \ge b_0 - 2t_0$ Take $N_{i,Rd}$ as the value of $N_{i,Rd}$ for a T, X or Y joint from Table 7.11, but with k_n = 1,0 and t_0 replaced by t_p for chord face failure, brace failure and punching shear only.
Reinforced with side plates to avoid chord side wall buckling or chord side wall shear.	
	$\ell_p \ge 1{,}5h_i / \sin\theta_i$ Take $N_{i,Rd}$ as the value of $N_{i,Rd}$ for a T, X or Y joint from Table 7.11, but with t_0 replaced by (t_0 + t_p) for chord side wall buckling failure and chord side wall shear failure only.

Table 7.18: Design resistances of reinforced welded K and N joints between RHS or CHS brace members and RHS chords

<table>
<tr><th>Type of joint</th><th>Design resistance [i = 1 or 2]</th></tr>
<tr><td colspan="2">Reinforced with flange plates to avoid chord face failure, brace failure or punching shear.</td></tr>
<tr><td>
</td><td>$$\ell_p \geq 1{,}5\left(\frac{h_1}{\sin\theta_1} + g + \frac{h_2}{\sin\theta_2}\right)$$

$b_p \geq b_0 - 2t_0$
$t_p \geq 2t_1$ and $2t_2$

Take $N_{i,Rd}$ as the value of $N_{i,Rd}$ for a K or N joint from Table 7.12, but with t_0 replaced by t_p for chord face failure, brace failure and punching shear only.</td></tr>
<tr><td colspan="2">Reinforced with a pair of side plates to avoid chord shear failure.</td></tr>
<tr><td>
</td><td>$$\ell_p \geq 1{,}5\left(\frac{h_1}{\sin\theta_1} + g + \frac{h_2}{\sin\theta_2}\right)$$

Take $N_{i,Rd}$ as the value of $N_{i,Rd}$ for a K or N joint from Table 7.12, but with t_0 replaced by $(t_0 + t_p)$ for chord shear failure only.</td></tr>
<tr><td colspan="2">Reinforced by a division plate between the brace members because of insufficient overlap.</td></tr>
<tr><td>
</td><td>$t_p \geq 2t_1$ and $2t_2$

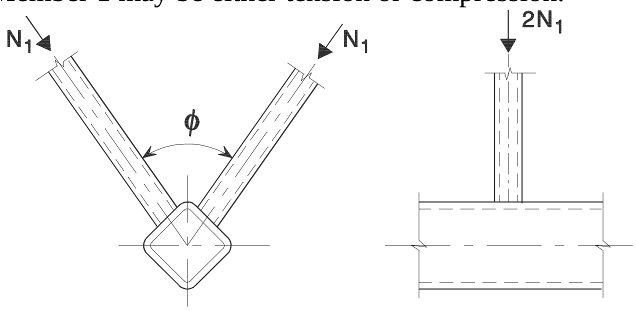
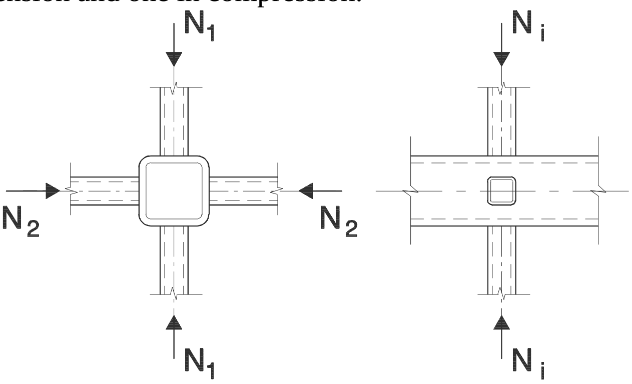
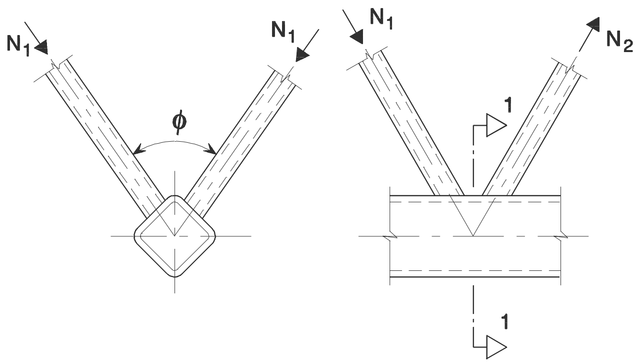
Take $N_{i,Rd}$ as the value of $N_{i,Rd}$ for a K or N overlap joint from Table 7.12 with $\lambda_{ov} < 80\%$, but with b_j, t_j and f_{yj} replaced by b_p, t_p and f_{yp} in the expression for $b_{e,ov}$ given in Table 7.10.</td></tr>
</table>

7.5.3 Multiplanar joints

(1) In each relevant plane of a multiplanar joint, the design criteria given in 7.5.2 should be satisfied using the reduced design resistances obtained from 7.5.3(2).

(2) The design resistances for each relevant plane of a multiplanar joint should be determined by applying the appropriate reduction factor μ given in Table 7.19 to the resistance of the corresponding uniplanar joint calculated according to 7.5.2 with the appropriate chord load in the multiplanar situation.

Table 7.19: Reduction factors for multiplanar joints

Type of joint	Reduction factor μ
TT joint	$60° \leq \varphi \leq 90°$
Member 1 may be either tension or compression. 	$\mu = 0{,}9$
XX joint	
Members 1 and 2 can be either in compression or tension. $N_{2,\mathrm{Ed}}/N_{1,\mathrm{Ed}}$ is negative if one member is in tension and one in compression. 	$\mu = 0{,}9(1 + 0{,}33 N_{2,Ed} / N_{1,Ed})$ taking account of the sign of $N_{1,\mathrm{Ed}}$ and $N_{2,\mathrm{Ed}}$ where $\lvert N_{2,\mathrm{Ed}} \rvert \leq \lvert N_{1,\mathrm{Ed}} \rvert$
KK joint	$60° \leq \varphi \leq 90°$
	$\mu = 0{,}9$ provided that, in a gap-type joint, at section 1-1 the chord satisfies: $\left[\frac{N_{0,Ed}}{N_{p(,0,Rd}}\right]^2 + \left[\frac{V_{0,Ed}}{V_{p(,0,Rd}}\right]^2 \leq 1{,}0$

7.6 Welded joints between CHS or RHS brace members and I or H section chords

(1) Provided that the geometry of the joints is within the range of validity given in Table 7.20, the design resistances of the joints should be determined using the expressions given in Table 7.21 or Table 7.22 as appropriate.

Table 7.20: Range of validity for welded joints between CHS or RHS brace members and I or H section chord members

<table>
<tr><th rowspan="3">Type of joint</th><th colspan="6">Joint parameter [i = 1 or 2, j = overlapped brace]</th></tr>
<tr><th rowspan="2">d_w/t_w</th><th colspan="2">b_i/t_i and h_i/t_i or d_i/t_i</th><th rowspan="2">h_i/b_i</th><th rowspan="2">b_0/t_f</th><th rowspan="2">b_i/b_j</th></tr>
<tr><th>Compression</th><th>Tension</th></tr>
<tr><td>X</td><td>Class 1
and
$d_w \leq 400$ mm</td><td rowspan="4">Class 1
and
$\frac{h_i}{t_i} \leq 35$
$\frac{b_i}{t_i} \leq 35$
$\frac{d_i}{t_i} \leq 50$</td><td rowspan="4">$\frac{h_i}{t_i} \leq 35$
$\frac{b_i}{t_i} \leq 35$
$\frac{d_i}{t_i} \leq 50$</td><td>$\geq 0,5$
but
$\leq 2,0$</td><td rowspan="4">Class 2</td><td>–</td></tr>
<tr><td>T or Y</td><td rowspan="3">Class 2
and
$d_w \leq 400$ mm</td><td rowspan="2">1,0</td><td rowspan="2">–</td></tr>
<tr><td>K gap
N gap</td></tr>
<tr><td>K overlap
N overlap</td><td>$\geq 0,5$
but
$\leq 2,0$</td><td>$\geq 0,75$</td></tr>
</table>

(2) For joints within the range of validity given in Table 7.20, only the design criteria covered in the appropriate table need be considered. The design resistance of a connection should be taken as the minimum value for all applicable criteria.

(3) For joints outside the range of validity given in Table 7.20, all the criteria given in 7.2.2 should be considered. In addition, the secondary moments in the joints caused by their rotational stiffness should be taken into account.

(4) In brace member connections subjected only to axial forces, the design axial force $N_{i,Ed}$ should not exceed the design axial resistance of the welded joint $N_{i,Rd}$, determined from Table 7.21.

(5) Brace member connections subject to combined bending and axial force should satisfy:

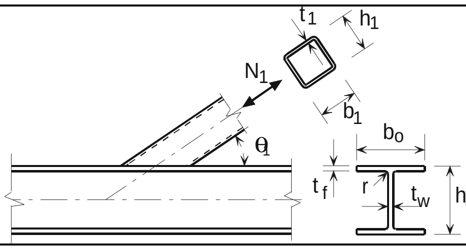
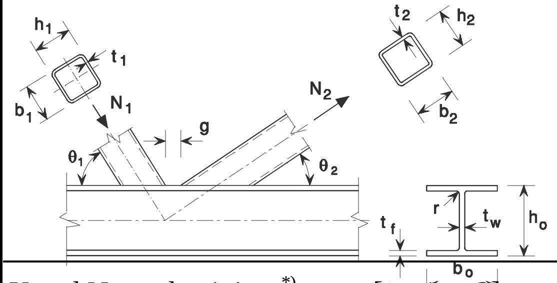
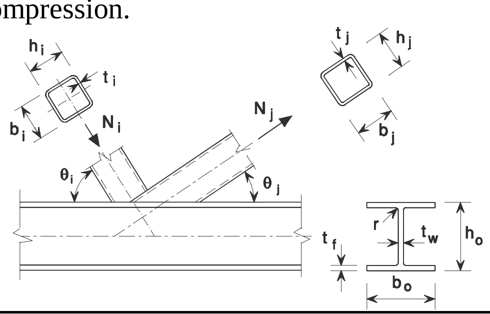
$$\frac{N_{i,Ed}}{N_{i,Rd}} + \frac{M_{ip,i,Ed}}{M_{ip,i,Rd}} \leq 1,0 \qquad \text{... (7.5)}$$

where:

$M_{ip,i,Rd}$ is the design in-plane moment resistance;

$M_{ip,i,Ed}$ is the design in-plane internal moment.

Table 7.21: Design resistances of welded joints between RHS or CHS brace members and I or H section chords

Type of joint	Design resistance [i = 1 or 2, j = overlapped brace]	
T, Y and X joints	Chord web yielding	
	$N_{1,\mathrm{Rd}} = \dfrac{f_{y0} t_w b_w}{\sin\theta_1} / \gamma_{M5}$	
	Brace failure	
	$N_{1,\mathrm{Rd}} = 2 f_{y1} t_1 p_{eff} / \gamma_{M5}$	
K and N gap joints [i = 1 or 2]	Chord web stability	Brace failure need not be checked if: $g/t_f \le 20 - 28\beta$; $\beta \le 1,0 - 0,03\gamma$ where $\gamma = b_0/2t_f$ and for CHS: $0,75 \le d_1 / d_2 \le 1,33$ or for RHS: $0.75 \le b / b \le 1.33$
	$N_{i,\mathrm{Rd}} = \dfrac{f_{y0} t_w b_w}{6} / \gamma_{M5}$	
	Brace failure	
	$N_{i,\mathrm{Rd}} = 2 f_{yi} t_i p_{eff} / \gamma_{M5}$	
	Chord shear	
	$N_{i,\mathrm{Rd}} = \dfrac{f_{y0} A_v}{\sqrt{3}\sin\theta_i} / \gamma_{M5}$	
	$N_{0,\mathrm{Rd}} = \left[(A_0 - A_v) f_{y0} + A_v f_{y0} \sqrt{1 - (V_{Ed} / V_{p1,Rd})^2} \right] / \gamma_{M5}$	
K and N overlap joints *) [i = 1 or 2]	Brace failure $25\% \le \lambda_{ov} < 50\%$	
Members i and j may be in either tension or compression. 	$N_{i,\mathrm{Rd}} = f_{yi} t_i \left(p_{eff} + b_{e,ov} + (h_i - 2t_i)\lambda_{ov} / 50 \right) / \gamma_{M5}$	
	Brace failure $50\% \le \lambda_{ov} < 80\%$	
	$N_{i,\mathrm{Rd}} = f_{yi} t_i \left(p_{eff} + b_{e,ov} + h_i - 2t_i \right) / \gamma_{M5}$	
	Brace failure $\lambda_{ov} \ge 80\%$	
	$N_{i,\mathrm{Rd}} = f_{yi} t_i \left(b_i + b_{e,ov} + 2h_i - 4t_i \right) / \gamma_{M5}$	
$A_v = A_0 - (2 - \alpha)\, b_0\, t_f + (t_w + 2r)\, t_f$ For RHS brace: $\alpha = \sqrt{\dfrac{1}{(1 + 4g^2 / 3t_f^{\,2})}}$ For CHS brace: $\alpha = 0$	$p_{eff} = t_w + 2r + 7 t_f f_{y0} / f_{yi}$ but $p_{eff} \le b_i + h_i - 2t_i$ for T, Y, X joints and K and N gap joints and $b_{eff} \le b_i + h_i - 2t_i$ for K and N overlap joints. $b_{e,ov} = \dfrac{10}{b_j / t_j} \dfrac{f_{yj} t_j}{f_{yi} t_i} b_i$ but $b_{e,ov} \le b_i$	$b_w = \dfrac{h_i}{\sin\theta_i} + 5(t_f + r)$ but $b_w \le 2t_i + 10\,(t_f + r)$

For CHS braces multiply the above resistances for brace failure by $\pi/4$ and replace both b_1 and h_1 by d_1 and both b_2 and h_2 by d_2.

*) Only the overlapping brace member i need be checked. The efficiency (i.e. the design resistance of the joint divided by the design plastic resistance of the brace member) of the overlapped brace member j should be taken as equal to that of the overlapping brace member.

(6) The design internal moment $M_{i,Ed}$ may be taken as the value at the point where the centreline of the brace member meets the face of the chord member.

(7) The design in-plane moment resistance $M_{ip,1,Rd}$ should be obtained from Table 7.22.

(8) If stiffeners in the chord (see Figure 7.7) are used, then the design bracing failure resistance $N_{i,Rd}$ for T-, X-, Y-, K-gap and N-gap joints (Table 7.22) is determined as follows:

$$N_{i,Rd} = 2\, f_{yi}\, t_i\, (b_{eff} + b_{eff,s}) \,/\, \gamma_{M5} \qquad \text{... (7.6)}$$

where:

$b_{eff} = t_w + 2r + 7\, t_f\, f_{y0} / f_{yi}$ but $\leq b_i + h_i - 2t_i$

$b_{eff,s} = t_s + 2a + 7\, t_f\, f_{y0} / f_{yi}$ but $\leq b_i + h_i - 2t_i$

$b_{eff} + b_{eff,s} \leq b_i + h_i - 2t_i$

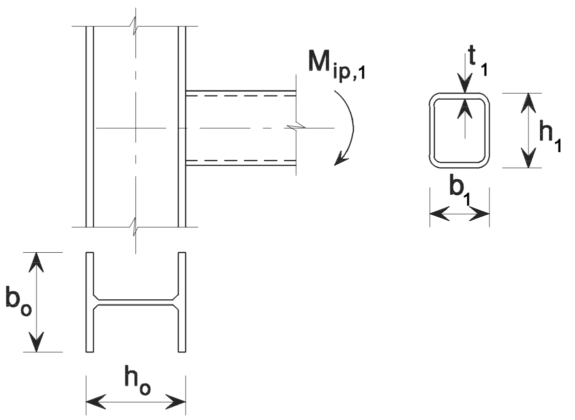
where:

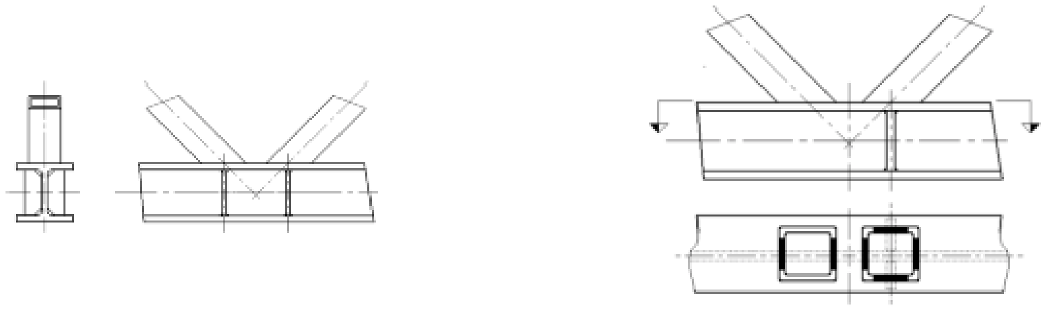
a is stiffener weld throat thickness, '$2a$' becomes 'a' if single sided fillet welds are used;

s refers to the stiffener.

(9) The stiffeners should be at least as thick as the I-section web.

Table 7.22: Design moment resistances of welded joints between rectangular hollow section brace members and I or H section chords

Type of joint	Design resistance [i = 1 or 2, j = overlapped brace]
T and Y joints	Chord web yielding
	$M_{ip,1,Rd} = 0{,}5 f_{y0} t_w b_w h_1 / \gamma_{M5}$
	Brace failure
	$M_{ip,1,Rd} = f_{y1} t_1 b_{eff} (h_1 - t_1) / \gamma_{M5}$
Parameters b_{eff} and b_w	
$b_{eff} = t_w + 2r + 7 t_f f_{y0} / f_{y1}$ but $b_{eff} \leq b_i$	$b_w = \dfrac{h_1}{\sin\theta_1} + 5(t_f + r)$ but $b_w \leq 2t_1 + 10(t_f + r)$



Bracing effective perimeter, without (left) and with (right) stiffeners

Figure 7.7: Stiffeners for I-section chords

7.7 Welded joints between CHS or RHS brace members and channel section chord members

(1) Provided that the geometry of the joints is within the range of validity given in Table 7.23, the design resistances of welded joints between hollow section brace members and channel section chord members may be determined using Table 7.24.

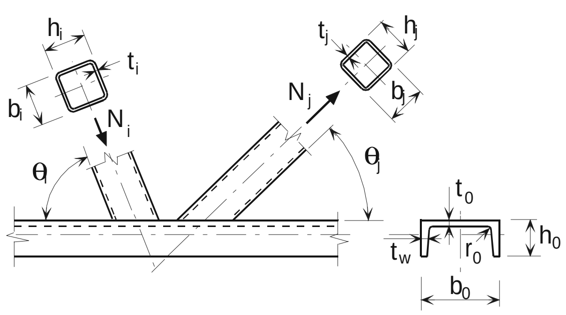
(2) The secondary moments in the joints caused by their bending stiffness should be taken into account.

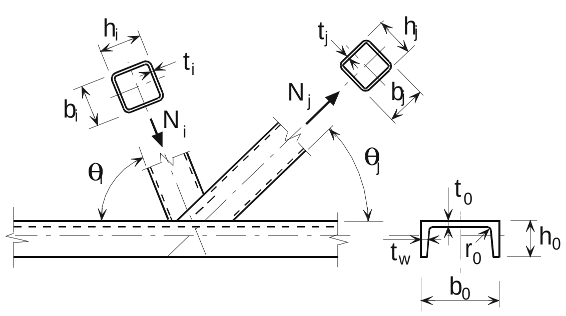
(3) In a gap type joint, the design axial resistance of the chord cross-section $N_{0,Rd}$ should be determined allowing for the shear force transferred between the brace members by the chord, neglecting the associated secondary moment. Verification should be made according to EN 1993-1-1.

Table 7.23: Range of validity for welded joints between CHS or RHS brace members and channel section chord

<table>
<tr><td rowspan="3">Type of joint</td><td colspan="6">Joint parameter [i = 1 or 2, j = overlapped brace]</td></tr>
<tr><td rowspan="2">b_i/b_0</td><td colspan="2">b_i/t_i and h_i/t_i or d_i/t_i</td><td rowspan="2">h_i/b_i</td><td rowspan="2">b_0/t_0</td><td rowspan="2">Gap or overlap
b_i/b_j</td></tr>
<tr><td>Compression</td><td>Tension</td></tr>
<tr><td>K gap
N gap</td><td>$\geq 0,4$
and
$b_0 \leq 400$ mm</td><td rowspan="2">Class 1
and
$\frac{h_i}{t_i} \leq 35$
$\frac{b_i}{t_i} \leq 35$
$\frac{d_i}{t_i} \leq 50$</td><td rowspan="2">$\frac{h_i}{t_i} \leq 35$
$\frac{b_i}{t_i} \leq 35$
$\frac{d_i}{t_i} \leq 50$</td><td rowspan="2">$\geq 0,5$
but
$\leq 2,0$</td><td rowspan="2">Class 2</td><td>$0,5(1-\beta^*) \leq g/b_0^* \leq 1,5(1-\beta^*)$ 1)
and
$g \geq t_1 + t_2$</td></tr>
<tr><td>K overlap
N overlap</td><td>$\geq 0,25$
and
$b_0 \leq 400$ mm</td><td>$25\% \leq \lambda_{ov} < 100\%$
$b_i/b_j \geq 0,75$</td></tr>
<tr><td colspan="7">$\beta^* = b_1/b_0^*$
$b_0^* = b_0 - 2\,(t_w + r_0)$
1) This condition only apply when $\beta \leq 0,85$.</td></tr>
</table>

Table 7.24: Design resistance of welded joints between RHS or CHS brace members and channel section chords

<table>
<tr><td>Type of joint</td><td>Design resistance [i = 1 or 2, j = overlapped brace]</td></tr>
<tr><td>K and N gap joints</td><td>Brace failure</td></tr>
<tr><td rowspan="4">
</td><td>$N_{i,Rd} = f_{yi} t_i \left(b_i + b_{eff} + 2h_i - 4t_i\right) / \gamma_{M5}$</td></tr>
<tr><td>Chord failure</td></tr>
<tr><td>$N_{i,Rd} = \dfrac{f_{y0} A_v}{\sqrt{3}\sin\theta_i} / \gamma_{M5}$

$N_{0,Rd} = \left[\left(A_0 - A_v\right) f_{y0} + A_v f_{y0} \sqrt{1 - \left(V_{Ed} / V_{pl,Rd}\right)^2}\right] / \gamma_{M5}$</td></tr>
<tr><td></td></tr>
<tr><td>K and N overlap joints *)</td><td>Brace failure 25% ≤ λ_{ov} < 50%</td></tr>
<tr><td rowspan="5">
</td><td>$N_{i,Rd} = f_{yi} t_i \left(b_{eff} + b_{e,ov} + \left(2h_i - 4t_i\right)\lambda_{ov} / 50\right) / \gamma_{M5}$</td></tr>
<tr><td>Brace failure 50% ≤ λ_{ov} < 80%</td></tr>
<tr><td>$N_{i,Rd} = f_{yi} t_i \left(b_{eff} + b_{e,ov} + 2h_i - 4t_i\right) / \gamma_{M5}$</td></tr>
<tr><td>Brace failure $\lambda_{ov} \geq$ 80%</td></tr>
<tr><td>$N_{i,Rd} = f_{yi} t_i \left(b_i + b_{e,ov} + 2h_i - 4t_i\right) / \gamma_{M5}$</td></tr>
<tr><td>$A_v = A_0 - (1 - \alpha)\, b_0^* \, t_0$
$b_0^* = b_0 - 2\,(t_w + r_0)$

For RHS: $\alpha = \sqrt{\dfrac{1}{\left(1 + 4g^2 / 3t_f^{\,2}\right)}}$

For CHS: $\alpha = 0$

$V_{pl,Rd} = \dfrac{f_{y0} A_v}{\sqrt{3}} / \gamma_{M5}$
$V_{Ed} = (N_{i,Ed} \sin\theta_i)_{max}$</td><td>$b_{eff} = \dfrac{10}{b_0^* / t_0} \dfrac{f_{y0} t_0}{f_{yi} t_i} b_i$ but $b_{eff} \leq b_i$

$b_{e,ov} = \dfrac{10}{b_j / t_j} \dfrac{f_{yj} t_j}{f_{yi} t_i} b_i$ but $b_{e,ov} \leq b_i$</td></tr>
<tr><td colspan="2">For CHS braces except the chord failure, multiply the above resistances by $\pi/4$ and replace both b_1 and h_1 by d_1 as well as b_2 and h_2 by d_2.</td></tr>
<tr><td colspan="2">*) Only the overlapping brace member i needs to be checked. The efficiency (i.e. the design resistance of the joint divided by the design plastic resistance of the brace member) of the overlapped brace member j should be taken as equal to that of the overlapping brace member.</td></tr>
</table>

EUROPEAN STANDARD
NORME EUROPÉENNE
EUROPÄISCHE NORM

prEN 1993-1-9 : 2003

17 April 2003

UDC

Descriptors:

English version

Eurocode 3 : Design of steel structures

Part 1.9 : Fatigue

Calcul des structures en acier	Bemessung und Konstruktion von Stahlbauten
Partie 1.9 :	Teil 1.9 :
Fatigue	Ermüdung

Stage 49 draft

CEN

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

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

Ref. No. EN 1993-1.9 : 2003. E

Contents

National annex for EN 1993-1-9

This standard gives alternative procedures, values and recommendations with notes indicating where national choices may have to be made. The National Standard implementing EN 1993-1-9 should have a National Annex containing all Nationally Determined Parameters for the design of steel structures to be constructed in the relevant country.

National choice is allowed in EN 1993-1-9 through:

- 1.1(2)
- 2(2)
- 2(4)
- 3(2)
- 3(7)
- 5(2)
- 6.1(1)
- 6.2(2)
- 7.1(3)
- 7.1(5)
- 8(4)

1 General

1.1 Scope

(1) EN 1993-1-9 gives methods for the assessment of fatigue resistance of members, connections and joints subjected to fatigue loading.

(2) These methods are derived from fatigue tests with large scale specimens, that include effects of geometrical and structural imperfections from material production and execution (e.g. the effects of tolerances and residual stresses from welding).

> **NOTE 1** For tolerances see EN 1090. The choice of the execution standard may be given in the National Annex, until such time as EN 1090 is published.

> **NOTE 2** The National Annex may give supplementary information on inspection requirements during fabrication.

(3) The rules are applicable to structures where execution conforms with EN 1090.

> **NOTE** Where appropriate, supplementary requirements are indicated in the detail category tables.

(4) The assessment methods given in this part are applicable to all grades of structural steels, stainless steels and unprotected weathering steels except where noted otherwise in the detail category tables. This part only applies to materials which conform to the toughness requirements of EN 1993-1-10.

(5) Fatigue assessment methods other than the $\Delta\sigma_R$-N methods as the notch strain method or fracture mechanics methods are not covered by this part.

(6) Post fabrication treatments to improve the fatigue strength other than stress relief are not covered in this part.

(7) The fatigue strengths given in this part apply to structures operating under normal atmospheric conditions and with sufficient corrosion protection and regular maintenance. The effect of seawater corrosion is not covered. Microstructural damage from high temperature (> 150 °C) is not covered.

1.2 Definitions

(1) For the purpose of this European Standard the following definitions apply.

1.2.1 General

1.2.1.1
fatigue
The process of initiation and propagation of cracks through a structural part due to action of fluctuating stress.

1.2.1.2
nominal stress
A stress in the parent material or in a weld adjacent to a potential crack location calculated in accordance with elastic theory excluding all stress concentration effects.

> **NOTE** The nominal stress as specified in this part can be a direct stress, a shear stress, a principal stress or an equivalent stress.

1.2.1.3
modified nominal stress
A nominal stress multiplied by an appropriate stress concentration factor k_f, to allow for a geometric discontinuity that has not been taken into account in the classification of a particular constructional detail.

1.2.1.4
geometric stress
hot spot stress
The maximum principal stress in the parent material adjacent to the weld toe, taking into account stress concentration effects due to the overall geometry of a particular constructional detail.

> **NOTE** Local stress concentration effects e.g. from the weld profile shape (which is already included in the detail categories in Annex B) need not be considered.

1.2.1.5
residual stress
Residual stress is a permanent state of stress in a structure that is in static equilibrium and is independent of any applied action. Residual stresses can arise from rolling stresses, cutting processes, welding shrinkage or lack of fit between members or from any loading event that causes yielding of part of the structure.

1.2.2 Fatigue loading parameters

1.2.2.1
loading event
A defined loading sequence applied to the structure and giving rise to a stress history, which is normally repeated a defined number of times in the life of the structure.

1.2.2.2
stress history
A record or a calculation of the stress variation at a particular point in a structure during a loading event.

1.2.2.3
rainflow method
Particular cycle counting method of producing a stress-range spectrum from a given stress history.

1.2.2.4
reservoir method
Particular cycle counting method of producing a stress-range spectrum from a given stress history.

> **NOTE** For the mathematical determination see annex A.

1.2.2.5
stress range
The algebraic difference between the two extremes of a particular stress cycle derived from a stress history.

1.2.2.6
stress-range spectrum
Histogram of the number of occurrences for all stress ranges of different magnitudes recorded or calculated for a particular loading event.

1.2.2.7
design spectrum
The total of all stress-range spectra in the design life of a structure relevant to the fatigue assessment.

1.2.2.8
design life
The reference period of time for which a structure is required to perform safely with an acceptable probability that failure by fatigue cracking will not occur.

1.2.2.9
fatigue life
The predicted period of time to cause fatigue failure under the application of the design spectrum.

1.2.2.10
Miner's summation
A linear cumulative damage calculation based on the Palmgren-Miner rule.

1.2.2.11
equivalent constant amplitude stress range
The constant-amplitude stress range that would result in the same fatigue life as for the design spectrum, when the comparison is based on a Miner's summation.

NOTE For the mathematical determination see Annex A.

1.2.2.12
fatigue loading
A set of action parameters based on typical loading events described by the positions of loads, their magnitudes, frequencies of occurrence, sequence and relative phasing.

NOTE 1 The fatigue actions in EN 1991 are upper bound values based on evaluations of measurements of loading effects according to Annex A.

NOTE 2 The action parameters as given in EN 1991 are either

- Q_{max}, n_{max}, standardised spectrum or
- $Q_{E,n_{max}}$ related to n_{max} or
- $Q_{E,2}$ corresponding to $n = 2\times10^6$ cycles.

Dynamic effects are included in these parameters unless otherwise stated.

1.2.2.13
equivalent constant amplitude fatigue loading
Simplified constant amplitude loading causing the same fatigue damage effects as a series of actual variable amplitude loading events

1.2.3 Fatigue strength

1.2.3.1
fatigue strength curve
The quantitative relationship between the stress range and number of stress cycles to fatigue failure, used for the fatigue assessment of a particular category of structural detail.

NOTE The fatigue strengths given in this part are lower bound values based on the evaluation of fatigue tests with large scale test specimens in accordance with EN 1990 – Annex D.

1.2.3.2
detail category
The numerical designation given to a particular detail for a given direction of stress fluctuation, in order to indicate which fatigue strength curve is applicable for the fatigue assessment (The detail category number indicates the reference fatigue strength $\Delta\sigma_C$ in N/mm²).

1.2.3.3
constant amplitude fatigue limit
The limiting direct or shear stress range value below which no fatigue damage will occur in tests under constant amplitude stress conditions. Under variable amplitude conditions all stress ranges have to be below this limit for no fatigue damage to occur.

1.2.3.4
cut-off limit
Limit below which stress ranges of the design spectrum do not contribute to the calculated cumulative damage.

1.2.3.5
endurance
The life to failure expressed in cycles, under the action of a constant amplitude stress history.

1.2.3.6
reference fatigue strength
The constant amplitude stress range $\Delta\sigma_C$, for a particular detail category for an endurance $N = 2\times10^6$ cycles

1.3 Symbols

$\Delta\sigma$	stress range (direct stress)
$\Delta\tau$	stress range (shear stress)
$\Delta\sigma_E$, $\Delta\tau_E$	equivalent constant amplitude stress range related to n_{max}
$\Delta\sigma_{E,2}$, $\Delta\tau_{E,2}$	equivalent constant amplitude stress range related to 2 million cycles
$\Delta\sigma_C$, $\Delta\tau_C$	reference value of the fatigue strength at N_C = 2 million cycles
$\Delta\sigma_D$, $\Delta\tau_D$	fatigue limit for constant amplitude stress ranges at the number of cycles N_D
$\Delta\sigma_L$, $\Delta\tau_L$	cut-off limit for stress ranges at the number of cycle N_L
$\Delta\sigma_{eq}$	equivalent stress range for connections in webs of orthotropic decks
$\Delta\sigma_{C,red}$	reduced reference value of the fatigue strength
γ_{Ff}	partial factor for equivalent constant amplitude stress ranges $\Delta\sigma_E$, $\Delta\tau_E$
γ_{Mf}	partial factor for fatigue strength $\Delta\sigma_C$, $\Delta\tau_C$
m	slope of fatigue strength curve
λ_i	damage equivalent factors
ψ_1	factor for frequent value of a variable action
Q_k	characteristic value of a single variable action
k_s	reduction factor for fatigue stress to account for size effects
k_1	magnification factor for nominal stress ranges to account for secondary bending moments in trusses
k_f	stress concentration factor

2 Basic requirements and methods

(1) Structural members shall be designed for fatigue such that there is an acceptable level of probability that their performance will be satisfactory throughout their design life.

> **NOTE** Structures designed using fatigue actions from EN 1991 and fatigue resistance according to this part are deemed to satisfy this requirement.

(2) Annex A may be used to determine a specific loading model, if

- no fatigue load model is available in EN 1991,
- a more realistic fatigue load model is required.

> **NOTE** Requirements for determining specific fatigue loading models may be specified in the National Annex.

(3) Fatigue tests may be carried out

- to determine the fatigue strength for details not included in this part,
- to determine the fatigue life of prototypes, for actual or for damage equivalent fatigue loads.

(4) In performing and evaluating fatigue tests EN 1990 shall be taken into account (see also 7.1).

> **NOTE** Requirements for determining fatigue strength from tests may be specified in the National Annex.

(5) The methods for the fatigue assessment given in this part follows the principle of design verification by comparing action effects and fatigue strengths; such a comparison is only possible when fatigue actions are determined with parameters of fatigue strengths contained in this standard.

(6) Fatigue actions are determined according to the requirements of the fatigue assessment. They are different from actions for ultimate limit state and serviceability limit state verifications.

> **NOTE** Any fatigue cracks that develop during service life do not necessarily mean the end of the service life. Cracks should be repaired with particular care for execution to avoid introducing more severe notch conditions.

3 Assessment methods

(1) Fatigue assessment shall be undertaken using either:

- damage tolerant method or
- safe life method.

(2) The damage tolerant method should provide an acceptable reliability that a structure will perform satisfactorily for its design life, provided that a prescribed inspection and maintenance regime for detecting and correcting fatigue damage is implemented throughout the design life of the structure.

> **NOTE 1** The damage tolerant method may be applied when in the event of fatigue damage occurring a load redistribution between components of structural elements can occur.

> **NOTE 2** The National Annex may give provisions for inspection programmes.

> **NOTE 3** Structures that are assessed to this part, the material of which is chosen according to EN 1993-1-10 and which are subjected to regular maintenance are deemed to be damage tolerant.

(3) The safe life method should provide an acceptable level of reliability that a structure will perform satisfactorily for its design life without the need for regular in-service inspection for fatigue damage. The safe life method should be applied in cases where local formation of cracks in one component could rapidly lead to failure of the structural element or structure.

(4) For the purpose of fatigue assessment using this part, an acceptable reliability level may be achieved by adjustment of the partial factor for fatigue strength γ_{Mf} taking into account the consequences of failure and the design assessment used.

(5) Fatigue strengths are determined by considering the structural detail together with its metallurgical and geometric notch effects. In the fatigue details presented in this part the probable site of crack initiation is also indicated.

(6) The assessment methods presented in this code use fatigue resistance in terms of fatigue strength curves for

- standard details applicable to nominal stresses
- reference weld configurations applicable to geometric stresses.

(7) The required reliability can be achieved as follows:

a) damage tolerant method

- selecting details, materials and stress levels so that in the event of the formation of cracks a low rate of crack propagation and a long critical crack length would result,
- provision of multiple load path
- provision of crack-arresting details,
- provision of readily inspectable details during regular inspections.

b) safe-life method

- selecting details and stress levels resulting in a fatigue life sufficient to achieve the β – values equal to those for ultimate limit state verifications at the end of the design service life.

NOTE The National Annex may give the choice of the assessment method, definitions of classes of consequences and numerical values for γ_{Mf}. Recommended values for γ_{Mf} are given in Table 3.1.

Table 3.1: Recommended values for partial factors for fatigue strength

Assessment method	Consequence of failure	
	Low consequence	High consequence
Damage tolerant	1,00	1,15
Safe life	1,15	1,35

4 Stresses from fatigue actions

(1) Modelling for nominal stresses shall take into account all action effects including distortional effects and should be based on a linear elastic analysis for members and connections

(2) For latticed girders made of hollow sections the modelling may be based on a simplified truss model with pinned connections. Provided that the stresses due to external loading applied to members between joints are taken into account the effects from secondary moments due to the stiffness of the connection can be allowed for by the use of k_1-factors (see Table 4.1 for circular sections, Table 4.2 for rectangular sections).

Table 4.1: k_1-factors for circular hollow sections under in-plane loading

Type of joint		Chords	Verticals	Diagonals
Gap joints	K type	1,5	1,0	1,3
	N type / KT type	1,5	1,8	1,4
Overlap joints	K type	1,5	1,0	1,2
	N type / KT type	1,5	1,65	1,25

Table 4.2: k_1-factors for rectangular hollow sections under in-plane loading

Type of joint		Chords	Verticals	Diagonals
Gap joints	K type	1,5	1,0	1,5
	N type / KT type	1,5	2,2	1,6
Overlap joints	K type	1,5	1,0	1,3
	N type / KT type	1,5	2,0	1,4

NOTE For the definition of joint types see EN 1993-1-8.

5 Calculation of stresses

(1) Stresses shall be calculated at the serviceability limit state.

(2) Class 4 cross sections are assessed for fatigue loads according to EN 1993-1-5

NOTE 1 For guidance see EN 1993-2 to EN 1993-6.

NOTE 2 The National Annex may give limitations for class 4 sections.

(3) Nominal stresses should be calculated at the site of potential fatigue initiation. Effects producing stress concentrations at details other than those included in Table 8.1 to Table 8.10 shall be accounted for by using a stress concentration factor (SCF) according to 6.3 to give a modified nominal stress.

(4) When using geometrical (hot spot) stress methods for details covered by Table B.1, the stresses shall be calculated as shown in 6.5.

(5) The relevant stresses for details in the parent material are

- nominal direct stresses σ
- nominal shear stresses τ

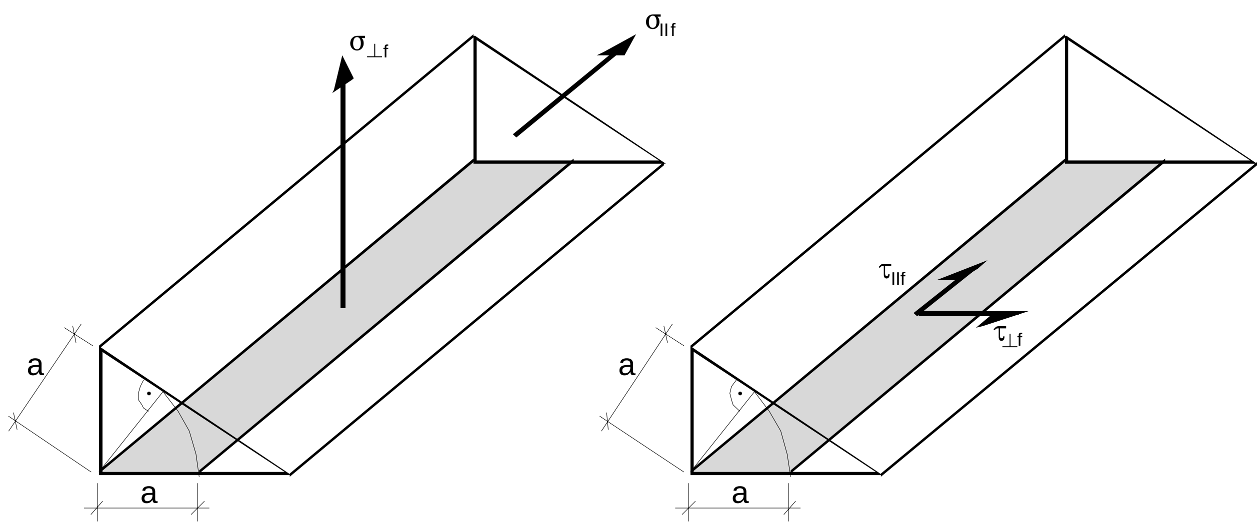
NOTE For effects of combined nominal stresses see 8(2).

(6) The relevant stresses in the welds are (see Figure 5.1)

- normal stresses σ_{wf} transverse to the axis of the weld: $\sigma_{wf} = \sqrt{\sigma_{\perp f}^2 + \tau_{\perp f}^2}$
- shear stresses τ_{wf} longitudinal to the axis of the weld: $\tau_{wf} = \tau_{\| f}$

for which two separate checks should be performed.

NOTE The above procedure differs from the procedure given for the verification of fillet welds for the ultimate limit state, given in EN 1993-1-8.



relevant stresses σ_f relevant stresses τ_f

Figure 5.1: Relevant stresses in the fillet welds

6 Calculation of stress ranges

6.1 General

(1) The fatigue assessment should be carried out using

- nominal stress ranges for details shown in Table 8.1 to Table 8.10,
- modified nominal stress ranges where abrupt changes of section occur close to the initiation site which are not included in Table 8.1 to Table 8.10 or
- geometric stress ranges where high stress gradients occur close to a weld toe in joints covered by Table B.1

NOTE The National Annex may give information on the use of the nominal stress ranges, modified nominal stress ranges or the geometric stress ranges. For detail categories for geometric stress ranges see Annex B.

(2) The design value of stress range to be used for the fatigue assessment should be the stress ranges $\gamma_{Ff}\ \Delta\sigma E_{,2}$ corresponding to $N_C = 2\times10^6$ cycles.

6.2 Design value of nominal stress range

(1) The design value of nominal stress ranges $\gamma_{Ff}\ \Delta\sigma_{E,2}$ and $\gamma_{Ff}\ \Delta\tau_{E,2}$ should be determined as follows:

$$\gamma_{Ff}\ \Delta\sigma_{E,2} = \lambda_1 \times \lambda_2 \times \lambda_i \times ... \times \lambda_n \times \Delta\sigma(\gamma_{Ff}\ Q_k)$$

(6.1)

$$\gamma_{Ff}\ \Delta\tau_{E,2} = \lambda_1 \times \lambda_2 \times \lambda_i \times ... \times \lambda_n \times \Delta\tau(\gamma_{Ff}\ Q_k)$$

where $\Delta\sigma(\gamma_{Ff}\ Q_k)$, $\Delta\tau(\gamma_{Ff}\ Q_k)$ is the stress range caused by the fatigue loads specified in EN 1991

λ_i are damage equivalent factors depending on the spectra as specified in the relevant parts of EN 1993.

(2) Where no appropriate data for λ_i are available the design value of nominal stress range may be determined using the principles in Annex A.

NOTE The National Annex may give informations supplementing Annex A.

6.3 Design value of modified nominal stress range

(1) The design value of modified nominal stress ranges $\gamma_{Ff}\ \Delta\sigma_{E,2}$ and $\gamma_{Ff}\ \Delta\tau_{E,2}$ should be determined as follows:

$$\gamma_{Ff}\ \Delta\sigma_{E,2} = k_f \times \lambda_1 \times \lambda_2 \times \lambda_i \times ... \times \lambda_n \times \Delta\sigma(\gamma_{Ff}\ Q_k)$$

(6.2)

$$\gamma_{Ff}\ \Delta\tau_{E,2} = k_f \times \lambda_1 \times \lambda_2 \times \lambda_i \times ... \times \lambda_n \times \Delta\tau(\gamma_{Ff}\ Q_k)$$

where k_f is the stress concentration factor to take account of the local stress magnification in relation to detail geometry not included in the reference $\Delta\sigma_R$-N-curve

NOTE k_f-values may be taken from handbooks or from appropriate finite element calculations.

6.4 Design value of stress range for welded joints of hollow sections

(1) Unless more accurate calculations are carried out the design value of modified nominal stress range $\gamma_{Ff}\Delta\sigma_{E,2}$ should be determined as follows using the simplified model in 4(2):

$$\gamma_{Ff}\ \Delta\sigma_{E,2} = k_1\left(\gamma_{Ff}\ \Delta\sigma^*_{E,2}\right) \tag{6.3}$$

where $\gamma_{Ff}\ \Delta\sigma^*_{E,2}$ is the design value of stress range calculated with a simplified truss model with pinned joints

k_1 is the magnification factor according to Table 4.1 and Table 4.2.

6.5 Design value of stress range for geometrical (hot spot) stress

(1) The design value of geometrical (hot spot) stress range $\gamma_{Ff}\ \Delta\sigma_{E,2}$ should be determined as follows:

$$\gamma_{Ff}\ \Delta\sigma_{E,2} = k_f\left(\gamma_{Ff}\ \Delta\sigma^*_{E,2}\right) \tag{6.4}$$

where k_f is the stress concentration factor

7 Fatigue strength

7.1 General

(1) The fatigue strength for nominal stresses is represented by a series of (log $\Delta\sigma_R$) – (log N) curves and (log $\Delta\tau_R$) – (log N) curves (S-N-curves), which correspond to typical detail categories. Each detail category is designated by a number which represents, in N/mm^2, the reference value $\Delta\sigma_C$ and $\Delta\tau_C$ for the fatigue strength at 2 million cycles.

(2) For constant amplitude nominal stresses as shown in Figure 7.1 and Figure 7.2 fatigue strengths can be obtained as follows:

$$\Delta\sigma_R^m\ N_R = \Delta\sigma_C^m\ 2\times10^6 \quad \text{with } m = 3 \text{ for } N \le 5\times10^6\text{, see Figure 7.1}$$

$$\Delta\tau_R^m\ N_R = \Delta\tau_C^m\ 2\times10^6 \quad \text{with } m = 5 \text{ for } N \le 10^8\text{, see Figure 7.2}$$

$$\Delta\sigma_D = \left(\frac{2}{5}\right)^{1/3}\Delta\sigma_C = 0{,}737\Delta\sigma_C \quad \text{is the constant amplitude fatigue limit, see Figure 7.1, and}$$

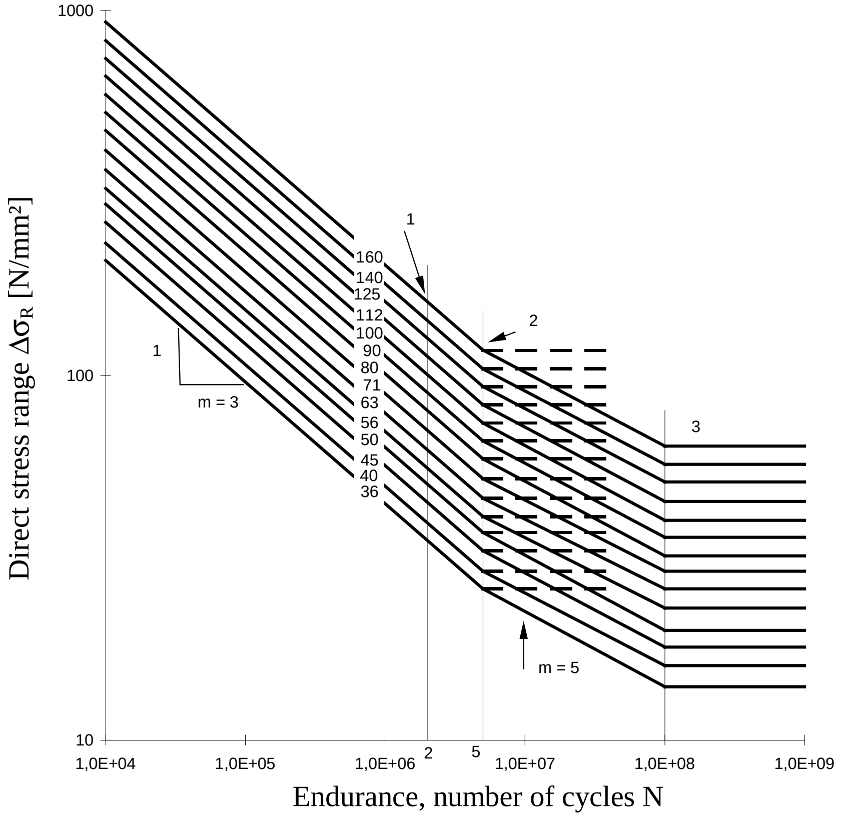
$$\Delta\tau_L = \left(\frac{2}{100}\right)^{1/5}\Delta\tau_C = 0{,}457\Delta\tau_C \quad \text{is the cut off limit, see Figure 7.2.}$$

(3) For nominal stress spectra with stress ranges above and below the constant amplitude fatigue limit $\Delta\sigma_D$ the fatigue strength should be based on the extended fatigue strength curves as follows:

$$\Delta\sigma_R^m\ N_R = \Delta\sigma_C^m\ 2\times10^6 \quad \text{with } m = 3 \text{ for } N \le 5\times10^6$$

$$\Delta\sigma_R^m\ N_R = \Delta\sigma_D^m\ 5\times10^6 \quad \text{with } m = 5 \text{ for } 5\times10^6 \le N_R \le 10^8$$

$$\Delta\sigma_L = \left(\frac{5}{100}\right)^{1/5}\Delta\sigma_D = 0{,}549\Delta\sigma_D \quad \text{is the cut off limit, see Figure 7.1.}$$

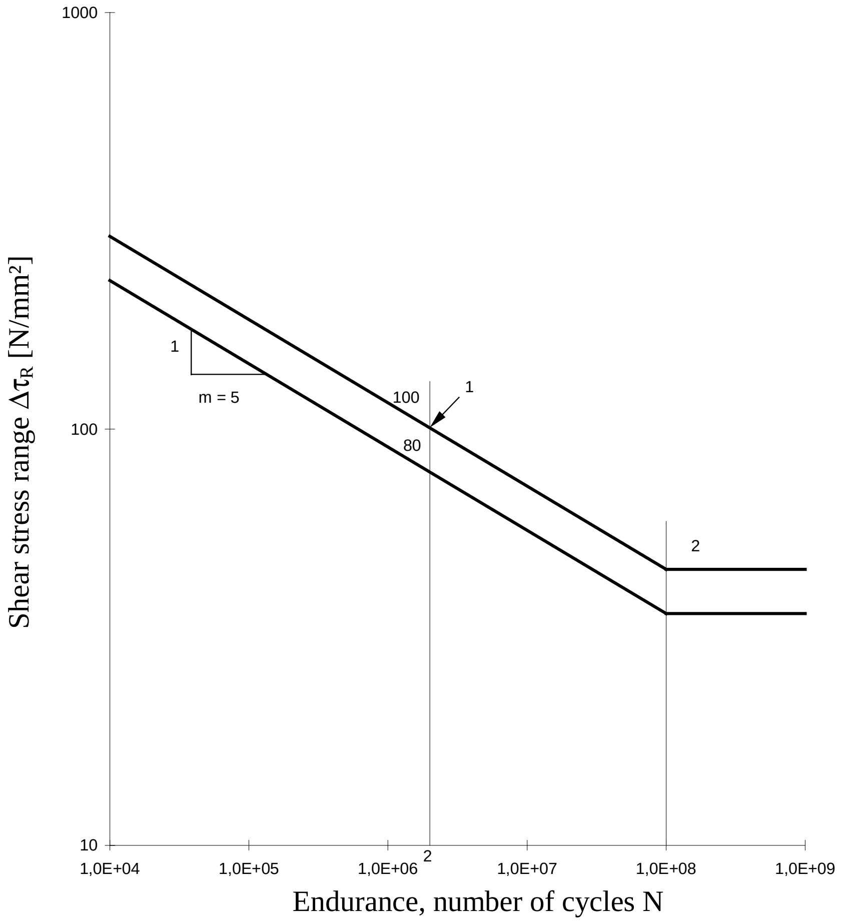


1 Detail category $\Delta\sigma_C$

2 Constant amplitude fatigue limit $\Delta\sigma_D$

3 Cut-off limit $\Delta\sigma_L$

Figure 7.1: Fatigue strength curves for direct stress ranges



1 Detail category $\Delta\tau_C$

2 Cut-off limit $\Delta\tau_L$

Figure 7.2: Fatigue strength curves for shear stress ranges

NOTE 1 When test data were used to determine the appropriate detail category for a particular constructional detail, the value of the stress range $\Delta\sigma_C$ corresponding to a value of N_C = 2 million cycles were calculated for a 75% confidence level of 95% probability of survival for log N, taking into account the standard deviation and the sample size and residual stress effects. The number of data points (not lower than 10) was considered in the statistical analysis, see annex D of EN 1990.

NOTE 2 The National Annex may permit the verification of a fatigue strength category for a particular application provided that it is evaluated in accordance with NOTE 1.

NOTE 3 Test data for some details do not exactly fit the fatigue strength curves in Figure 7.1. In order to ensure that non conservative conditions are avoided, such details, marked with an asterisk, are located one detail category lower than their fatigue strength at 2×10^6 cycles would require. An alternative assessment may increase the classification of such details by one detail category provided that the constant amplitude fatigue limit $\Delta\sigma_D$ is defined as the fatigue strength at 10^7 cycles for m=3 (see Figure 7.3).

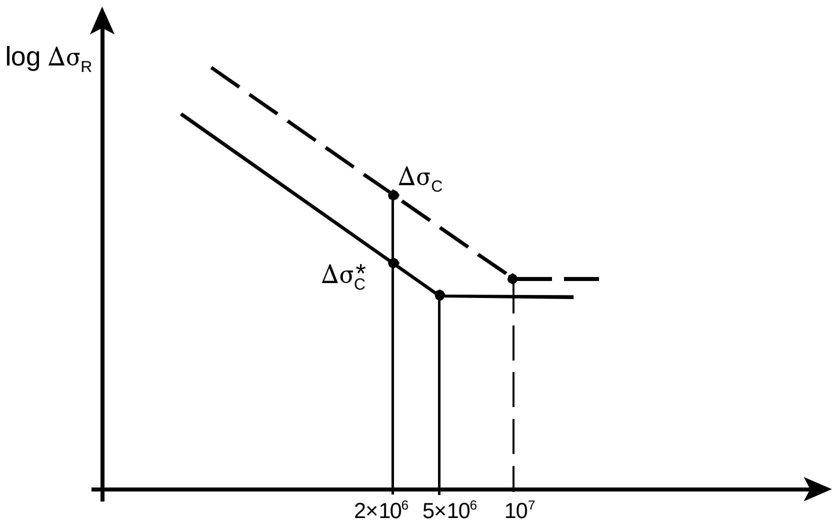


Figure 7.3: Alternative strength $\Delta\sigma_C$ for details classified as $\Delta\sigma_C^*$

(4) Detail categories $\Delta\sigma_C$ and $\Delta\tau_C$ for nominal stresses are given in

Table 8.1 for plain members and mechanically fastened joints

Table 8.2 for welded built-up sections

Table 8.3 for transverse butt welds

Table 8.4 for weld attachments and stiffeners

Table 8.5 for load carrying welded joints

Table 8.6 for hollow sections

Table 8.7 for lattice girder node joints

Table 8.8 for orthotropic decks – closed stringers

Table 8.9 for orthotropic decks – open stringers

Table 8.10 for top flange to web junctions of runway beams

(5) The fatigue strength categories $\Delta\sigma_C$ for geometric stress ranges are given in Annex B.

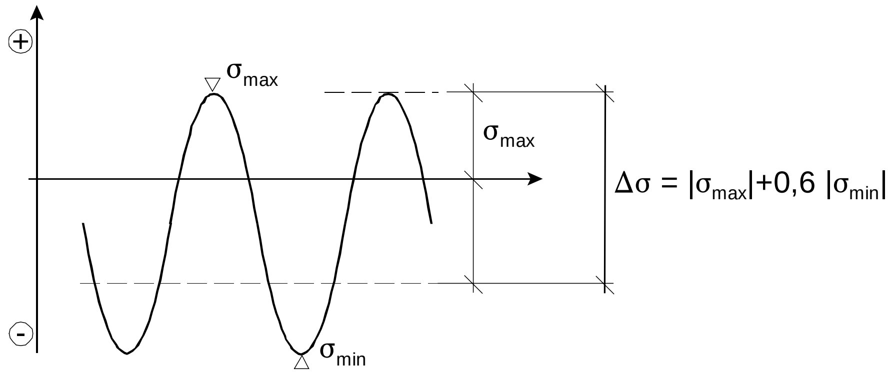
NOTE The National Annex may give fatigue strength categories $\Delta\sigma_C$ and $\Delta\tau_C$ for details not covered by Table 8.1 to Table 8.10 and by Annex B.

7.2 Fatigue strength modifications

7.2.1 Non-welded or stress-relieved welded details in compression

(1) In non-welded details or stress-relieved welded details, the mean stress influence on the fatigue strength may be taken into account by determining a reduced effective stress range $\Delta\sigma_{E,2}$ in the fatigue assessment when part or all of the stress cycle is compressive.

(2) The effective stress range may be calculated by adding the tensile portion of the stress range and 60% of the magnitude of the compressive portion of the stress range, see Figure 7.4.



+ *tension*
– *compression*

Figure 7.4: Modified stress range for non-welded or stress relieved details

7.2.2 Size effect

(1) The size effect due to thickness or other dimensional effects should be taken into account as given in Table 8.1 to Table 8.10. The fatigue strength then is given by:

$$\Delta\sigma_{C,red} = k_s \Delta\sigma_C \tag{7.1}$$

8 Fatigue verification

(1) Nominal, modified nominal or geometric stress ranges due to frequent loads $\psi_1\, Q_k$ (see EN 1990) shall not exceed

$$\begin{aligned} &\Delta\sigma \le 1{,}5\, f_y && \text{for direct stress ranges} \\ &\Delta\tau \le 1{,}5\, f_y / \sqrt{3} && \text{for shear stress ranges} \end{aligned} \tag{8.1}$$

(2) It shall be verified that under fatigue loading

$$\frac{\gamma_{Ff}\ \Delta\sigma_{E,2}}{\Delta\sigma_C / \gamma_{Mf}} \le 1{,}0$$

and (8.2)

$$\frac{\gamma_{Ff}\ \Delta\tau_{E,2}}{\Delta\tau_C / \gamma_{Mf}} \le 1{,}0$$

NOTE Table 8.1 to Table 8.9 require stress ranges to be based on principal stresses for some details.

(3) Unless otherwise stated in the fatigue strength categories in Table 8.8 and Table 8.9, in the case of combined stress ranges $\Delta\sigma_{E,2}$ and $\Delta\tau_{E,2}$ it shall be verified that:

$$\left(\frac{\gamma_{Ff}\ \Delta\sigma_{E,2}}{\Delta\sigma_C / \gamma_{Mf}}\right)^3 + \left(\frac{\gamma_{Ff}\ \Delta\tau_{E,2}}{\Delta\tau_C / \gamma_{Mf}}\right)^5 \le 1{,}0 \tag{8.3}$$

(4) When no data for $\Delta\sigma_{E,2}$ or $\Delta\tau_{E,2}$ are available the verification format in Annex A may be used.

NOTE The National Annex may give information on the use of Annex A.

Table 8.1: Plain members and mechanically fastened joints

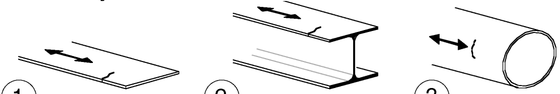
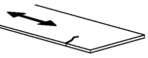
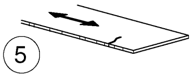
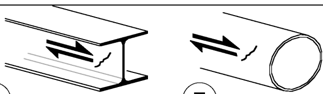
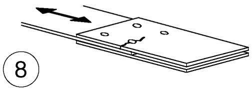
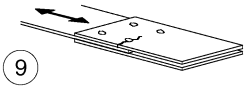
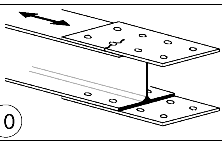
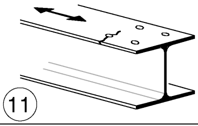
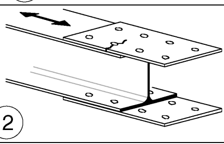
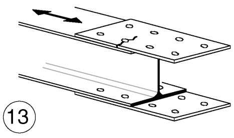
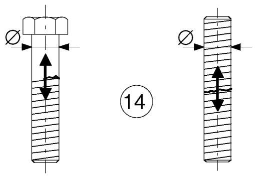
Detail category	Constructional detail	Description	Requirements	
160	**NOTE** The fatigue strength curve associated with category 160 is the highest. No detail can reach a better fatigue strength at any number of cycles.  1 2 3	Rolled and extruded products: 1) Plates and flats; 2) Rolled sections; 3) Seamless hollow sections, either rectangular or circular.	Details 1) to 3): Sharp edges, surface and rolling flaws to be improved by grinding until removed and smooth transition achieved.	
140	 4	Sheared or gas cut plates: 4) Machine gas cut or sheared material with subsequent dressing. 5) Material with machine gas cut edges having shallow and regular drag lines or manual gas cut material, subsequently dressed to remove all edge discontinuities. Machine gas cut with cut quality according to EN 1090.	4) All visible signs of edge discontinuities to be removed. The cut areas are to be machined or ground and all burrs to be removed. Any machinery scratches for example from grinding operations, can only be parallel to the stresses. Details 4) and 5): - Re-entrant corners to be improved by grinding (slope ≤ ¼) or evaluated using the appropriate stress concentration factors. - No repair by weld refill.	
125	 5			
100 m = 5	 6 7	6) and 7) Rolled and extruded products as in details 1), 2), 3)	Details 6) and 7): Δτ calculated from: $\tau = \frac{V\,S(t)}{I\,t}$	
For detail 1 – 5 made of weathering steel use the next lower category.				
112	 8	8) Double covered symmetrical joint with preloaded high strength bolts.	8) Δσ to be calculated on the gross cross-section.	For bolted connections (Details 8) to 13)) in general: End distance: $e_1 \geq 1{,}5\ d$ Edge distance: $e_2 \geq 1{,}5\ d$ Spacing: $p_1 \geq 2{,}5\ d$ Spacing: $p_2 \geq 2{,}5\ d$ Detailing to EN 1993-1-8, Figure 3.1
		8) Double covered symmetrical joint with preloaded injection bolts.	8) ... gross cross-section.	
90	 9	9) Double covered joint with fitted bolts.	9) ... net cross-section.	
		9) Double covered joint with non preloaded injection bolts.	9) ... net cross-section.	
	 10	10) One sided connection with preloaded high strength bolts.	10) ... gross cross-section.	
		10) One sided connection with preloaded injection bolts.	10) ... gross cross-section.	
	 11	11) Structural element with holes subject to bending and axial forces	11) ... net cross-section.	
80	 12	12) One sided connection with fitted bolts.	12) ... net cross-section.	
		12) One sided connection with non-preloaded injection bolts.	12) ... net cross-section.	
50	 13	13) One sided or double covered symmetrical connection with non-preloaded bolts in normal clearance holes. No load reversals.	13) ... net cross-section.	
50	size effect for ⌀ > 30mm: $k_s=(30/\varnothing)^{0,25}$  14	14) Bolts and rods with rolled or cut threads in tension. For large diameters (anchor bolts) the size effect has to be taken into account with k_s.	14) Δσ to be calculated using the tensile stress area of the bolt. Bending and tension resulting from prying effects and bending stresses from other sources must be taken into account. For preloaded bolts, the reduction of the stress range may be taken into account.	

Table 8.1 (continued): Non-welded details

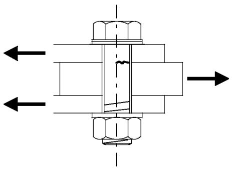
Detail category	Constructional detail	Description	Requirements
100 m=5	15 	Bolts in single or double shear Thread not in the shear plane 15) - Fitted bolts - normal bolts without load reversal (bolts of grade 5.6, 8.8 or 10.9)	15) $\Delta\tau$ calculated on the shank area of the bolt.

Table 8.2: Welded built-up sections

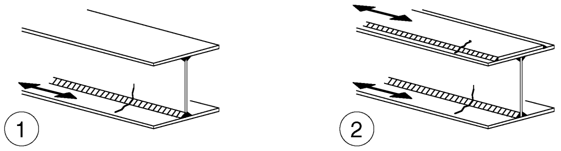
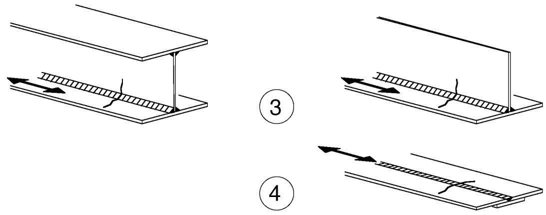
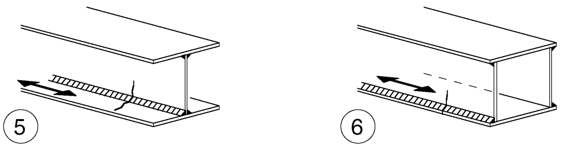
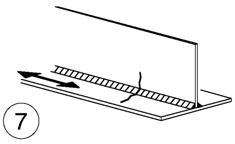
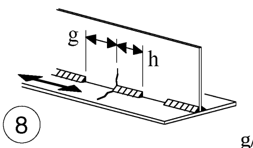
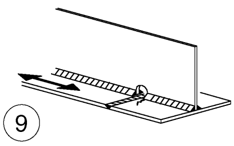
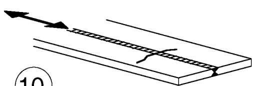
Detail category	Constructional detail	Description	Requirements
125	 1 2	Continuous longitudinal welds: 1) Automatic butt welds carried out from both sides. 2) Automatic fillet welds. Cover plate ends to be checked using detail 6) or 7) in Table 8.5.	Details 1) and 2): No stop/start position is permitted except when the repair is performed by a specialist and inspection is carried out to verify the proper execution of the repair.
112	 3 4	3) Automatic fillet or butt weld carried out from both sides but containing stop/start positions. 4) Automatic butt welds made from one side only, with a continuous backing bar, but without stop/start positions.	4) When this detail contains stop/start positions category 100 to be used.
100	 5 6	5) Manual fillet or butt weld. 6) Manual or automatic butt welds carried out from one side only, particularly for box girders	5), 6) A very good fit between the flange and web plates is essential. The web edge to be prepared such that the root face is adequate for the achievement of regular root penetration without break-out.
100	 7	7) Repaired automatic or manual fillet or butt welds for categories 1) to 6).	7) Improvement by grinding performed by specialist to remove all visible signs and adequate verification can restore the original category.
80	 8 g/h ≤ 2,5	8) Intermittent longitudinal fillet welds.	8) $\Delta\sigma$ based on direct stress in flange.
71	 9	9) Longitudinal butt weld, fillet weld or intermittent weld with a cope hole height not greater than 60 mm. For cope holes with a height > 60 mm see detail 1) in Table 8.4	9) $\Delta\sigma$ based on direct stress in flange.
125	 10	10) Longitudinal butt weld, both sides ground flush parallel to load direction, 100% NDT	
112		10) No grinding and no start/stop	
90		10) with start/stop positions	
140	 11	11) Automatic longitudinal seam weld without stop/start positions in hollow sections	11) Free from defects outside the tolerances of EN 1090. Wall thickness t ≤ 12,5 mm.
125		11) Automatic longitudinal seam weld without stop/start positions in hollow sections	11) Wall thickness t > 12,5 mm.
90		11) with stop/start positions	
For details 1 to 11 made with fully mechanized welding the categories for automatic welding apply.			

Table 8.3: Transverse butt welds

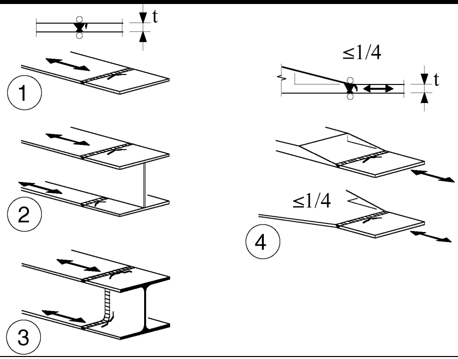
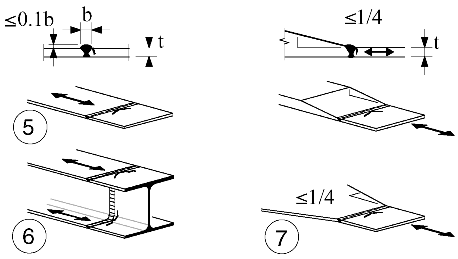
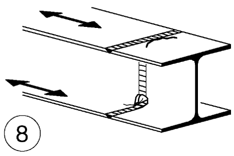
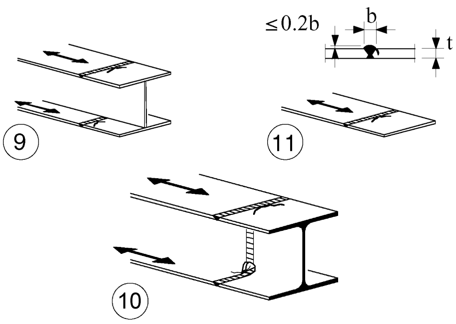
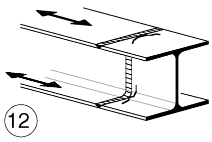
Detail category	Constructional detail		Description	Requirements
112	size effect for t>25mm: $k_s=(25/t)^{0,2}$		<u>Without backing bar:</u> 1) Transverse splices in plates and flats. 2) Flange and web splices in plate girders before assembly. 3) Full cross-section butt welds of rolled sections without cope holes. 4) Transverse splices in plates or flats tapered in width or in thickness, with a slope ≤ ¼.	- All welds ground flush to plate surface parallel to direction of the arrow. - Weld run-on and run-off pieces to be used and subsequently removed, plate edges to be ground flush in direction of stress. - Welded from both sides; checked by NDT. <u>Detail 3):</u> Applies only to joints of rolled sections, cut and rewelded.
90	size effect for t>25mm: $k_s=(25/t)^{0,2}$		5) Transverse splices in plates or flats. 6) Full cross-section butt welds of rolled sections without cope holes. 7) Transverse splices in plates or flats tapered in width or in thickness with a slope ≤ ¼. Translation of welds to be machined notch free.	- The height of the weld convexity to be not greater than 10% of the weld width, with smooth transition to the plate surface. - Weld run-on and run-off pieces to be used and subsequently removed, plate edges to be ground flush in direction of stress. - Welded from both sides; checked by NDT. <u>Details 5 and 7:</u> Welds made in flat position.
90	size effect for t>25mm: $k_s=(25/t)^{0,2}$		8) As detail 3) but with cope holes.	- All welds ground flush to plate surface parallel to direction of the arrow. - Weld run-on and run-off pieces to be used and subsequently removed, plate edges to be ground flush in direction of stress. - Welded from both sides; checked by NDT. - Rolled sections with the same dimensions without tolerance differences
80	size effect for t>25mm: $k_s=(25/t)^{0,2}$		9) Transverse splices in welded plate girders without cope hole. 10) Full cross-section butt welds of rolled sections with cope holes. 11) Transverse splices in plates, flats, rolled sections or plate girders.	- The height of the weld convexity to be not greater than 20% of the weld width, with smooth transition to the plate surface. - Weld not ground flush - Weld run-on and run-off pieces to be used and subsequently removed, plate edges to be ground flush in direction of stress. - Welded from both sides; checked by NDT. <u>Detail 10:</u> The height of the weld convexity to be not greater than 10% of the weld width, with smooth transition to the plate surface.
63			12) Full cross-section butt welds of rolled sections without cope hole.	- Weld run-on and run-off pieces to be used and subsequently removed, plate edges to be ground flush in direction of stress. - Welded from both sides.

Table 8.3 (continued): Transverse butt welds

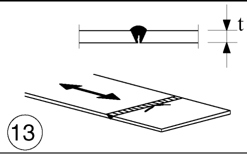
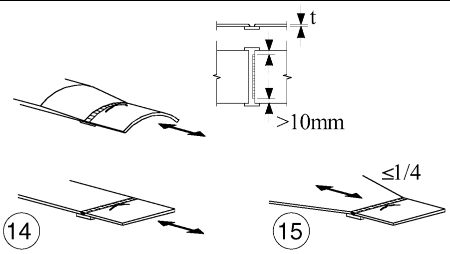
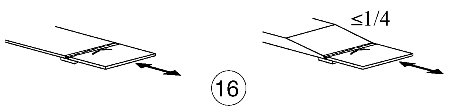
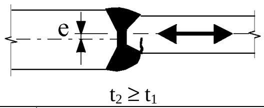
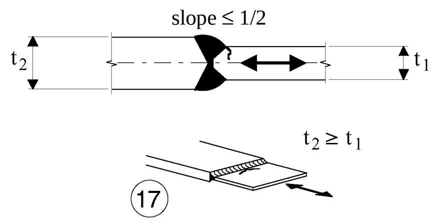
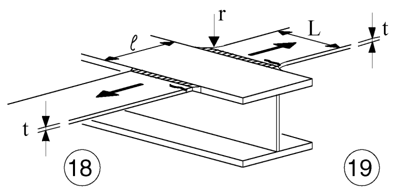
Detail category	Constructional detail	Description	Requirements
36	 	13) Butt welds made from one side only.	13) Without backing strip.
71	size effect for t>25mm: $k_s=(25/t)^{0,2}$	13) Butt welds made from one side only when full penetration checked by appropriate NDT.	
71	size effect for t>25mm: $k_s=(25/t)^{0,2}$  	<u>With backing strip:</u> 14) Transverse splice. 15) Transverse butt weld tapered in width or thickness with a slope ≤ ¼. Also valid for curved plates.	<u>Details 14) and 15):</u> Fillet welds attaching the backing strip to terminate ≥ 10 mm from the edges of the stressed plate. Tack welds inside the shape of butt welds.
50	size effect for t>25mm: $k_s=(25/t)^{0,2}$  	16) Transverse butt weld on a permanent backing strip tapered in width or thickness with a slope ≤ ¼. Also valid for curved plates.	16) Where backing strip fillet welds end < 10 mm from the plate edge, or if a good fit cannot be guaranteed.
71	size effect for t>25mm and/or generalisation for eccentricity: $k_s = \left(\frac{25}{t_1}\right)^{0,2} \Big/ \left(1+\frac{6e}{t_1}\,\frac{t_1^{1,5}}{t_1^{1,5}+t_2^{1,5}}\right)$   	17) Transverse butt weld, different thicknesses without transition, centrelines aligned.	
As detail 1 in Table 8.5	 	18) Transverse butt weld at intersecting flanges.	<u>Details 18) and 19)</u> The fatigue strength of the continuous component has to be checked with Table 8.4, detail 4 or detail 5.
As detail 4 in Table 8.4		19) With transition radius according to Table 8.4, detail 4	

Table 8.4: Weld attachments and stiffeners

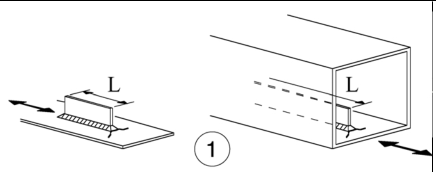
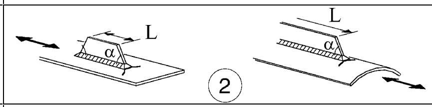
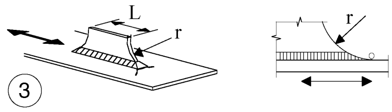
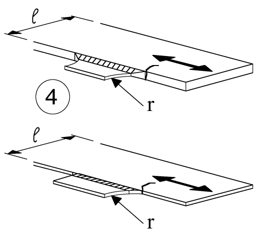
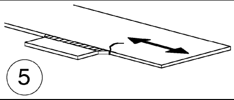
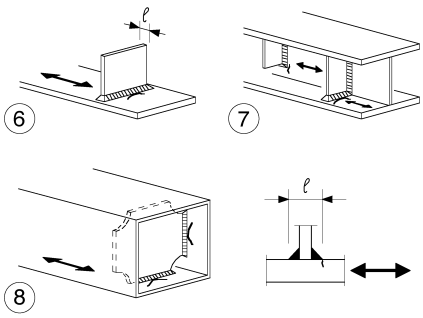
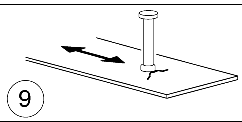
Detail category	Constructional detail		Description	Requirements
80	L≤50mm		<u>Longitudinal attachments:</u> 1) The detail category varies according to the length of the attachment L.	The thickness of the attachment must be less than its height. If not see Table 8.5, details 5 or 6.
71	50<L≤80mm			
63	80<L≤100mm			
56	L>100mm			
71	L>100mm α<45°		2) Longitudinal attachments to plate or tube.	
80	r>150mm	 reinforced	3) Longitudinal fillet welded gusset with radius transition to plate or tube; end of fillet weld reinforced (full penetration); length of reinforced weld > r.	<u>Details 3) and 4):</u> Smooth transition radius r formed by initially machining or gas cutting the gusset plate before welding, then subsequently grinding the weld area parallel to the direction of the arrow so that the transverse weld toe is fully removed.
90	$\frac{r}{L} \geq \frac{1}{3}$ or r>150mm		4) Gusset plate, welded to the edge of a plate or beam flange.	
71	$\frac{1}{6} \leq \frac{r}{L} \leq \frac{1}{3}$			
50	$\frac{r}{L} < \frac{1}{6}$	L: attachment length as above		
40			5) As welded, no radius transition.	
80	ℓ≤50mm		<u>Transverse attachments:</u> 6) Welded to plate. 7) Vertical stiffeners welded to a beam or plate girder. 8) Diaphragm of box girders welded to the flange or the web. May not be possible for small hollow sections. The values are also valid for ring stiffeners.	<u>Details 6) and 7):</u> Ends of welds to be carefully ground to remove any undercut that may be present. 7) $\Delta\sigma$ to be calculated using principal stresses if the stiffener terminates in the web, see left side.
71	50<ℓ≤80mm			
80			9) The effect of welded shear studs on base material.	

Table 8.5: Load carrying welded joints

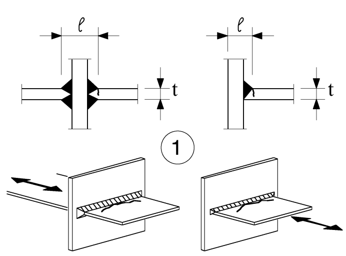
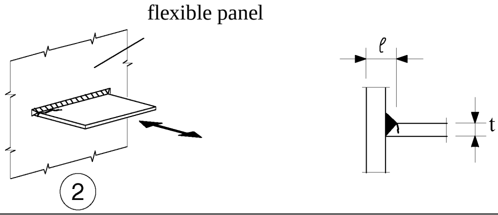
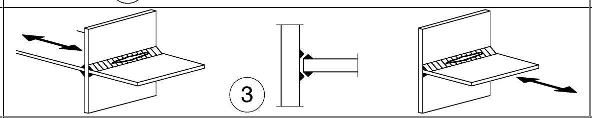
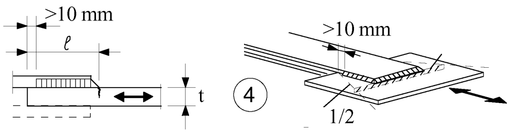
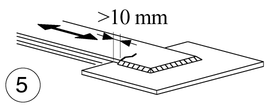
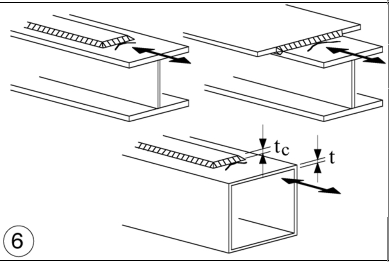
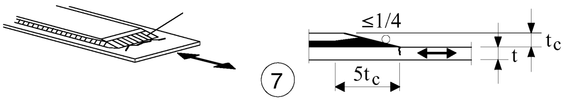
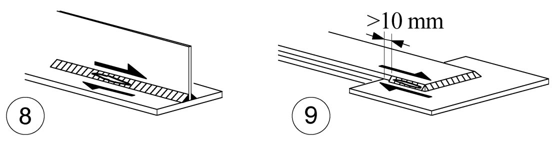
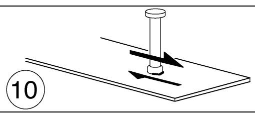
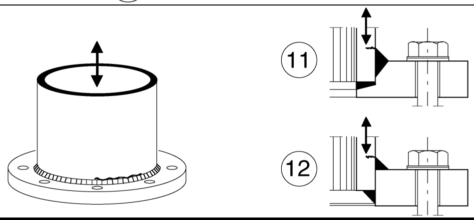
Detail category	Constructional detail			Description	Requirements
80	ℓ<50 mm	all t [mm]		<u>Cruciform and Tee joints:</u> 1) Toe failure in full penetration butt welds and all partial penetration joints.	1) Inspected and found free from discontinuities and misalignments outside the tolerances of EN 1090. 2) For computing $\Delta\sigma$, use modified nominal stress. 3) In partial penetration joints two fatigue assessments are required. Firstly, root cracking evaluated according to stresses defined in section 5, using category 36* for $\Delta\sigma_w$ and category 80 for $\Delta\tau_w$. Secondly, toe cracking is evaluated by determining $\Delta\sigma$ in the load-carrying plate. <u>Details 1) to 3):</u> The misalignment of the load-carrying plates should not exceed 15 % of the thickness of the intermediate plate.
71	50<ℓ≤80	all t			
63	80<ℓ≤100	all t			
56	100<ℓ≤120	all t			
56	ℓ>120	t≤20			
50	120<ℓ≤200 ℓ>200	t>20 20<t≤30			
45	200<ℓ≤300 ℓ>300	t>30 30<t≤50			
40	ℓ>300	t>50			
As detail 1 in Table 8.5	flexible panel 			2) Toe failure from edge of attachment to plate, with stress peaks at weld ends due to local plate deformations.	
36*				3) Root failure in partial penetration Tee-butt joints or fillet welded joint and effective full penetration in Tee-butt joint.	
As detail 1 in Table 8.5	stressed area of main panel: slope = 1/2			<u>Overlapped welded joints:</u> 4) Fillet welded lap joint.	4) $\Delta\sigma$ in the main plate to be calculated on the basis of area shown in the sketch. 5) $\Delta\sigma$ to be calculated in the overlapping plates. <u>Details 4) and 5):</u> - Weld terminations more than 10 mm from plate edge. - Shear cracking in the weld should be checked using detail 8).
45*				<u>Overlapped:</u> 5) Fillet welded lap joint.	
	t_c<t	t_c≥t		<u>Cover plates in beams and plate girders:</u> 6) End zones of single or multiple welded cover plates, with or without transverse end weld.	6) If the cover plate is wider than the flange, a transverse end weld is needed. This weld should be carefully ground to remove undercut. The minimum length of the cover plate is 300 mm. For shorter attachments size effect see detail 1).
56*	t≤20	-			
50	20<t≤30	t≤20			
45	30<t≤50	20<t≤30			
40	t>50	30<t≤50			
36	-	t>50			
56	reinforced transverse end weld 			7) Cover plates in beams and plate girders. $5t_c$ is the minimum length of the reinforcement weld.	7) Transverse end weld ground flush. In addition, if t_c>20mm, front of plate at the end ground with a slope < 1 in 4.
80 m=5				8) Continuous fillet welds transmitting a shear flow, such as web to flange welds in plate girders. 9) Fillet welded lap joint.	8) $\Delta\tau$ to be calculated from the weld throat area. 9) $\Delta\tau$ to be calculated from the weld throat area considering the total length of the weld. Weld terminations more than 10 mm from the plate edge, see also 4) and 5) above.
see EN 1994-2 (90 m=8)				<u>Welded stud shear connectors:</u> 10) For composite application	10) $\Delta\tau$ to be calculated from the nominal cross section of the stud.
71				11) Tube socket joint with 80% full penetration butt welds.	11) Weld toe ground. $\Delta\sigma$ computed in tube.
40				12) Tube socket joint with fillet welds.	12) $\Delta\sigma$ computed in tube.

Table 8.6: Hollow sections (t ≤ 12,5 mm)

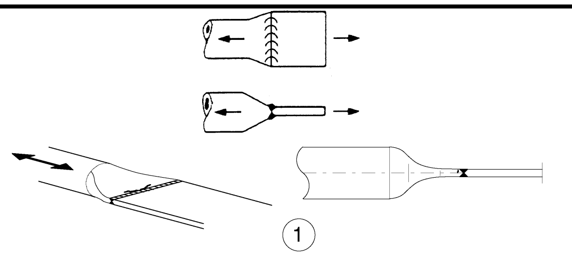
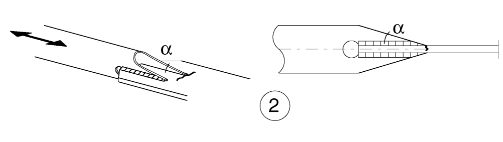
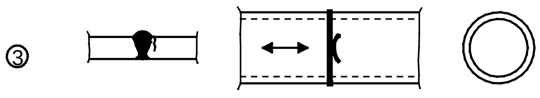
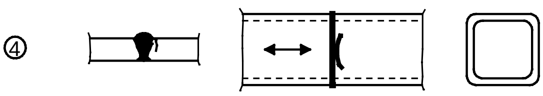
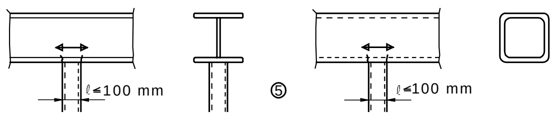
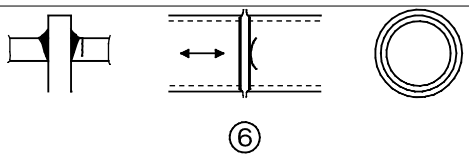
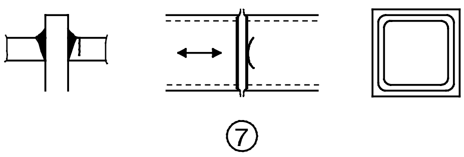
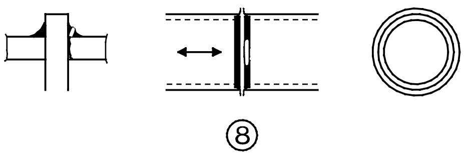
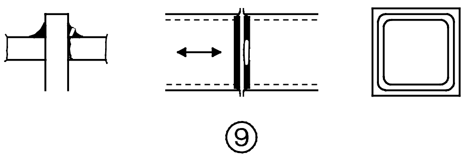
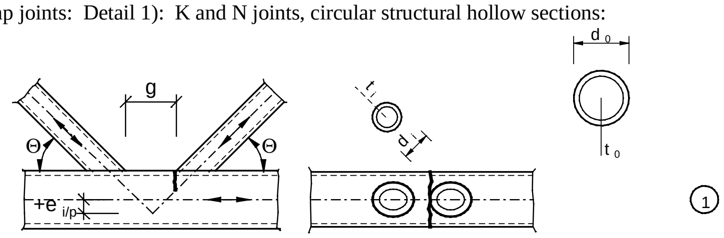
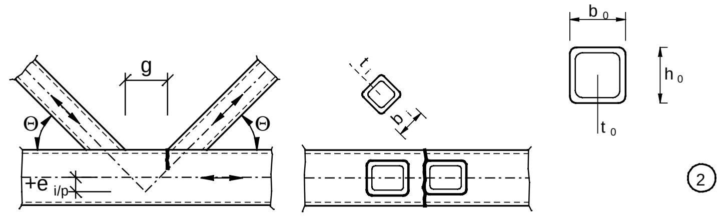
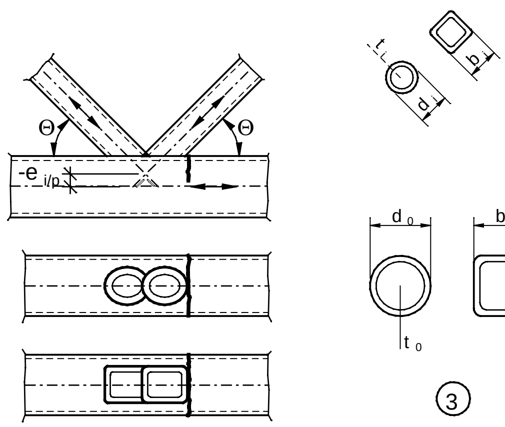
Detail category	Constructional detail	Description	Requirements
71		1) Tube-plate joint, tubes flatted, butt weld (X-groove)	1) $\Delta\sigma$ computed in tube. Only valid for tube diameter less than 200 mm.
71	α≤45° 	2) Tube-plate joint, tube slitted and welded to plate. Holes at end of slit.	2) $\Delta\sigma$ computed in tube. Shear cracking in the weld should be verified using Table 8.5, detail 8).
63	α>45°		
71		Transverse butt welds: 3) Butt-welded end-to-end connections between circular structural hollow sections.	Details 3) and 4): - Weld convexity ≤ 10% of weld width, with smooth transitions. - Welded in flat position, inspected and found free from defects outside the tolerances EN 1090. - Classify 2 detail categories higher if t > 8 mm.
56		4) Butt-welded end-to-end connections between rectangular structural hollow sections.	
71		Welded attachments: 5) Circular or rectangular structural hollow section, fillet-welded to another section.	5) - Non load-carrying welds. - Width parallel to stress direction ℓ ≤ 100 mm. - Other cases see Table 8.4.
50		Welded splices: 6) Circular structural hollow sections, butt-welded end-to-end with an intermediate plate.	Details 6) and 7): - Load-carrying welds. - Welds inspected and found free from defects outside the tolerances of EN 1090. - Classify 1 detail category higher if t > 8 mm.
45		7) Rectangular structural hollow sections, butt welded end-to-end with an intermediate plate.	
40		8) Circular structural hollow sections, fillet-welded end-to-end with an intermediate plate.	Details 8) and 9): - Load-carrying welds. - Wall thickness t ≤ 8 mm.
36		9) Rectangular structural hollow sections, fillet-welded end-to-end with an intermediate plate.	

Table 8.7: Lattice girder node joints

<table>
<tr><th>Detail category</th><th colspan="2">Constructional detail</th><th>Requirements</th></tr>
<tr><td>90
m=5</td><td>$\frac{t_0}{t_i} \geq 2,0$</td><td rowspan="2">Gap joints: Detail 1): K and N joints, circular structural hollow sections:

</td><td rowspan="4"><u>Details 1) and 2):</u>

- Separate assessments needed for the chords and the braces.
- For intermediate values of the ratio t_o/t_i interpolate linearly between detail categories.
- Fillet welds permitted for braces with wall thickness $t \leq 8$ mm.
- t_0 and $t_i \leq 8$mm
- $35° \leq \theta \leq 50°$
- $b_0/t_0 \times t_0/t_i \leq 25$
- $d_0/t_0 \times t_0/t_i \leq 25$
- $0,4 \leq b_i/b_0 \leq 1,0$
- $0,25 \leq d_i/d_0 \leq 1,0$
- $b_0 \leq 200$ mm
- $d_0 \leq 300$ mm
- $-0,5h_0 \leq e_{i/p} \leq 0,25h_0$
- $-0,5d_0 \leq e_{i/p} \leq 0,25d_0$
- $e_{o/p} \leq 0,02b_0$ or $\leq 0,02d_0$

[$e_{o/p}$ is out-of-plane eccentricity]

<u>Detail 2):</u>
$0,5(b_o - b_i) \leq g \leq 1,1(b_o - b_i)$
and $g \geq 2t_o$</td></tr>
<tr><td>45
m=5</td><td>$\frac{t_0}{t_i} = 1,0$</td></tr>
<tr><td>71
m=5</td><td>$\frac{t_0}{t_i} \geq 2,0$</td><td rowspan="2">Gap joints: Detail 2): K and N joints, rectangular structural hollow sections:

</td></tr>
<tr><td>36
m=5</td><td>$\frac{t_0}{t_i} = 1,0$</td></tr>
<tr><td>71
m=5</td><td>$\frac{t_0}{t_i} \geq 1,4$</td><td rowspan="2">Overlap joints: Detail 3): K joints, circular or rectangular structural hollow sections:

</td><td rowspan="4"><u>Details 3) and 4):</u>

- 30 % ≤ overlap ≤ 100 %
- overlap = (q/p) × 100 %
- Separate assessments needed for the chords and the braces.
- For intermediate values of the ratio t_o/t_i interpolate linearly between detail categories.
- Fillet welds permitted for braces with wall thickness $t \leq 8$ mm.
- t_0 and $t_i \leq 8$mm
- $35° \leq \theta \leq 50°$
- $b_0/t_0 \times t_0/t_i \leq 25$
- $d_0/t_0 \times t_0/t_i \leq 25$
- $0,4 \leq b_i/b_0 \leq 1,0$
- $0,25 \leq d_i/d_0 \leq 1,0$
- $b_0 \leq 200$ mm
- $d_0 \leq 300$ mm
- $-0,5h_0 \leq e_{i/p} \leq 0,25h_0$
- $-0,5d_0 \leq e_{i/p} \leq 0,25d_0$
- $e_{o/p} \leq 0,02b_0$ or $\leq 0,02d_0$

[$e_{o/p}$ is out-of-plane eccentricity]

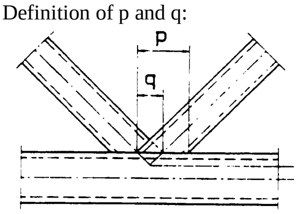
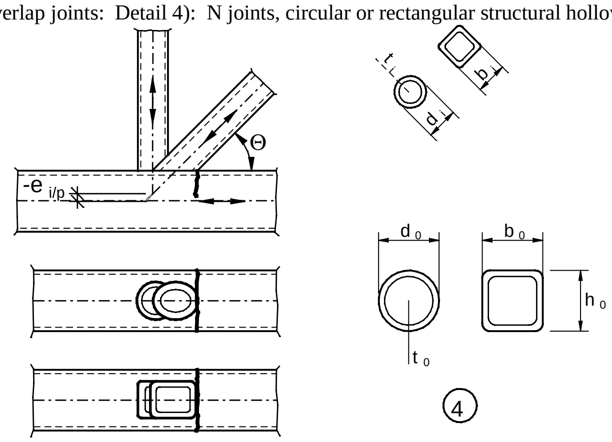
Definition of p and q:

</td></tr>
<tr><td>56
m=5</td><td>$\frac{t_0}{t_i} = 1,0$</td></tr>
<tr><td>71
m=5</td><td>$\frac{t_0}{t_i} \geq 1,4$</td><td rowspan="2">Overlap joints: Detail 4): N joints, circular or rectangular structural hollow sections:

</td></tr>
<tr><td>50
m=5</td><td>$\frac{t_0}{t_i} = 1,0$</td></tr>
</table>

Table 8.8: Orthotropic decks – closed stringers

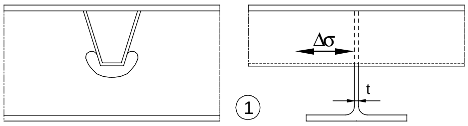
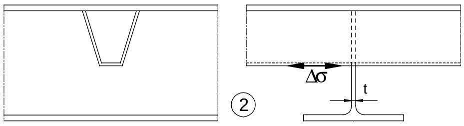
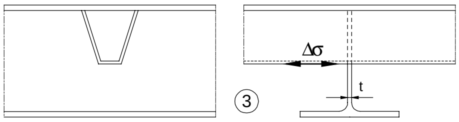
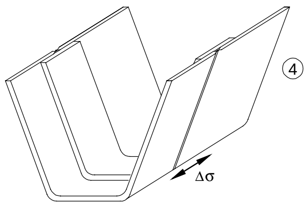
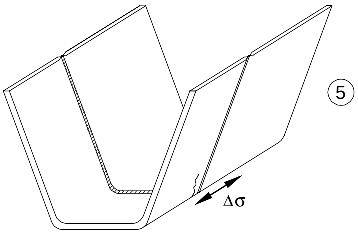
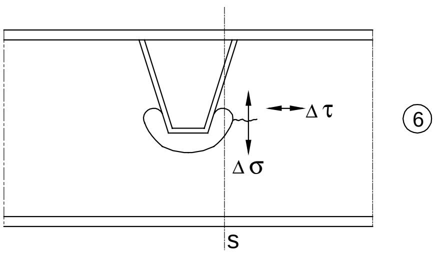
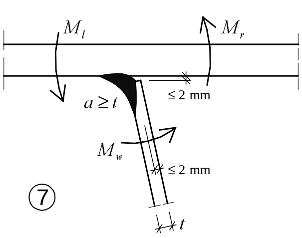
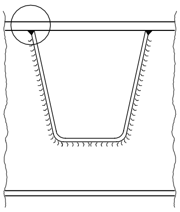
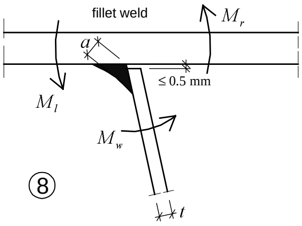
Detail category	Constructional detail		Description	Requirements
80	t≤12mm		1) Continuous longitudinal stringer, with additional cutout in cross girder.	1) Assessment based on the direct stress range Δσ in the longitudinal stringer.
71	t>12mm			
80	t≤12mm		2) Continuous longitudinal stringer, no additional cutout in cross girder.	2) Assessment based on the direct stress range Δσ in the stringer.
71	t>12mm			
36			3) Separate longitudinal stringer each side of the cross girder.	3) Assessment based on the direct stress range Δσ in the stringer.
71			4) Joint in rib, full penetration butt weld with steel backing plate.	4) Assessment based on the direct stress range Δσ in the stringer.
112	As detail 1, 2, 4 in Table 8.3		5) Full penetration butt weld in rib, welded from both sides, without backing plate.	5) Assessment based on the direct stress range Δσ in the stringer. Tack welds inside the shape of butt welds.
90	As detail 5, 7 in Table 8.3			
80	As detail 9, 11 in Table 8.3			
36		 $\Delta\sigma = \frac{\Delta M_s}{W_{net,s}}$ $\Delta\tau = \frac{\Delta V_s}{A_{w,net,s}}$	6) Connection of continuous longitudinal rib to cross girder.	6) Assessment based on $\Delta\sigma_{eq}$ combining the shear stress range Δτ and direct stress range Δσ in the web, as an equivalent stress range: $\Delta\sigma_{eq} = \frac{1}{2}\left(\Delta\sigma + \sqrt{\Delta\sigma^2 + 4\Delta\tau^2}\right)$
71		$\Delta\sigma = \frac{\Delta M_w}{W_w}$ 	<u>Weld connecting deck plate to trapezoidal or V-section rib</u> 7) Partial penetration weld with a ≥ t	7) Assessment based on direct stress range from bending in the plate.
50			8) Fillet weld or partial penetration welds out of the range of detail 7)	8) Assessment based on direct stress range from bending in the plate.

Table 8.9: Orthotropic decks – open stringers

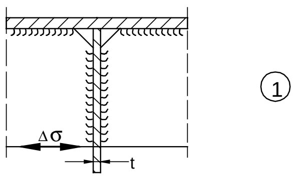
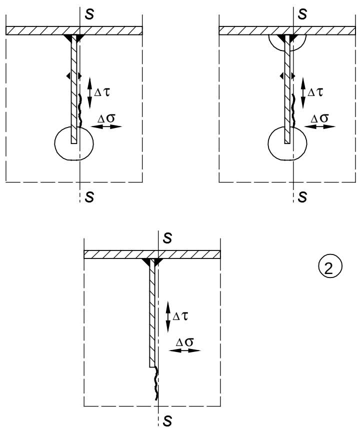
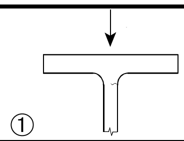
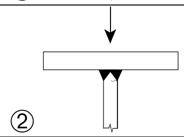
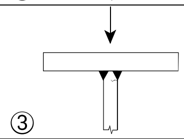
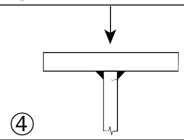
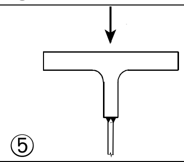
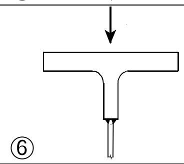
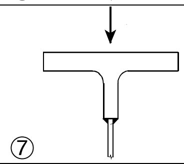
Detail category	Constructional detail	Description	Requirements
80	t≤12mm 	1) Connection of continuous longitudinal stringer to cross girder.	1) Assessment based on the direct stress range $\Delta\sigma$ in the stringer.
71	t>12mm		
56		2) Connection of continuous longitudinal stringer to cross girder. $\Delta\sigma = \frac{\Delta M_s}{W_{net,s}}$ $\Delta\tau = \frac{\Delta V_s}{A_{w,net,s}}$ Check also stress range between stringers as defined in EN 1993-2.	2) Assessment based on combining the shear stress range $\Delta\tau$ and direct stress range $\Delta\sigma$ in the web of the cross girder, as an equivalent stress range: $\Delta\sigma_{eq} = \frac{1}{2}\left(\Delta\sigma + \sqrt{\Delta\sigma^2 + 4\Delta\tau^2}\right)$

Table 8.10: Top flange to web junction of runway beams

Detail category	Constructional detail	Description	Requirements
160		1) Rolled I- or H-sections	1) Vertical compressive stress range $\Delta\sigma_{vert.}$ in web due to wheel loads
71		2) Full penetration tee-butt weld	2) Vertical compressive stress range $\Delta\sigma_{vert.}$ in web due to wheel loads
36*		3) Partial penetration tee-butt welds, or effective full penetration tee-butt weld conforming with EN 1993-1-8	3) Stress range $\Delta\sigma_{vert.}$ in weld throat due to vertical compression from wheel loads
36*		4) Fillet welds	4) Stress range $\Delta\sigma_{vert.}$ in weld throat due to vertical compression from wheel loads
71		5) T-section flange with full penetration tee-butt weld	5) Vertical compressive stress range $\Delta\sigma_{vert.}$ in web due to wheel loads
36*		6) T-section flange with partial penetration tee-butt weld, or effective full penetration tee-butt weld conforming with EN 1993-1-8	6) Stress range $\Delta\sigma_{vert.}$ in weld throat due to vertical compression from wheel loads
36*		7) T-section flange with fillet welds	7) Stress range $\Delta\sigma_{vert.}$ in weld throat due to vertical compression from wheel loads

Annex A [normative] – Determination of fatigue load parameters and verification formats

A.1 Determination of loading events

(1) Typical loading sequences that represent a credible estimated upper bound of all service load events expected during the fatigue design life should be determined using prior knowledge from similar structures, see Figure A.1 a).

A.2 Stress history at detail

(1) A stress history should be determined from the loading events at the structural detail under consideration taking account of the type and shape of the relevant influence lines to be considered and the effects of dynamic magnification of the structural response, see Figure A.1 b).

(2) Stress histories may also be determined from measurements on similar structures or from dynamic calculations of the structural response.

A.3 Cycle counting

(1) Stress histories may be evaluated by either of the following cycle counting methods:

- rainflow method
- reservoir method, see Figure A.1 c).

to determine

- stress ranges and their numbers of cycles
- mean stresses, where the mean stress influence needs to be taken into account.

A.4 Stress range spectrum

(1) The stress range spectrum should be determined by presenting the stress ranges and the associated number of cycles in descending order, see Figure A.1 d).

(2) Stress range spectra may be modified by neglecting peak values of stress ranges representing less than 1% of the total damage and small stress ranges below the cut off limit.

(3) Stress range spectra may be standardised according to their shape, e.g. with the coordinates $\overline{\Delta\sigma} = 1{,}0$ and $\overline{\Sigma n} = 1{,}0$.

A.5 Cycles to failure

(1) When using the design spectrum the applied stress ranges $\Delta\sigma_i$ should be multiplied by γ_{Ff} and the fatigue strength values $\Delta\sigma_C$ divided by γ_{Mf} in order to obtain the endurance value N_{Ri} for each band in the spectrum. The damage D_d during the design life should be calculated from:

$$D_d = \sum_i^n \frac{n_{Ei}}{N_{Ri}} \tag{A.1}$$

where n_{Ei} is the number of cycles associated with the stress range $\gamma_{Ff}\Delta\sigma_i$ for band i in the factored spectrum

N_{Ri} is the endurance (in cycles) obtained from the factored $\frac{\Delta\sigma_C}{\gamma_{Mf}} - N_R$ curve for a stress range of $\gamma_{Ff}\,\Delta\sigma_i$

(2) On the basis of equivalence of D_d the design stress range spectrum may be transformed into any equivalent design stress range spectrum, e.g. a constant amplitude design stress range spectrum yielding the fatigue equivalent load Q_e associated with the cycle number $n_{max} = \sum n_i$ or $Q_{E,2}$ associated with the cycle number $N_C = 2\times10^6$.

A.6 Verification formats

(1) The fatigue assessment based on damage accumulation shall meet the following criteria:

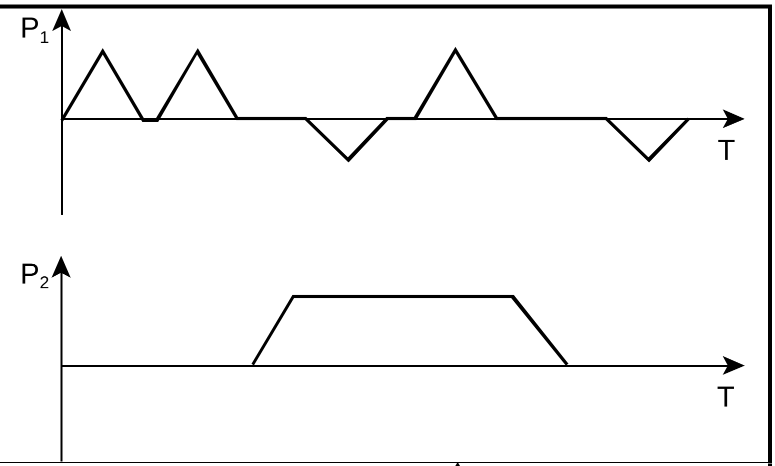
– based on damage accumulation:

$$D_d \leq 1{,}0 \qquad \text{(A.2)}$$

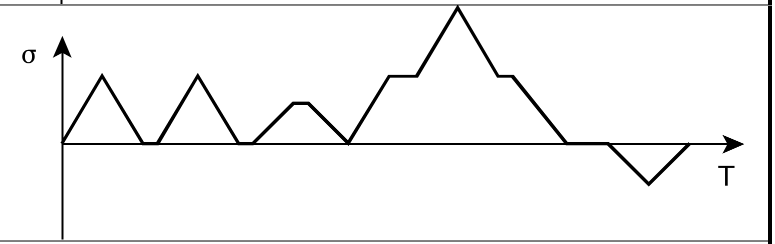
– based on stress range:

$$\gamma_{Ff}\,\Delta\sigma_{E,2} \leq \sqrt[m]{D_d}\,\frac{\Delta\sigma_C}{\gamma_{Mf}} \quad \text{where } m = 3 \qquad \text{(A.3)}$$

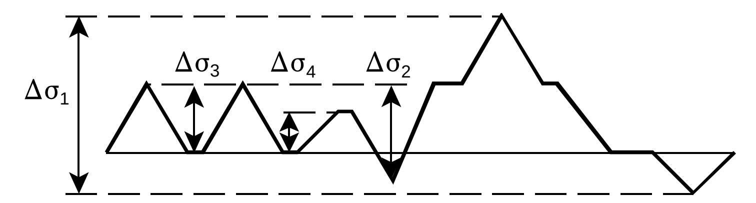
a) Loading sequence: Typical load cycle (repeated n-times in the design life)



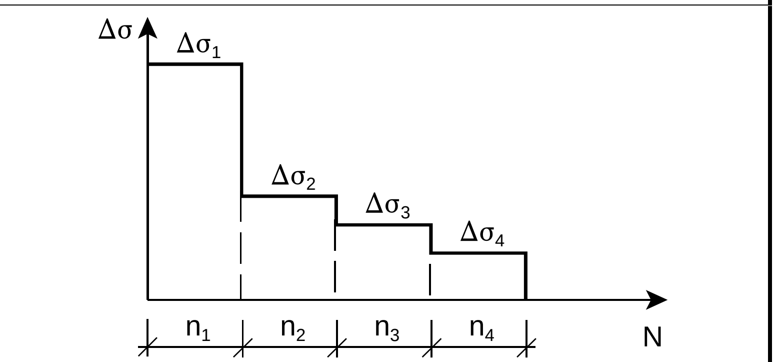
b) Stress history at detail



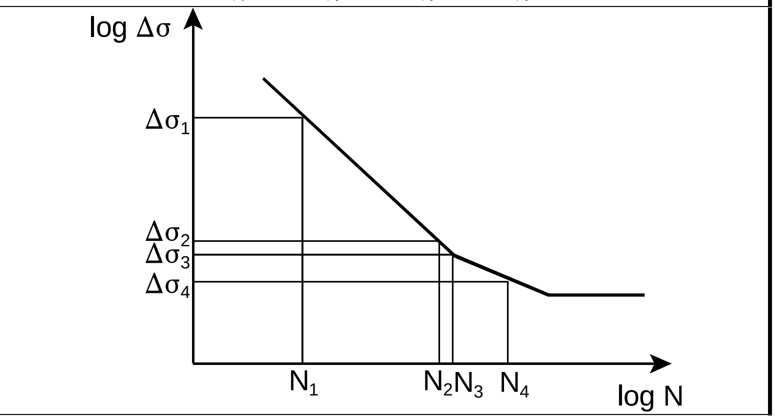
c) Cycle counting (e.g. reservoir method)



d) Stress range spectrum



e) Cycles to failure



f) Damage summation (Palmgren-Miner rule)

$$\sum \frac{n_i}{N_i} = \frac{n_1}{N_1} + \frac{n_2}{N_2} + \frac{n_3}{N_3} + \frac{n_4}{N_4} \leq D_L$$

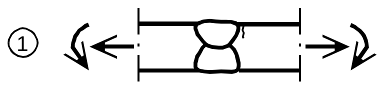
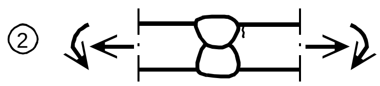
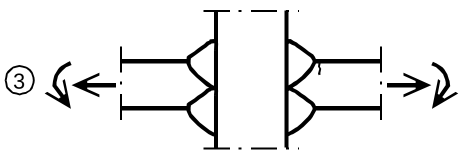
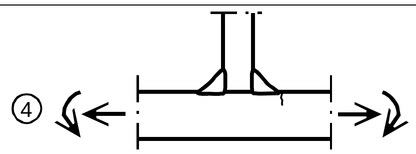
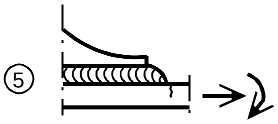
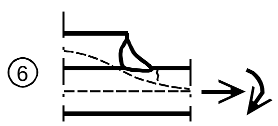
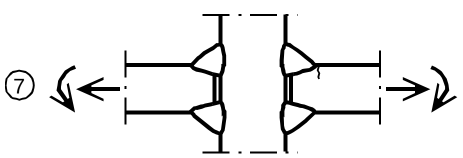
Figure A.1: Cumulative damage method

Annex B [normative] – Fatigue resistance using the geometric (hot spot) stress method

(1) For the application of the geometric stress method detail categories are given in Table B.1 for cracks initiating from

- toes of butt welds,
- toes of fillet welded attachments,
- toes of fillet welds in cruciform joints.

Table B.1: Detail categories for use with geometric (hot spot) stress method

Detail category	Constructional detail	Description	Requirements
112		1) Full penetration butt joint.	1) - All welds ground flush to plate surface parallel to direction of the arrow. - Weld run-on and run-off pieces to be used and subsequently removed, plate edges to be ground flush in direction of stress. - Welded from both sides, checked by NDT. - For misalignment see NOTE 1.
100		2) Full penetration butt joint.	2) - Weld not ground flush - Weld run-on and run-off pieces to be used and subsequently removed, plate edges to be ground flush in direction of stress. - Welded from both sides. - For misalignment see NOTE 1.
100		3) Cruciform joint with full penetration K-butt welds.	3) - Weld toe angle ≤60°. - For misalignment see NOTE 1.
100		4) Non load-carrying fillet welds.	4) - Weld toe angle ≤60°. - See also NOTE 2.
100		5) Bracket ends, ends of longitudinal stiffeners.	5) - Weld toe angle ≤60°. - See also NOTE 2.
100		6) Cover plate ends and similar joints.	6) - Weld toe angle ≤60°. - See also NOTE 2.
90		7) Cruciform joints with load-carrying fillet welds.	7) - Weld toe angle ≤60°. - For misalignment see NOTE 1. - See also NOTE 2.

NOTE 1 Table B.1 does not cover effects of misalignment. They have to be considered explicitly in determination of stress.

NOTE 2 Table B.1 does not cover fatigue initiation from the root followed by propagation through the throat.

EUROPEAN STANDARD
NORME EUROPÉENNE
EUROPÄISCHE NORM

prEN 1993-2 : 2004

12 March 2004

UDC

Descriptors:

English version

Eurocode 3 : Design of steel structures

Part 2 : Steel Bridges

Calcul des structures en acier

Partie 2 : Ponts métalliques

Bemessung und Konstruktion von Stahlbauten

Teil 2 : Stahlbrücken

Stage 34 draft

CEN

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

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

Ref. No. EN 1993-2 : 2004. E

Contents

Foreword

This document (prEN 1993-2: 2004) has been prepared by Technical Committee CEN/TC 250 "Structural Eurocodes", the secretariat of which is held be BSI.

This document is currently submitted to the Formal Vote.

This document will supersede ENV 1993-2.

Background of the Eurocode programme

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

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

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

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

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

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

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

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

Status and field of application of Eurocodes

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

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

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

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

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

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

National Standards implementing Eurocodes

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

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

– values for partial factors and/or classes where alternatives are given in the Eurocode,

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

– geographical and climatic data specific to the Member State, e.g. snow map,

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

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

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

There is a need for consistency between the harmonised technical specifications for construction products and the technical rules for works[4]. Furthermore, all the information accompanying the CE Marking of the

---

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

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

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

construction products which refer to Eurocodes should clearly mention which Nationally Determined Parameters have been taken into account.

Additional information specific to EN 1993-2

EN 1993-2 is the second part of seven parts of EN 1993 – Design of Steel Structures – and describes the principles and application rules for the safety and serviceability and durability of steel structures for bridges.

EN 1993-2 gives design rules in supplement to the generic rules in EN 1993-1.

EN 1993-2 is intended to be used with Eurocodes EN 1990 – Basis of design, EN 1991 – Actions on structures and the parts 2 of EN 1992 to EN 1998 when steel structures or steel components for bridges are referred to.

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

EN 1993-2 is intended for use by

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

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

National annex for EN 1993-2

This standard gives alternative procedures, values and recommendations with notes indicating where national choices may have to be made. The National Standard implementing EN 1993-2 should have a National Annex containing all Nationally Determined Parameters to be used for the design of steel structures to be constructed in the relevant country.

National choice is allowed in EN 1993-2 through:

- 2.1.3.2(1)
- 2.1.3.3(5)
- 2.1.3.4(1)
- 2.1.3.4(2)
- 2.3.1(1) (2 times)
- 3.2.3(2)
- 3.2.3(3)
- 3.2.4(1)
- 3.4(1)
- 3.5(1)
- 3.6(1)
- 3.6(2)
- 4(1)
- 4(4)
- 5.4.1(1)
- 6.1(1)

- 6.2.2.3(1)
- 6.2.2.4(1)
- 7.1(3)
- 7.3(1)
- 8.1.3.2.1(1)
- 8.1.6.3(1)
- 8.2.1.4(1)
- 8.2.1.5(1)
- 8.2.1.6(1)
- 8.2.10(1)
- 8.2.13(1)
- 8.2.14(1)
- 9.1.2(1)
- 9.1.3(1)
- 9.3(1)
- 9.3(2)
- 9.4.1(6)
- 9.5.2(3)
- 9.5.2(4)
- 9.5.2(6)
- 9.5.2(7)
- 9.5.2(8)
- 9.5.3(2)
- 9.6(1)
- 9.7(1)
- A.3.3(1)
- A.3.6(2)
- A.4.2.1(2)
- A.4.2.1(3)
- A.4.2.1(4)
- A.4.2.4(2)
- C.1.1(2)
- C.1.2.2(1)
- C.1.2.2(2)
- E.2(1)

1 General

1.1 Scope

1.1.1 Scope of Eurocode 3

(1) See 1.1.1(1), (2), (3), (4), (5) and (6) of EN 1993-1-1.

1.1.2 Scope of Part 2 of Eurocode 3

(1) EN 1993-2 gives a general basis for the structural design of steel bridges and steel parts of composite bridges. It gives provisions that supplement, modify or supersede the equivalent provisions given in the various parts of EN 1993-1.

(2) The design criteria for composite bridges are covered in EN 1994-2.

(3) The design of high strength cables and related parts are included in EN 1993-1-11.

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

(5) For the execution of steel bridge structures, EN 1090 should be taken into account.

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

(7) Special requirements of seismic design are not covered. Reference shall be made to the requirements given in EN 1998, which complements and modifies the rules of EN 1993-2 specifically for this purpose.

1.2 Normative references

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

(2) For the purpose of this part 2 of EN 1993, in addition to the normative references given in EN 1990 and EN 1993-1 the following references apply:

EN 1337	Structural bearings
EN 25817	...
EN ISO 9013	...
EN 288-3	...
EN 288-8	...
ISO 12944-3	...

1.3 Assumptions

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

1.4 Distinction between principles and application rules

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

1.5 Terms and definitions

(1) In addition to the terns and definitions given in EN 1990 and EN 1993-1, the following termn and definitions apply:

1.5.1
bridges
civil engineering construction works mainly intended to carry traffic or pedestrian loads over a natural obstacle or a communication line; railway bridges and bridges which carry canals, service pipes or other vehicles such as an aircraft are also covered

1.5.2
abutment
any end support of a bridge; a distinction is made between rigid abutments and flexible abutments where relevant

1.5.3
integral abutment
an abutment that is connected to the deck without any movement joint

1.5.4
pier
intermediate support of a bridge, situated under the deck

1.5.5
bearing
structural support located between the superstructure and an abutment or pier of the bridge and transferring loads from the deck to the abutment or pier

1.5.6
cable stay
a tensioned element which connects the deck of a bridge to the pylon or pylons above the deck

1.5.7
prestress
permanent effect due to controlled forces and /or controlled deformations imposed within a structure; various types of prestress are distinguished from each other as relevant (such as prestress by tendons or prestress by imposed deformation of supports)

1.5.8
headroom
the free height available for traffic

1.5.9
breathing (of plates)
out-of-plane deformation of a plate caused by repeated application of in-plane loading

1.5.10
secondary structural elements
structural elements that do not form part of the main structure of the bridge, but are provided for other reasons, such as guard rails, parapets, ladders and access covers

1.6 Symbols

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

Draft note: ... to be inserted later.

1.7 Conventions for member axes

(1) See 1.7(1), (2), (3) and (4) of EN 1993-1-1.

2 Basis of design

2.1 Requirements

2.1.1 Basic requirements

(1) See 2.1.1(1), (2) and (3) of EN 1993-1-1.

2.1.2 Reliability management

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

2.1.3 Design working life, durability and robustness

2.1.3.1 General

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

(2) Bridges shall be designed for fatigue for their design working life.

2.1.3.2 Design working life

(1) The design working life should be taken as the period for which a bridge is required to be used for its intended purpose, with anticipated maintenance but without major repair being necessary.

NOTE 1 The National Annex may specify the design working life. A design working life of a permanent bridge of 100 years is recommended.

NOTE 2 For temporary bridges the design working life may be stated in the project specifications.

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

2.1.3.3 Durability

(1) To ensure durability, bridges and their components should be designed to minimise damage from excessive deformation, deterioration, fatigue and accidental actions that are expected during the design working life, or else protected from them.

(2) Structural parts of a bridge to which guardrails or parapets are connected, should be designed to ensure that plastic deformations of the guardrails or parapets can occur without damaging the structure.

(3) Where a bridge includes components that need to be replaceable, see 4(3), the possibility of their safe replacement should be verified as a transient design situation.

(4) Permanent connections of structural parts of the bridge should preferably be made with preloaded bolts in a Category B or C connection alternatively fit bolts alternatively rivets or welding should be used to prevent slipping.

(5) Joints with transmission of forces by contact may only be used where justified by fatigue assessments.

NOTE The National Annex may give recommendations for durable details for which experiences exist.

2.1.3.4 Robustness and structural integrity

(1) The design of the bridge should ensure that when the damage of a component due to accidental actions occurs, the remaining structure can sustain at least the accidental load combination if possible with reasonable means.

NOTE The National Annex may define components subject to accidental design situations. Examples for such components are hangers, cables, bearings etc.

(2) The effects of deterioration of material, corrosion or fatigue on components should be taken into account by appropriate detailing, see EN 1993-1-9 and Annex C, choice of material, see EN 1993-1-10 and corrosion protection system.

NOTE 1 For design concepts to achieve damage tolerance or safe life for fatigue see section 3 of EN 1993-1-9.

NOTE 2 The National Annex may give a choice of the design concept.

NOTE 3 For accessibility for maintenance and inspection, see 4.

2.2 Principles of limit state design

(1) See 2.2(1) and (2) of EN 1993-1-1.

(3) For damage limitation at the ultimate limit state global analysis models should be elastic for transient and persistent design situations, see 5.4.

(4) Sufficient fatigue life should be achieved by design for fatigue and/or appropriate detailing, see Annex C, and by serviceability checks that substitute particular fatigue checks, see 7.

2.3 Basic variables

2.3.1 Actions and environmental influences

(1) Actions for the design of bridges should be taken from EN 1991. For the combination of actions and partial factors of actions see Annex A.2 to EN 1990.

NOTE 1 For actions on steel bridge decks of road bridges see Annex E.

NOTE 2 For actions not specified in EN 1991 see National Annex.

NOTE 3 The National Annex may also give information on particular transient situations provided for maintenance and repair.

(2) See 2.3(2), (3), (4) and (5) of EN 1993-1-1.

(6) For actions for bearings see Annex A.

2.3.2 Material and product properties

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

2.4 Verification by the partial factor method

(1) See 2.4.1(1), 2.4.2(1) and (2), 2.4.3(1) and 2.4.4(1) of EN 1993-1-1.

2.5 Design assisted by testing

(1) See 2.5(1), (2) and (3) of EN 1993-1-1.

3 Materials

3.1 General

(1) See 3.1(1) and (2) of EN 1993-1-1.

3.2 Structural steel

3.2.1 Material properties

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

3.2.2 Ductility requirements

(1) See 3.2.2(1) and (2) of EN 1993-1-1.

3.2.3 Fracture toughness

(1) The material shall have sufficient material toughness to prevent brittle fracture within the intended design working life of the structure.

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

> **NOTE 1** The lowest service temperature to be adopted in design may be taken from EN 1991-1-5.

> **NOTE 2** The National Annex may specify additional requirements depending on the plate thickness. An examples are given in Table 3.1.

Table 3.1: Example for additional requirements for toughness of base material

Example	Nominal thickness	Additional requirement
1	$t \leq 30$ mm	T_{27J} = -20 °C acc. to EN 10025
	$30 < t \leq 80$ mm	Fine grain steel acc. to EN 10025, e.g. S355N/M
	$t > 80$ mm	Fine grain steel acc. to EN 10025, e.g. S355NL/ML

(3) For bridge components under compressions a suitable minimum toughness property should be selected.

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

3.2.4 Through thickness properties

(1) Steel with improved through thickness properties to EN 10164 should be used where required, see EN 1993-1-10.

NOTE Where Z_E values according to EN 1993-1-10 have been determined, the required Z-class according to EN 10164 may be chosen in the National Annex. The choice in Table 3.2 is recommended.

Table 3.2: Recommendations for choice of quality class according to EN 10164

Target value Z_{Ed}	required value Z_{Rd} according to EN 10164
$Z_{Ed} \leq 10$	–
$10 < Z_{Ed} \leq 20$	Z15
$20 < Z_{Ed} \leq 30$	Z25
$Z_{Ed} > 30$	Z35

3.2.5 Tolerances

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

NOTE For guidance for tolerances for plates and cold formed profiles for stiffeners see Annex C.

(2) For welded components the tolerances in EN 1090 should be applied unless more severe tolerances are required for durability reasons.

NOTE Annex C gives guidance for the structural detailing, tolerances and inspections that comply with the assumptions made for strength, serviceability and durability.

(3) See 3.2.5(3) of EN 1993-1-1.

3.2.6 Design values of material coefficients

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

3.3 Connecting devices

3.3.1 Fasteners

3.3.1.1 Bolts, nuts and washers

(1) Bolts, nuts and washers should conform with Reference Standards of Group 4in 2.8 of EN 1993-1-8, as appropriate.

(2) The rules in this part are applicable to bolts of grades given in Table 3.3.

(3) The nominal values of the yield strength f_{yb} and the ultimate tensile strength f_{ub} are given in Table 3.3 and they should be adopted as characteristic values in calculations.

Table 3.3: Nominal values of the yield strength f_{yb} and the ultimate tensile strength f_{ub} for bolts

Bolt grade	4.6	5.6	6.8	8.8	10.9
f_{yb} (N/mm^2)	240	300	480	640	900
f_{ub} (N/mm^2)	400	500	600	800	1000

3.3.1.2 Preloaded bolts

(1) High strength structural bolts for preloading of bolt grades 8.8 and 10.9 which conform with the requirements for this type of product in Group 4 of the Reference Standards, may be used as preloaded bolts when controlled tightening is carried out in accordance with the relevant requirements of Group 7 in 2.8 of EN 1993-1-8.

3.3.1.3 Rivets

(1) The material properties, dimensions and tolerances of steel rivets should conform with Reference Standards of Group 6 in 2.8 of EN 1993-1-8.

3.3.1.4 Anchor bolts

(1) The following steel grades may be used for anchor bolts:

- Steel grades according to appropriate Reference Standards of Group 1 in 2.8 of EN 1993-1-8;
- Steel grades according to appropriate Reference Standards of Group 4 in 2.8 of EN 1993-1-8;
- Reinforcing bars according to EN 10080,

The nominal yield strength for anchor bolts should not exceed 640 N/mm^2.

3.3.2 Welding consumables

(1) All welding consumables should comply with the Reference Standards of Group 5 in EN 1993-1-8.

(2) The specified yield strength, ultimate tensile strength, elongation at failure and minimum Charpy V-notch energy value of the filler metal, should all be such that the performance of the weld metal should not be less than the corresponding values specified for steel grade being welded.

3.4 Cables and other tension elements

(1) For cables and other tension elements see EN 1993-1-11.

NOTE The National Annex may specify the types of cables complying with the durability requirements for bridges for the specific climate.

3.5 Bearings

(1) Bearings should comply with EN 1337.

NOTE The National Annex may give informations on the types of bearings applicable for bridges.

3.6 Other bridge components

(1) Expansion joints, guardrails, parapets and other ancillary items should comply with the relevant technical specifications.

NOTE The National Annex may give informations on the types of expansion joints, guardrails, parapets and other ancillary items applicable for bridges.

(2) The bridge deck surfacing system, the products used and the method of application should meet with the relevant technical specification.

NOTE The National Annex may give informations on the bridge deck surfacing system, the products used and the method of application relevant for bridges.

4 Durability

(1) See 4(1), (2) and (3) of EN 1993-1-1.

NOTE The National Annex may give information on requirements for accessibility.

(4) For elements that cannot be inspected the fatigue (see EN 1993-1-9) and corrosion allowances should be appropriate.

NOTE The National Annex may give information on sealing against corrosion, measures to ensure airtightness of box girders or extra thickness of inaccessible surfaces.

(5) Sufficient fatigue life of the structure and its components should be achieved by

- fatigue design of details in accordance with (1) and (4) and EN 1993-1-9 and with serviceability checks carried out to section 7;
- structural detailing according to Annex C for steel decks;
- choice of material according to section 3;
- fabrication according to EN 1090.

(6) Components that cannot be designed with sufficient reliability to achieve the total design working life of the bridge should be replaceable. Such parts may include:

- the corrosion protection;
- stays, cables, hangers;
- bearings;
- expansion joints;
- drainage devices;
- guardrails, parapets;
- asphalt layer and other surface protection;
- wind shields;
- noise barriers.

5 Structural analysis

5.1 Structural modelling for analysis

5.1.1 Structural modelling and basic assumptions

(1) See 5.1.1(1), (2) and (3) of EN 1993-1-1.

(4) For the structural modelling and basic assumptions for components of bridges see EN 1993-1.

NOTE For stiffness assumptions of plated components and cables see EN 1993-1-5 and EN 1993-1-11 respectively.

5.1.2 Joint modelling

(1) See 5.1.2(1), (2), (3) and (4) of EN 1993-1-1 and EN 1993-1-8.

(5) For bridges the choice of the type of joint and its modelling should be such that sufficient fatigue life can be verified.

NOTE In general continuous joints are chosen between members of bridges except for bearings or pinned connections or cables, so that the fatigue detail categories of EN 1993-1-9 can be used.

5.1.3 Ground structure interaction

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

NOTE 2 The deformation characteristics of the supports can include the characteristics of the bearings, piers and foundation.

5.2 Global analysis

5.2.1 Effects of deformed geometry of the structure

(1) See 5.2.1(1), (2) and (3) of EN 1993-1-1.

(4) The bridges and its components may be checked with first order theory if the following criterion is satisfied for each section:

$$\alpha_{crit} \geq 10 \tag{5.1}$$

where α_{crit} is defined in 5.2.1(3) of EN 1993-1-1

NOTE The above criterion may be applied to components like arches, longitudinal stiffeners of bridge decks in compression etc.

(5) See 5.2.1(5) and (6) of EN 1993-1-1.

5.2.2 Structural stability of frames

(1) See 5.2.1(1), (2), (3) and (4) of EN 1993-1-1.

(5) For any bridge or its component the behaviour of which is mainly governed by the first buckling mode (single degree of freedom system) the second order effects M_{II} may be calculated by means of applying a factor to the bending moments M_I:

$$M_{II} = M_I \frac{1}{1 - \frac{1}{\alpha_{crit}}} \tag{5.2}$$

(6) See 5.2.1(7) and (8) of EN 1993-1-1.

5.3 Imperfections

5.3.1 Basis

(1) See 5.3.1(1), (2) and (3) of EN 1993-1-1.

5.3.2 Imperfections for global analysis of frames

(1) See 5.3.2(1), (2) and (3) of EN 1993-1-1.

NOTE 1 For piers α_m would be applicable, if cumulative effects from contributions of various piers occur (e.g. for piers forming a frame with the superstructure).

NOTE 2 For the use of member imperfections see also Annex D.

(4) See 5.3.2(6), (7), (8), (10) and (11) of EN 1993-1-1.

5.3.3 Imperfection for analysis of bracing systems

(1) See 5.3.3(1), (2), (3), (4) and (5) of EN 1993-1-1.

5.3.4 Member imperfections

(1) See 5.3.4(1), (2) and (3) of EN 1993-1-1.

5.4 Methods of analysis considering material non-linearities

5.4.1 General

(1) The internal forces and moments should be determined using an elastic analysis for all persistent and transient design situations.

NOTE The National Annex may give information as to when a plastic global analysis may be used for accidental design situations. For plastic global analysis see relevant parts of 5.4 and 5.5 of EN 1993-1-1.

5.4.2 Elastic global analysis

(1) See 5.4.2(1), (2) and (3) of EN 1993-1-1.

(2) If all sections are class 1 it is permitted to ignore the effects of differential temperature, shrinkage and settlement effects at the ultimate limit state.

5.5 Classification of cross sections

5.5.1 Basis

(1) See 5.5.1(1) of EN 1993-1-1.

5.5.2 Classification

(1) See 5.5.2(1), (2), (3), (4), (5), (6), (7), (8), (9) and (10) of EN 1993-1-1.

6 Ultimate limit states

6.1 General

(1) The partial factors γ_M as defined in 2.4.3 of EN 1993-1-1 are applied to the various characteristic values of resistance in this section as follows, see Table 6.1:

Table 6.1: Partial factors

a) resistance of members and cross section:	
– resistance of cross sections to excessive yielding including local buckling	γ_{M0}
– resistance of members to instability assessed by member checks	γ_{M1}
– resistance of cross sections in tension to fracture	γ_{M2}
b) resistance of joints	
– resistance of bolts – resistance of rivets – resistance of pins – resistance of welds – resistance of plates in bearing	γ_{M2}
– slip resistance – for hybrid connections or connections under fatigue loading – for other situations	 γ_{M3} γ_{M3}
– bearing resistance of an injection bolt	γ_{M4}
– resistance of joints in hollow section lattice girders	γ_{M5}
– resistance of pins at serviceability limit state	γ_{M6ser}
– preload of high strength bolts	γ_{M7}

NOTE 1 For the partial factor for the resistance of concrete γ_c see EN 1992.

NOTE 2 The partial factors γ_{Mi} for bridges may be defined in the National Annex. The following numerical values are recommended:

γ_{M0} = 1,00

γ_{M1} = 1,10

γ_{M2} = 1,25

γ_{M3} = 1,10

γ_{M4} = 1,10

γ_{M5} = 1,10

γ_{M6} = 1,00

γ_{M7} = 1,10

6.2 Resistance of cross-sections

6.2.1 General

(1) See 6.2.1(1), (2), (3), (4), (5), (6), (7) and (9) of EN 1993-1-1.

6.2.2 Section properties

6.2.2.1 Gross cross section

(1) See 6.2.1.1(1) of EN 1993-1-1.

6.2.2.2 Net area

(1) See 6.2.2.2(1), (2), (3), (4) and (5) of EN 1993-1-1.

6.2.2.3 Shear lag effects

(1) See 6.2.2.3(1) and (2) of EN 1993-1-1 and 3.2 and 3.3 of EN 1993-1-5.

NOTE The National Annex may give information on the treatment of shear lag effects at the ultimate limit state.

6.2.2.4 Effects of local buckling for class 4 cross sections

(1) The effects of local buckling should be considered by one of the following methods specified in EN 1993-1-5:

1. effective cross section properties of class 4 sections according to section 4
2. limiting the stress levels to achieve cross section properties according to section 10

NOTE The National Annex may give a choice of the method to be used.

6.2.2.5 Effective cross section properties of class 4 sections

(1) See 6.2.2.5(1), (2), (3), (4) and (5) of EN 1993-1-1.

(2) For stress limits of circular hollow sections to obtain class 3 section properties, see EN 1993-1-6.

6.2.3 Tension

(1) See 6.2.3(1), (2), (3), (4) and (5) of EN 1993-1-1.

6.2.4 Compression

(1) See 6.2.4(1) of EN 1993-1-1.

(2) The design resistance of cross sections for uniform compression $N_{c,Rd}$ should be determined as follows:

a) without local buckling:

$$N_{c,Rd} = \frac{A\, f_y}{\gamma_{M0}} \quad \text{for class 1, 2 and 3 cross sections} \tag{6.1}$$

b) with local buckling:

$$N_{c,Rd} = \frac{A_{eff}\, f_y}{\gamma_{M0}} \quad \text{for class 4 cross sections or} \tag{6.2}$$

$$N_{c,Rd} = \frac{A\, \sigma_{limit}}{\gamma_{M0}} \quad \text{for stress limits} \tag{6.3}$$

where $\sigma_{limit} = \rho_x\, f_y / \gamma_{M0}$ is the limiting stress of the weakest part of the cross section in compression (see 10(5) of EN 1993-1-5)

(3) See 6.2.4(3) and (4) of EN 1993-1-1.

6.2.5 Bending moment

(1) See 6.2.5(1) of EN 1993-1-1.

(2) The design resistance for bending about the major axis should be determined as follows:

a) without local buckling:

$$M_{c,Rd} = \frac{W_{pl}\ f_y}{\gamma_{M0}} \quad \text{for class 1 and 2 cross sections} \tag{6.4}$$

$$M_{c,Rd} = \frac{W_{el,min}\ f_y}{\gamma_{M0}} \quad \text{for class 3 cross sections} \tag{6.5}$$

b) with local buckling:

$$M_{c,Rd} = \frac{W_{eff,min}\ f_y}{\gamma_{M0}} \quad \text{for class 4 cross sections or} \tag{6.6}$$

$$M_{c,Rd} = \frac{W_{el,min}\ \sigma_{limit}}{\gamma_{M0}} \quad \text{for stress limits} \tag{6.7}$$

where $W_{el,min}$ and $W_{eff,min}$ are the elastic moduli which correspond to the fibre with the maximum elastic stress

σ_{limit} is the limiting stress of the weakest part of the cross section in compression (see 2.4 of EN 1993-1-5)

(3) See 6.2.5(3), (4), (5) and (6) of EN 1993-1-1.

6.2.6 Shear

(1) See 6.2.6(1), (2), (3), (4), (5), (6), (7) and (8) of EN 1993-1-1 and 5 of EN 1993-1-5.

6.2.7 Torsion

6.2.7.1 General

(1) For members subject to torsion both torsional and distortional effects should be taken into account.

(2) Where the effects of transverse stiffness in the cross section or of diaphragms that are built in to reduce distortional deformations shall be determined, the combined effect of bending, torsion and distortion may be analysed with an appropriate elastic model for the members.

(3) Distortional effects may be disregarded in the member where due to the transverse bending stiffness in the cross section and/or diaphragm action, the effects from distortion do not exceed 10% of the bending effects.

(4) Diaphragms should be designed for the action effects resulting from their load distributing effect.

6.2.7.2 Torsion for which distortional effects may be neglected

(1) See 6.2.7(1), (2), (3), (4), (5), (6), (7), (8), and (9) of EN 1993-1-1.

6.2.8 Bending, axial load, shear and transverse loads

(1) The interaction between bending, axial load, shear and transverse loads may be determined as follows:

1. Interaction methods given in 6.2.8 to 6.2.10.

NOTE For local buckling effects see section 4 to 7 of EN 1993-1-5.

2. Interaction of stresses by the yielding criterion as in 6.2.1

NOTE For local buckling effects see section 10 of EN 1993-1-5.

6.2.9 Bending and shear

(1) See 6.2.8(1), (2), (3), (4), (5) and (6) of EN 1993-1-1.

6.2.10 Bending and axial force

6.2.10.1 Class 1 and class 2 cross sections

(1) See 6.2.9.1(1), (2), (3), (4), (5) and (6) of EN 1993-1-1.

6.2.10.2 Class 3 cross sections

(1) See 6.2.9.2(1) of EN 1993-1-1.

(2) For local buckling consideration the following shall be met

$$\sigma_{x,Ed} \leq \frac{\sigma_{limit}}{\gamma_{M0}} \qquad (6.8)$$

where σ_{limit} should be determined from section 10 of EN 1993-1-5, but the value must not exceed f_y.

6.2.10.3 Class 4 cross sections

(1) See 6.2.9.3(1) and (2) of EN 1993-1-1.

6.2.11 Bending, shear and axial force

(1) See 6.2.10(1), (2) and (3) of EN 1993-1-1.

6.3 Buckling resistance of members

6.3.1 Uniform members in compression

6.3.1.1 Buckling resistance

(1) See 6.3.1.1(1), (2), (3) and (4) of EN 1993-1-1.

6.3.1.2 Buckling curves

(1) See 6.3.1.2(1), (2), (3) and (4) of EN 1993-1-1.

6.3.1.3 Slenderness for flexural buckling

(1) See 6.3.1.3(1) and (2) of EN 1993-1-1.

6.3.1.4 Slenderness for torsional and torsional flexural buckling

(1) See 6.3.1.4(1), (2) and (3) of EN 1993-1-1.

6.3.1.5 Use of class 3 section properties with stress limits

(1) As an alternative using class 4 section properties in (6.48), (6.49), (6.51) and (6.53) of EN 1993-1-1, class 3 section properties according to (6.47), (6.49), (6.50) and (6.52) of EN 1993-1-1 with stress limits in accordance with section 10 of EN 1993-1-5 may be used.

NOTE The σ_{limit} method from section 10 of EN 1993-1-5 is conservative.

6.3.2 Uniform members in bending

6.3.2.1 Buckling resistance

(1) See 6.3.2.1(1), (2), (3) and (4) of EN 1993-1-1.

6.3.2.2 Lateral torsional buckling curves – General case

(1) See 6.3.2.2(1), (2) and (3) of EN 1993-1-1.

(4) Lateral torsional buckling effects may be ignored if the slenderness parameter $\bar{\lambda}_{LT} \leq 0{,}2$ or $\frac{M_{Ed}}{M_{crit}} \leq 0{,}04$.

6.3.3 Uniform members in bending and axial compression

(1) Unless second order analysis is carried out using the imperfections given in 5.3.2 the stability of uniform members subject to axial compression and bending in the plane of buckling should be checked according to section 6.3.3 or 6.3.4 of EN 1993-1-1.

NOTE As a simplification of equation (6.61) in 6.3.3 of EN 1993-1-1 the following equation may be used:

$$\frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}} + \frac{\beta_m \left(M_{y,Ed} + \Delta M_{y,Ed}\right)}{\frac{M_{y,Rk}}{\gamma_{M1}}} \leq 0{,}9 \tag{6.9}$$

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

$M_{y,Ed}$ is the design value of the maximum moment about the y-y axis of the member calculated with first order analysis and without using imperfections

$\Delta M_{y,Ed}$ is the moment due to the shift of the centroidal axis according to 6.2.10.3,

β_m is the equivalent moment factor, see Table A.2 of EN 1993-1-1,

χ_y is the reduction factors due to flexural buckling from 6.3.1

$\bar{\lambda}_y$ is the slenderness parameter of the member in the plane of buckling

6.3.4 General method for lateral and lateral torsional buckling of structural components

6.3.4.1 General method

(1) See 6.3.4(1), (2), (3) and (4) of EN 1993-1-1.

6.3.4.2 Simplified method

(1) See 6.3.2.4(1) of EN 1993-1-1.

(2) Truss chords and flanges in compression subject to lateral buckling may be verified by modelling the elements as a column subject to the compression force N_{Ed} and supported by continuous or discrete elastic springs.

NOTE 1 Guidance on determining spring stiffnesses for U-frames is given in Annex D.2.4.

NOTE 2 Where truss chords and flanges are restrained by U-frame action, the U-frame members and the flanges or chords are loaded by forces induced by such restraint and due to the interaction of the U-frame and the flanges or chords.

(3) The buckling mode and the elastic critical buckling load N_{cr} may be determined from an eigenvalue analysis. If continuous springs are used to represent restraints which are actually discrete the critical buckling load should not be taken as larger than that corresponding to buckling with nodes at the spring locations.

(4) The safety verification may be carried out according to 6.3.2 using

$$\bar{\lambda}_{LT} = \sqrt{\frac{A_{eff}\ f_y}{N_{crit}}} \tag{6.10}$$

where A_{eff} is the effective area of the chord;

N_{crit} is the elastic critical load determined with A_{gross}.

(5) For chords in compression or bottom flanges of continuous girders between rigid supports, the effect of initial imperfections and second order effects on a supporting spring may be taken into account by applying an additional lateral force F_{Ed} at the connection of the chord to the spring:

$$F_{Ed} = \frac{N_{Ed}}{100} \qquad \text{if } \ell_k \leq 1{,}2\,\ell$$

$$F_{Ed} = \frac{\ell}{\ell_k}\frac{N_{Ed}}{80}\frac{1}{1-\frac{N_{Ed}}{N_{crit}}} \qquad \text{if } \ell_k > 1{,}2\,\ell \tag{6.11}$$

with $\ell_k = \pi\sqrt{\frac{EI}{N_{crit}}}$

where ℓ is the distance between the springs.

(6) If the compression force N_{Ed} is constant over the length of the chord, the critical axial load N_{crit} may be calculated from

$$N_{crit} = m\ N_E \tag{6.12}$$

with $N_E = \pi^2 \frac{EI}{L^2}$

$m = \frac{2}{\pi^2}\sqrt{\gamma}$ but not smaller than 1,0.

$\gamma = \frac{c\ L^4}{E\ I}$

$c = \frac{C_d}{\ell}$

where L is the span length between rigid supports;

ℓ is the distance between springs

C_d is the spring stiffness, see (1).

A lateral support to a compressed flange may be assumed to be rigid, if its stiffness satisfies

$$C_d > \frac{4N_E}{L} \tag{6.13}$$

provided that the critical load is determined assuming hinged ends.

(7) The procedure given in (2) to (6) may also be applied to flanges of girders in compression when A_f in (4) is substituted by $A_{eff} + \frac{A_{wc}}{3}$, where A_{wc} is the area of the compression zone of the web. In case of a class 4 section the areas should be taken as effective areas.

(8) For the bottom flange of a continuous girder according to Figure 6.1 and with rigid lateral supports at a distance L the factor m in equation (6.12) may be determined from the minimum of the two following values:

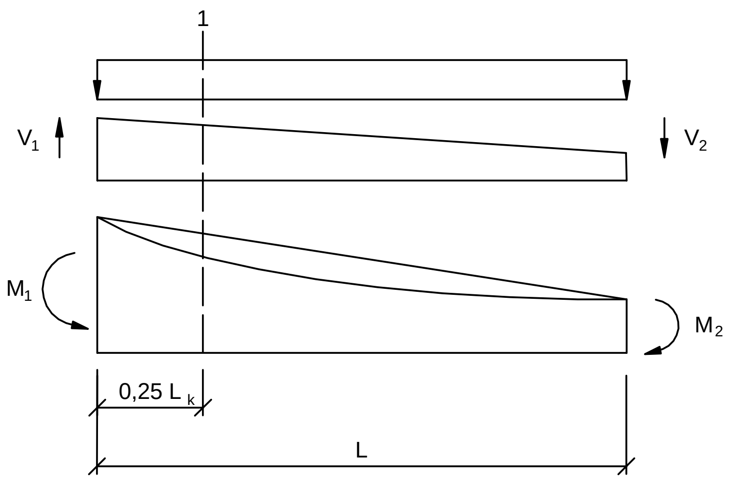
$$m = 1 + 0,44\,(1 + \mu)\,\Phi^{1,5} + (3 + 2\,\Phi)\,\gamma/(350 - 50\mu) \tag{6.14}$$

$$m = 1 + 0,44\,(1 + \mu)\,\Phi^{1,5} + (0,195 + (0,05 + \mu/100)\,\Phi)\,\gamma^{0,5}$$

with $\mu = V_2/V_1$, see Figure 6.1

$\Phi = 2\,(1 - M_2/M_1)/(1 + \mu)$ for $M_2 > 0$

NOTE Where the sign of the bending moment does change, equation (6.14) may be used as a conservative estimate by inserting $M_2 = 0$.



1 design section

Figure 6.1: Segment of beam between rigid lateral supports with bending moment varying as a parabola

(9) The verification of resistance to lateral torsional buckling according to 6.3.2.2 may be done in a design section at a distance 0,25 L_k from the section with the largest moment as shown in Figure 6.1, where $L_k = L/\sqrt{m}$, provided that the crosssectional resistance is also checked at the section with the largest moment. The rule in 6.3.2.2(5) of EN 1993-1-1 is not applicable in this case.

6.4 Built-up compression members

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

6.5 Buckling of plates

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

(2) The plate buckling verification of members at the ultimate limit state should be carried out using one of the following methods:

a) resistances to design direct stresses, shear stresses and transverse forces are determined according to section 4, 5 or 6 respectively of EN 1993-1-5, and combined using the appropriate interaction formulae in section 7 of EN 1993-1-5

b) a resistance is determined on the basis of stress limits governed by local buckling according to section 10 of EN 1993-1-5

NOTE See also 6.2.2.4.

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

7 Serviceability limit states

7.1 General

(1) See 7.1(1), (2) and (3) of EN 1993-1-1.

(2) In general the following serviceability requirements should be taken into account:

a) restriction to elastic behaviour in order to limit:

- excessive yielding, see 7.3(1);
- deviations from the intended geometry by residual deflections, see 7.3(1);
- excessive deformations, see 7.3(4);

b) limitation of deflections and curvature in order to prevent:

- unwanted dynamic impacts due to traffic (combination of deflection and natural frequency limitations), see 7.7 and 7.8;
- infringement of required clearances, see 7.5 or 7.6;
- cracking of surfacing layers, see 7.8;
- damage of drainage, see 7.12;

c) limitation of natural frequencies, see 7.8 and 7.9, in order to:

- exclude vibrations due to traffic or wind which are unacceptable to pedestrians or passengers in cars using the bridge;
- limit fatigue damages caused by resonance phenomena;
- limit excessive noise emission;

d) Restriction of plate slenderness, see 7.4, in order to limit:

- excessive rippling of plates;
- breathing of plates (also in view of fatigue);
- reduction of stiffness due to plate buckling, resulting in an increase of deflection, see EN 1993-1-5;

e) achievement of sufficient durability by appropriate detailing to reduce corrosion and excessive wear, see 7.11;

f) ease of maintenance and repair, see 7.11:

- accessibility of structural parts to permit maintenance, inspection and renewal (of corrosion protection and asphaltic pavements, for example);
- replacement of bearings, anchors, individual cables, expansion joints and the like, that might have a limited service life, with minimum disruption to the use of the structure.

NOTE Some of these serviceability requirements are relevant for all types of bridges, but others are relevant only for specific types such as road bridges, railway bridges or pedestrian bridges.

(3) Normally serviceability aspects may be dealt with in the conceptual design of the bridge, or by suitable detailing. However in appropriate cases, serviceability limit states may be verified by numerical assessment, e.g. for deflections or eigenfrequencies.

NOTE 1 The requirements given above are examples.

NOTE 2 The National Annex may give information on serviceability requirements.

7.2 Calculation models

(1) Stresses at serviceability limit states should be determined from a linear elastic analysis, using appropriate section properties, see EN 1993-1-5.

(2) In modelling the distribution of permanent weight and stiffness in a bridge, the non-uniform distribution resulting from changes in plate thickness, stiffening etc. should be taken into account.

(3) Deflections should be determined by linear elastic analysis, using appropriate section properties, see EN 1993-1-5.

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

7.3 Limitations for stress

(1) The nominal stresses in all elements of the bridge resulting from characteristic load combinations $\sigma_{Ed,ser}$ and $\tau_{Ed,ser}$, calculated making due allowance where relevant for the effects of shear lag in wide flanges and the secondary effects implied by deflections (for instance secondary moments in trusses), should be limited as follows:

$$\sigma_{Ed,ser} \leq \frac{f_y}{\gamma_{M,ser}} \tag{7.1}$$

$$\tau_{Ed,ser} \leq \frac{f_y}{\sqrt{3}\,\gamma_{M,ser}} \tag{7.2}$$

$$\sqrt{\sigma^2_{Ed,ser} + 3\tau^2_{Ed,ser}} \leq \frac{f_y}{\gamma_{M,ser}} \tag{7.3}$$

NOTE 1 Where relevant the above checks should include stresses σ_z from transverse loads, see EN 1993-1-5.

NOTE 2 γ_{Mser} may be chosen in the National Annex. The value γ_{Mser} = 1,00 is recommended.

(2) Local areas of yielding (for instance at the crest of a buckle in a plate) may be usually accepted unless precluded by other provisions of this standard.

(3) The nominal stress range $\Delta\sigma_{fre}$ due to the representative values of variable loads specified for the frequent load combination should be limited to 1,5 $f_y/\gamma_{M,ser}$, see EN 1993-1-9.

(4) For non-preloaded bolted connections subject to shear, the bolt forces due to the characteristic load combination should be limited to:

$$F_{b,Rd,ser} \leq 0{,}7\ F_{b,Rd} \tag{7.4}$$

in which $F_{b,Rd}$ is the bearing resistance for ultimate limit states verifications.

(5) For slip-resistant preloaded bolted connections category B (slip resistant at serviceability, see EN 1993-1-8), the assessment for serviceability shall be carried out using the characteristic load combination.

7.4 Limitation of web breathing

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

(2) Where in road bridges the effects of local and global buckling of plates are taken into account by stress limits according to section 10 of EN 1993-1-5, no further check to avoid excessive breathing is necessary.

(3) For railway bridges and where the effects of local and global buckling of plates are taken into account by the method given in section 4 to 7 of EN 1993-1-5, the procedure given in (4) below should be applied to verify resistance to excessive breathing.

(4) Excessive breathing may be neglected for web panels without longitudinal stiffeners or for subpanels of stiffened webs, where the following criteria are met:

$$b/t \leq 30 + 4{,}0\ L \quad \text{but } b/t \leq 300 \quad \text{for road bridges} \tag{7.5}$$

$$b/t \leq 55 + 3{,}3\ L \quad \text{but } b/t \leq 250 \quad \text{for railway bridges} \tag{7.6}$$

where L is the span length in [m], but not less than 20 m.

(5) If the criterion in (4) is not satisfied the following criterion should be applied:

$$\sqrt{\left(\frac{\sigma_{x,Ed,ser}}{k_\sigma\ \sigma_E}\right)^2 + \left(\frac{1{,}1\ \tau_{x,Ed,ser}}{k_\tau\ \sigma_E}\right)^2} \leq 1{,}1 \tag{7.7}$$

where $\sigma_{x,Ed,ser}$, $\tau_{Ed,ser}$ are the stresses for the frequent load combination. If the stresses are not constant along the length of the panel, see section 10 of EN 1993-1-5.

k_σ, k_τ are the linear elastic buckling coefficients assuming hinged edges of the panel

$$\sigma_E = 190000 \left(\frac{t}{b}\right)^2 \quad [N/mm^2]$$

b_p is the smaller of a and b

7.5 Limits for clearance gauges

(1) Specified clearance gauges shall be maintained without encroachment by any part of the structure under the effects of the characteristic load combination.

7.6 Limits for visual impression

(1) To avoid the visual impression of sagging, consideration should be given to precambering.

(2) In calculating precambering, the effects of shear deformation and slip in riveted or bolted connections should be considered.

(3) For connections with rivets or fitted bolts a fastener slip of 0,2 mm should be assumed. For preloaded bolts no slip need be considered.

7.7 Performance criteria for railway bridges

(1) Specific criteria for deflection and vibrations for railway bridges may be obtained from EN 1991-2.

(2) Any requirements for the limitation of possible noise emission may be given in the project specification.

7.8 Performance criteria for road bridges

7.8.1 General

(1) Excessive deformations should be avoided if they might:

- endanger traffic by excessive transverse slope when the surface is iced;
- affect the dynamic load on the bridge by impact from wheels;
- affect the dynamic behaviour causing discomfort to users;
- lead to cracks in asphaltic surfacings;
- adversely affect the drainage of water from the bridge deck.

NOTE For durability requirements see Annex C.

(2) Calculations of deformations should be carried out using the frequent load combination.

(3) The natural frequencies and deflections of the bridge structure should be limited to avoid discomfort of users.

7.8.2 Deflection limits to avoid excessive impact from traffic

(1) The deck structure should be designed such that its deflection along the length is uniform and there is no abrupt change in stiffness giving rise to impact. Sudden changes in slope of the deck and changes of level at expansion joints should be eliminated. Transverse girders at the end of the bridge should be designed such that the deflection does not exceed:

- the deflection limit specified for the proper functioning of the expansion joint;
- 5 mm under frequent loads unless other limits are specified for the particular type of expansion joint.

NOTE Information on the deflection limit of expansion joints is given in Annex B.

(2) Where the deck structure is irregularly supported (for instance by additional bracings at intermediate bridge piers) the deck area adjacent to these additional deck supports should be designed for the enhanced impact factors given in EN 1991-2 for the area close to expansion joints.

7.8.3 Resonance effects

(1) Mechanical resonance should be taken into account when relevant. Where light bracing members, cable stays or similar items have natural frequencies that are close to the frequency of any mechanical excitation due to regular passage of vehicles over deck joints consideration should be given to increasing the stiffness, or providing artificial damping of the members (by means of oscillation dampers).

NOTE Information on members supporting expansion joints is given in Annex B.

7.9 Performance criteria for pedestrian bridges

(1) For footbridges and cycle track bridges vibrations that might produce discomfort to users should be eliminated either by the structural design or providing suitable damping devices.

7.10 Performance criteria for effects of wind

(1) Vibrations of slender members induced by vortex excitation should be limited to prevent repetitive stresses of sufficient magnitude to cause fatigue.

NOTE Information for the determination of fatigue loads from vortex excitation is given in EN 1991-1-4.

7.11 Accessibility of joint details and surfaces

(1) All steelwork should be designed and detailed to minimise the risk of corrosion and to permit inspection and maintenance, see ISO 12944-3.

(2) All parts should normally be designed to be accessible for inspection, cleaning and painting. Where such access is not provided, either all parts should be effectively sealed against corrosion (for instance the interior of boxes or hollow portions) or they should be constructed in steel with improved atmospheric corrosion resistance. In all cases, if the environment or access provisions are such that corrosion can occur during the life of the bridge, a suitable allowance for this should be made in the proportioning of the parts.

7.12 Drainage

(1) All decks should be waterproofed; the surfaces of carriageways and footpaths should be sealed to prevent the ingress of water.

(2) The layout of the drainage should take into account the slope of the bridge deck, the location, diameter and slope of the pipes.

(3) Free fall drains should carry water to a point clear of the underside of the structure to prevent entering into the structure.

(4) Drainage pipes should be designed so that they can be easily cleaned out. The distance between centres of cleaning openings should be shown on drawings.

(5) Where drainage pipes are used inside box girder bridges, provisions shall be made to prevent accumulation of water during leaks or breakage of pipes.

(6) For road bridges, drains should be provided at expansion joints, on both sides where is appropriate.

(7) For railway bridges up to 40 m long carrying ballasted tracks, the deck may be assumed to be self-draining to abutment drainage systems and no further drainage provisions need be provided along the length of the deck.

(8) Provision should be made for drainage of all closed cross-sections, unless these are fully sealed by welding.

8 Fasteners, welds, connections and joints

8.1 Connections made of bolts, rivets and pins

8.1.1 Categories of bolted connections

8.1.1.1 Shear connections

(1) See 3.4.1(1) of EN 1993-1-8.

8.1.1.2 Tension connections

(1) See 3.4.2(1) of EN 1993-1-8.

8.1.2 Positioning of holes for bolts and rivets

(1) See 3.5(1) and (2) of EN 1993-1-8.

8.1.3 Design resistance of individual fasteners

8.1.3.1 Bolts and rivets

(1) See 3.6.1(1), (2), (3), (4), (5), (6), (7), (8), (9), (10), (11), (12), (13), (14), (15) and (16) of EN 1993-1-8.

8.1.3.2 Injection bolts

8.1.3.2.1 General

(1) See 3.6.2.1(1) and (2) of EN 1993-1-8.

NOTE The National Annex may give information on the use of injection bolts.

8.1.3.2.2 Design resistance

(1) See 3.6.2.2(1), (2), (3), (4), (5) and (6) of EN 1993-1-8.

8.1.4 Groups of fasteners

(1) See 3.7(1) of EN 1993-1-8.

8.1.5 Long joints

(1) See 3.8(1) and (2) of EN 1993-1-8.

8.1.6 Slip resistant connections using 8.8 and 10.9 bolts

8.1.6.1 Slip resistance

(1) See 3.9.1(1) and (2) of EN 1993-1-8.

8.1.6.2 Combined tension and shear

(1) See 3.9.2(1) and (2) of EN 1993-1-8.

8.1.6.3 Hybrid connections

(1) See 3.9.3(1) of EN 1993-1-8.

NOTE The National Annex may give information on the use of hybrid connections.

8.1.7 Deductions for fastener holes

8.1.7.1 General

(1) See 3.10.1(1) of EN 1993-1-8.

8.1.7.2 Design for block tearing

(1) See 3.10.2(1), (2) and (3) of EN 1993-1-8.

8.1.7.3 Angles connected by one leg and other unsymmetrically connected members in tension

(1) See 3.10.3(1) and (2) of EN 1993-1-8.

8.1.7.4 Lug angles

(1) See 3.10.4(1), (2), (3), (4), (5) and (6) of EN 1993-1-8.

8.1.8 Prying forces

(1) See 3.11(1) of EN 1993-1-8.

8.1.9 Distribution of forces between fasteners at the ultimate limit state

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

(2) See 3.12(3) of EN 1993-1-8.

8.1.10 Connections made with pins

8.1.10.1 General

(1) See 3.13.1(1), (2), (3) and (4) of EN 1993-1-8.

8.1.10.2 Design of pins

(1) See 3.13.2(1), (2) and (3) of EN 1993-1-8.

8.2 Welded connections

8.2.1 Geometry and dimensions

8.2.1.1 Type of weld

(1) See 4.3.1(1) and (2) of EN 1993-1-8.

8.2.1.2 Fillet welds

8.2.1.2.1 General

(1) See 4.3.2.1(1), (2), (3), (4), (5) and (6) of EN 1993-1-8.

8.2.1.2.2 Intermittent fillet welds

(1) Intermittent fillet weld shall not be used for bridges, where they would result in the formation of rust pockets.

NOTE Where the connection is protected for weather, e.g. in the interior of box sections, it may be allowed.

8.2.1.3 Fillet welds all round

(1) See 4.3.3(1), (2), (3) and (4) of EN 1993-1-8.

8.2.1.4 Butt welds

(1) See 4.3.4(1), (2) and (3) of EN 1993-1-8.

NOTE The National Annex may give information on the use of partial penetration butt welds for particular design situations.

8.2.1.5 Plug welds

(1) See 4.3.5(1) of EN 1993-1-8.

NOTE The National Annex may give further information for the use of plug welds.

(2) See 4.3.5(2), (3), (4) and (5) of EN 1993-1-8.

8.2.1.6 Flare groove welds

(1) See 4.3.6(1) of EN 1993-1-8.

NOTE The National Annex may give further information for the use of flare groove welds.

8.2.2 Welds with packings

(1) See 4.4(1), (2) and (3) of EN 1993-1-8.

8.2.3 Design resistance of a fillet weld

8.2.3.1 Length of welds

(1) See 4.5.1(1), (2), (3), (4) and (5) of EN 1993-1-8.

8.2.3.2 Effective throat thickness

(1) See 4.5.2(1), (2), (3) and (4) of EN 1993-1-8.

8.2.3.3 Resistance of fillet welds

(1) See 4.5.3(1), (2), (3), (4), (5), (6), (7), and (8) of EN 1993-1-8.

8.2.3.4 Simplified method for resistance of fillet welds

(1) See 4.5.4(1), (2), (3) and (4) of EN 1993-1-8.

8.2.4 Design resistance of fillet welds all round

(1) See 4.6(1) of EN 1993-1-8.

8.2.5 Design resistance of butt welds

8.2.5.1 Full penetration butt welds

(1) See 4.7.1(1) of EN 1993-1-8.

8.2.5.2 Partial penetration butt welds

(1) See 4.7.2(1), (2) and (3) of EN 1993-1-8.

8.2.5.3 T-butt joints

(1) See 4.7.3(1) and (2) of EN 1993-1-8.

8.2.6 Design resistance of plug welds

(1) See 4.8(1) and (2) of EN 1993-1-8.

8.2.7 Distribution of forces

(1) See 4.9(1), (2), (3), (4), (5) and (6) of EN 1993-1-8.

8.2.8 Connections to unstiffened flanges

(1) See 4.10(1), (2), (3), (4), (5) and (6) of EN 1993-1-8.

8.2.9 Long joints

(1) See 4.11(1), (2), (3) and (4) of EN 1993-1-8.

8.2.10 Eccentrically loaded single fillet or single-sided partial penetration butt welds

(1) See 4.12(1) and (2) of EN 1993-1-8.

NOTE The National Annex may give further information for the use of eccentrically loaded single fillet or single sided partial penetration butt welds.

8.2.11 Angles connected by one leg

(1) See 4.13(1), (2) and (3) of EN 1993-1-8.

8.2.12 Welding in cold-formed zones

(1) See 4.14(1) of EN 1993-1-8.

8.2.13 Analysis of structural joints connecting H- and I-sections

(1) For the analysis of structural joints connecting H- and I-sections at ultimate limit state see sections 5 and 6 of EN 1993-1-8.

NOTE The National Annex may give further information for the use of structural joints connecting H- and I-sections.

8.2.14 Hollow section joints

(1) For the analysis of structural joints connecting hollow sections at ultimate limit state see section 7 of EN 1993-1-8.

NOTE The National Annex may give further information for the use of structural joints connecting hollow sections.

9 Fatigue assessment

9.1 General

9.1.1 Requirements for fatigue assessment

(1) Fatigue assessment should be carried out for all critical areas according to EN 1993-1-9.

(2) No fatigue assessment need be carried out for:

– pedestrian bridges, bridges carrying canals or other bridges that are predominantly statically loaded, unless such bridges or parts of them are likely to be excited by wind loads or pedestrians.

– parts of railway or road bridges that are neither stressed by traffic loads nor likely to be excited by wind loads.

9.1.2 Design of road bridges for fatigue

(1) Fatigue checks should be carried out for all bridge components unless the structural detailing complies with standard requirements for durable structures established from experience and from testing.

NOTE The National Annex may give informations on the conditions where no fatigue check is necessary. The following conditions may be used as an example:

No fatigue check need be carried out for two girder bridges made of S235, S275 and S355 and with γ_{Mf} = 1,00:

a) for the following components of the bridge deck, if the structural detailing and the provisions for weld preparation, execution and testing are in conformity with the minimum requirements given in Annex C:

– deckplate

– stiffeners to the deckplate

– stiffener to cross beam connections

b) for the main girders and their attachments, if the structural detailing complies with a minimum fatigue class (e.g. not less than class 71) and a minimum span length (e.g. 45 m).

(2) Fatigue checks should be carried out using the procedure given in this section.

9.1.3 Design of railway bridges for fatigue

(1) Fatigue checks should be performed for all structural elements.

NOTE Elements for which no check is needed may be given in the National Annex.

(2) For the bridge deck the following components should be checked:

1. for decks with longitudinal stiffeners and crossbeams
   - deckplate
   - stiffeners
   - crossbeams
   - stiffener to crossbeam connections
2. for decks with transverse stiffeners only
   - deckplate
   - stiffeners

(2) Critical areas for fatigue checks may be taken from Figure 9.1 and Figure 9.2 (see also Table 9.8).

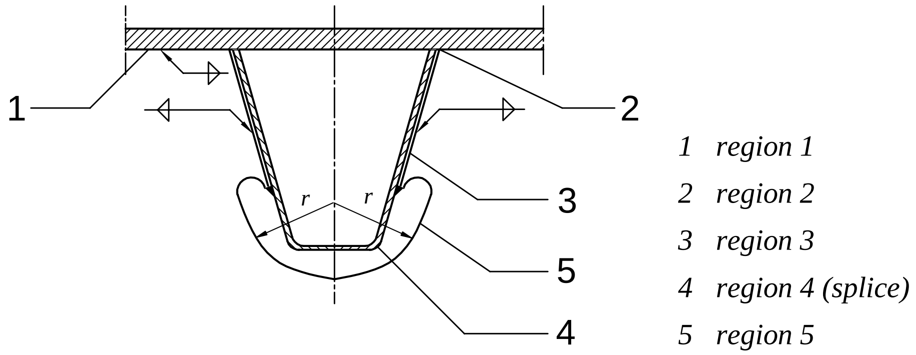


Figure 9.1: Critical regions for fatigue, see also Table 9.8

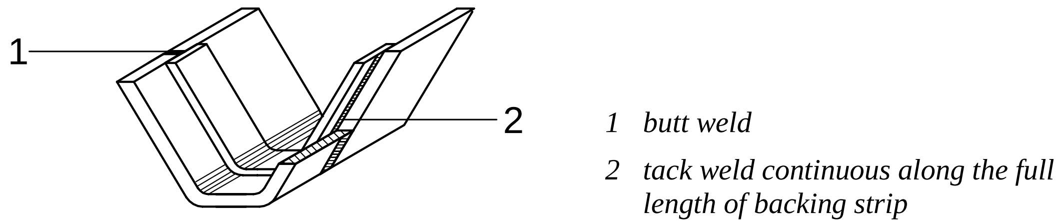


Figure 9.2: Stiffeners with splice plates and metallic backing strips

9.2 Fatigue loading

9.2.1 General

(1) The fatigue loading from traffic should be obtained from EN 1991-2.

(2) The fatigue loads on slender elements from wind excitations should be obtained from EN 1991-1-4.

9.2.2 Simplified fatigue load model for road bridges

(1) For the fatigue check of road bridges the fatigue load model 3 (single vehicle model) in conjunction with the traffic data specified for the bridge location according to EN 1991-2 should be applied.

NOTE See also 9.4.1(6).

9.2.3 Simplified fatigue load model for railway bridges

(1) For the fatigue check of railway bridges the characteristic values for load model 71 should be used, including the dynamic factor Φ_2 according to EN 1991-2.

9.3 Partial factors for fatigue verifications

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

NOTE The National Annex may give the value for γ_{Ff}. The use of γ_{Ff} = 1,0 is recommended.

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

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

9.4 Fatigue stress spectra

9.4.1 General

(1) For the simplified fatigue loading specified in 9.2.2 or 9.2.3, the following procedure may be used to determine the design stress range spectrum.

(2) The maximum stress $\sigma_{P,max}$ and the minimum stress $\sigma_{P,min}$ should be determined for a detail by evaluating influence areas.

(3) The reference stress range $\Delta\sigma_p$ for determining the damage effects of the stress range spectrum should be obtained from:

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

(4) The damage effects of the stress range spectrum may be represented by the damage equivalent stress range related to 2×10^6 cycles:

$$\Delta\sigma_{E2} = \lambda\Phi_2\Delta\sigma_p \qquad (9.2)$$

where λ is the damage equivalence factor as defined in 9.5;

Φ_2 is the damage equivalent impact factor.

(5) For railway bridges the value of Φ_2 should be obtained from EN 1991-2. For road bridges Φ_2 may be taken as equal to 1,0, because it is included in the fatigue load model.

(6) As an alternative to the procedure given above, fatigue stress spectra may be obtained from the evaluation of stress histories from fatigue load vehicles as specified in EN 1991-2, see EN 1993-1-9.

NOTE The National Annex may give informations on the use of EN 1991-2.

9.4.2 Analysis for fatigue

9.4.2.1 Longitudinal stiffeners

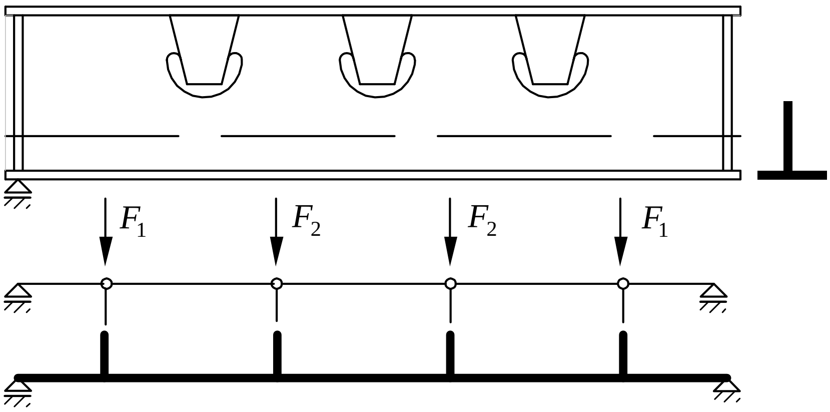
(1) Longitudinal stiffeners should be analysed using a realistic model for the integral structure.

NOTE For railway bridges longitudinal stiffeners may be analysed as continuous beams on elastic supports.

9.4.2.2 Crossbeams

(1) The influence of the cut outs should be taken into account by appropriate modelling.

NOTE Where crossbeams are provided with cut outs as given in Figure 9.3 the action effects may be determined with a Vierendeel-model, where the deckplate and a part of the crossbeam below the cut outs are the flanges and the areas between the cut outs are the posts.



F_i *action on web between cut outs*

Figure 9.3: Vierendeel-model for a crossbeam

(2) In the analysis of this model the following should be taken into account:

1. the connections of the crossbeam to the transverse stiffeners of the webs of main girders that together form a continuous transverse frame,
2. the contributions of the deformations of components of the Vierendeel-beams due to bending moments, axial forces and shear forces to the overall deformation,
3. the effects of shear between the deckplate and the web of the cross beam on the direct stresses and shear stresses at the critical section in Figure 9.4,
4. the superposition of effects of local introduction of loads from the stiffeners into the web,
5. the superposition of the shear stresses from horizontal and vertical shear in the critical section in Figure 9.4.

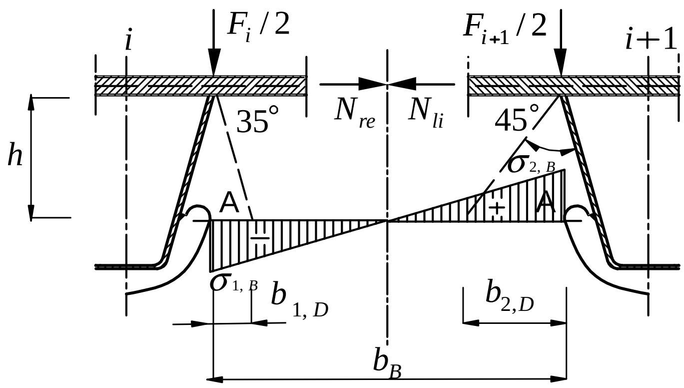


Figure 9.4: Stress distribution at cope hole

(3) The direct stresses in the critical section in Figure 9.4 may be determined as follows:

$$\sigma_1 = \sigma_{1b} + \sigma_{1c} \tag{9.3}$$

$$\sigma_2 = \sigma_{2b} + \sigma_{2c} \tag{9.4}$$

where $-\sigma_{1b} = +\sigma_{2b} = \dfrac{M_{Ed}}{W}$ are the stresses due to bending (9.5)

$\sigma_{1c} = -\dfrac{F_i}{2A_{ic}}$ and $\sigma_{2c} = -\dfrac{F_{i+1}}{2A_{2c}}$ are the compression stresses due to local load introduction (9.6)

and $W = \dfrac{1}{6}\, t\, b_B^2$

$A_{1c} = b_{1c}\ t$

$A_{2c} = b_{2c}\ t$

V_{Ed} is the horizontal shear force

$M_{Ed} = V_{Ed}\ h$ is the bending moment in the critical section

F_i, F_{i+1} are the loads introduced from the stiffeners

(4) Where no cope holes are provided the stresses in the critical section may be determined using flanges from the webs of the stiffeners with an effective width $b_{eff} = 5\ t_{w,st}$.

(5) The bending moments in the welds connecting the deckplate to the stiffeners need not be verified, provided the welds are in accordance with standard requirements for durability.

NOTE For standard requirements see C.1.3.3.

9.5 Fatigue assessment procedures

9.5.1 Fatigue assessment

(1) The fatigue assessment shall be made as follows:

$$\gamma_{Ff}\ \Delta\sigma_{E2} \leq \frac{\Delta\sigma_c}{\gamma_{Mf}} \qquad (9.7)$$

and

$$\gamma_{Ff}\ \Delta\tau_{E2} \leq \frac{\Delta\tau_c}{\gamma_{Mf}} \qquad (9.8)$$

9.5.2 Damage equivalence factors λ for road bridges

(1) The damage equivalence factor λ for road bridges up to 80m span should be obtained from:

$$\lambda = \lambda_1 \times \lambda_2 \times \lambda_3 \times \lambda_4 \qquad \text{but } \lambda \leq \lambda_{max} \qquad (9.9)$$

where λ_1 factor for different types of girder that takes into account the damage effect of traffic and depends on the length of the critical influence line or area;

λ_2 factor that takes into account the traffic volume;

λ_3 factor that takes into account the design life of the bridge;

λ_4 factor that takes into account traffic on other lanes;

λ_{max} maximum λ-value taking account of the fatigue limit, see (8).

(2) Depending on the type of influence line and the geometrical data, the factor λ_1 may be taken as follows, unless a more accurate determination is made:

(3) In determining λ_1 the critical length of the influence line or area may be taken as follows:

a) for moments:

- for a simply supported span, the span length L_i;
- for continuous spans in midspan sections, see Figure 9.7, the span length L_i of the span under consideration;
- for continuous spans in suppport sections, see Figure 9.7, the mean of the two spans L_i and L_j adjacent to that support;
- for cross girders supporting stringers, the sum of the two adjacent spans of the stiffeners carried by the cross girder;

b) for shear for a simply supported span (and as an approximation, for a continuous span):

- for the support section, see Figure 9.7, the span under consideration L_i;
- for the midspan section, see Figure 9.7, 0,4 × the span under consideration L_i;

c) for reactions:

- for end support, the span under consideration L_i;
- for intermediate supports, the sum of the two adjacent spans $L_i + L_j$;

d) in other cases:

- the same as for moments;

e) for arch bridges:

- for hangers, twice the distance of hangers;
- for arch, half the span of the arch.

NOTE The National Annex may give the relevant factors λ_1. The use of the factors λ_1 in Figure 9.5 is recommended.

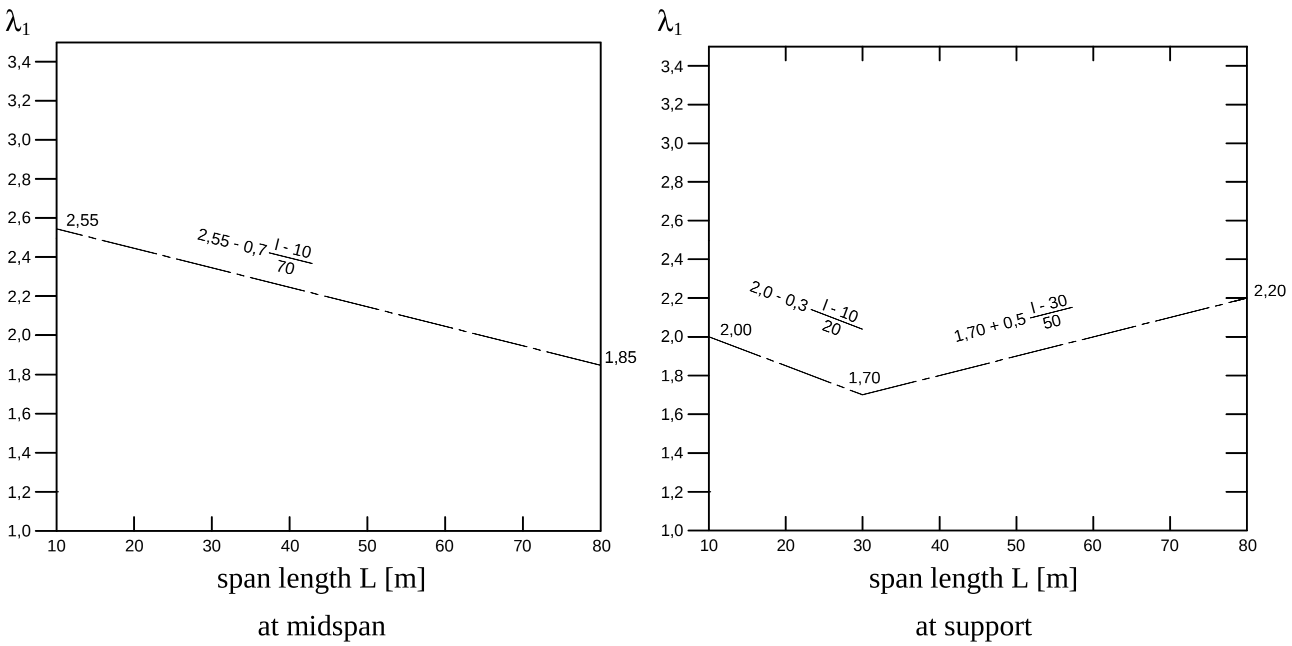


Figure 9.5: λ_1 for moments for road bridges

(4) λ_2 should be calculated as follows:

$$\lambda_2 = \frac{Q_{m1}}{Q_0}\left(\frac{N_{Obs}}{N_0}\right)^{1/5} \tag{9.10}$$

where Q_{m1} is the average gross weight (kN) of the lorries in the slow lane as is obtained from:

$$Q_{m1} = \left(\frac{\sum n_i Q_i^5}{\sum n_i}\right)^{1/5}$$

$Q_0 = 480$ kN

$N_0 = 0{,}5 \times 10^6$

N_{Obs} is the total number of lorries per year in the slow lane, see 9.2.2(2);

Q_i is the gross weight (in kN) of lorry i in the slow lane as specified by the competent authority;

n_i is the number of lorries of gross weight Q_i in the slow lane as specified by the competent authority.

NOTE The National Annex may give informations on λ_2.

(5) For given values of Q_{m1} and N_{Obs}, λ_2 may be obtained from Table 9.1.

Table 9.1: λ_2

Q_{m1}	N_{Obs}							
	$0{,}25\times10^6$	$0{,}50\times10^6$	$0{,}75\times10^6$	$1{,}00\times10^6$	$1{,}25\times10^6$	$1{,}50\times10^6$	$1{,}75\times10^6$	$2{,}00\times10^6$
200	0,362	0,417	0,452	0,479	0,500	0,519	0,535	0,550
300	0,544	0,625	0,678	0,712	0,751	0,779	0,803	0,825
400	0,725	0,833	0,904	0,957	1,001	1,038	1,071	1,100
500	0,907	1,042	1,130	1,197	1,251	1,298	1,338	1,374
600	1,088	1,250	1,356	1,436	1,501	1,557	1,606	1,649

(6) λ_3 should be calculated as follows:

$$\lambda_3 = \left(\frac{t_{Ld}}{100}\right)^{1/5} \tag{9.11}$$

where t_{Ld} is the design life of the bridge in years.

Table 9.2: λ_3

Design life in years	50	60	70	80	90	100	120
Factor λ_3	0,871	0,903	0,931	0,956	0,979	1,00	1,037

NOTE The design life of the bridge t_{Ld} may be specified in the National Annex. The choice of $t_{Ld} = 100$ years is recommended.

(7) λ_4 should be calculated as follows:

$$\lambda_4 = \left[1 + \frac{N_2}{N_1}\left(\frac{\eta_2 Q_{m2}}{\eta_1 Q_{m1}}\right)^5 + \frac{N_3}{N_1}\left(\frac{\eta_3 Q_{m3}}{\eta_1 Q_{m1}}\right)^5 + \cdots + \frac{N_k}{N_1}\left(\frac{\eta_k Q_{mk}}{\eta_1 Q_{m1}}\right)^5\right]^{1/5} \tag{9.12}$$

where k is the number of lanes with heavy traffic;

N_j is the number of lorries per year in lane j;

Q_{mj} is the average gross weight of the lorries in lane j;

η_j is the value of the influence line for the internal force that produces the stress range in the middle of lane j.

NOTE The National Annex may give information on λ_4.

(8) The factor λ_{max} should be obtained from the relevant fatigue stress spectra.

NOTE The National Annex may give the relevant factors λ_{max}. The use of the factor λ_{max} in Figure 9.6 is recommended.

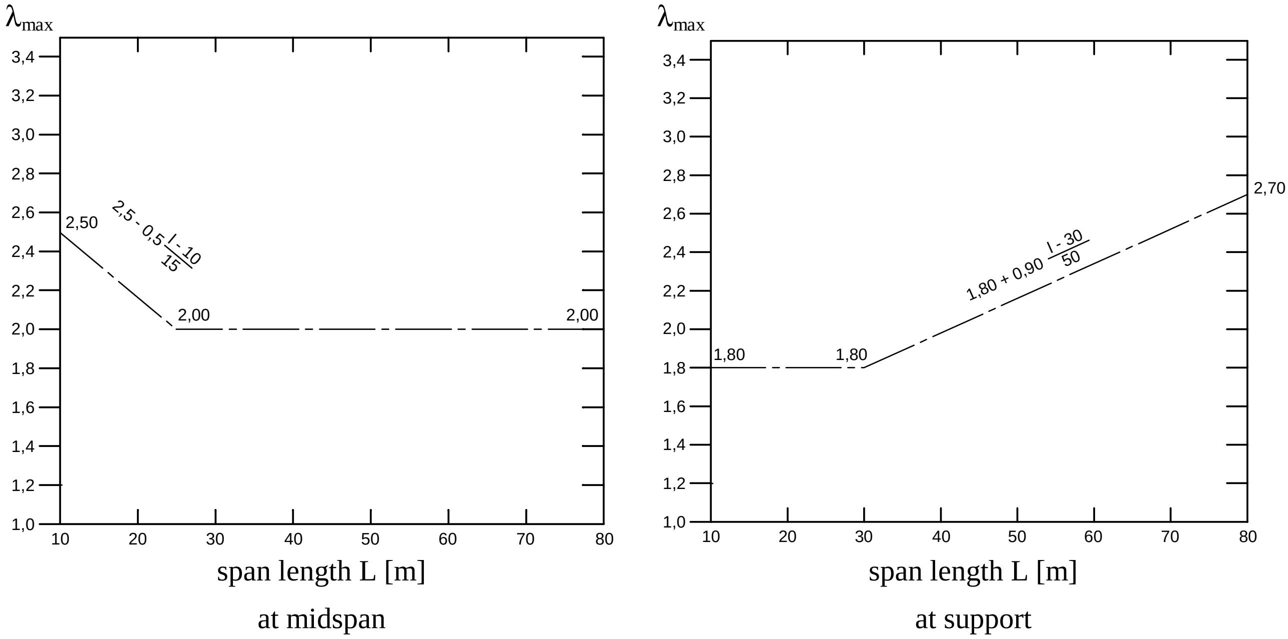


Figure 9.6: λ_{max} for moments for road bridges

9.5.3 Damage equivalence factors λ for railway bridges

(1) The damage equivalence factor λ for railway bridges up to 100m span should be determined as follows:

$$\lambda = \lambda_1 \times \lambda_2 \times \lambda_3 \times \lambda_4 \qquad \text{but } \lambda \leq \lambda_{max} \tag{9.13}$$

where λ_1 factor for different types of girder that takes into account the damage effect of traffic and depends on the length of the influence line or area;

λ_2 factor that takes into account the traffic volume;

λ_3 factor that takes into account the design life of the bridge;

λ_4 factor to be applied when the structural element is loaded by more than one track;

λ_{max} maximum λ value taking account of the fatigue limit, see (9).

(2) λ_1 may be obtained from Table 9.3 and Table 9.4.

NOTE 1 The underlined values in Table 9.3 and Table 9.4 give the envelope of all the train types shown in Annex F of EN 1991-2 (such as freight trains, passenger trains and high-speed trains in any combination) and cover the worst effect for a given span. The values given for mixed traffic correspond to the combination of train types given in Annex F of EN 1991-2.

NOTE 2 λ_1 for express multiple unit, underground and rail traffic with 25 t axles are given in Table 9.4.

NOTE 3 For lines with train type combinations other than those taken into consideration (specialised lines for example), the National Annex may directly specify values of λ_1 as demonstrated in Table 9.3 and Table 9.4.

Table 9.3: λ_1 for standard rail traffic

L	Type 1	Type 2	Type 3	Type 4	Type 5	Type 6	Type 7	Type 8	EC Mix
0,5	1,38	1,27	1,31	1,50	1,62	1,65	**1,69**	1,65	1,60
1,0	1,38	1,27	1,31	1,50	1,62	1,65	**1,69**	1,65	1,60
1,5	1,38	1,27	1,31	1,50	1,62	1,65	**1,69**	1,65	1,60
2,0	1,37	1,26	1,31	1,49	1,35	1,46	1,53	**1,64**	1,46
2,5	1,17	1,23	1,28	1,46	1,29	1,39	1,44	**1,60**	1,38
3,0	1,05	1,19	1,25	1,42	1,25	1,35	1,4	**1,56**	1,35
3,5	0,94	1,02	1,12	1,16	1,12	1,18	1,17	**1,40**	1,17
4,0	0,81	0,82	0,96	1,00	1,15	1,08	1,05	**1,20**	1,07
4,5	0,77	0,73	0,88	0,91	**1,14**	1,07	1,04	0,97	1,02
5,0	0,86	0,69	0,80	0,86	**1,16**	1,07	1,05	0,93	1,03
6,0	0,97	0,63	0,79	0,79	**1,12**	1,07	1,07	0,78	1,03
7,0	0,98	0,57	0,79	0,82	0,96	1,04	**1,07**	0,79	0,97
8,0	0,92	0,55	0,77	0,83	0,85	1,01	**1,06**	0,73	0,92
9,0	0,88	0,56	0,74	0,83	0,77	0,96	**1,05**	0,68	0,88
10,0	0,85	0,56	0,72	0,83	0,66	0,91	**1,04**	0,65	0,85
12,5	0,79	0,55	0,73	0,78	0,52	0,89	**1,00**	0,60	0,82
15,0	0,75	0,56	0,73	0,77	0,51	0,81	**0,91**	0,59	0,76
17,5	**0,74**	0,56	0,73	0,68	0,53	0,72	**0,80**	0,58	0,70
20,0	**0,74**	0,55	0,68	0,66	0,55	0,72	0,70	0,58	0,67
25,0	**0,76**	0,59	0,56	0,58	0,59	0,69	0,68	0,60	0,66
30,0	**0,77**	0,60	0,50	0,53	0,60	0,65	0,69	0,63	0,65
35,0	**0,76**	0,58	0,49	0,51	0,63	0,62	0,68	0,65	0,64
40,0	**0,73**	0,56	0,47	0,50	0,66	0,62	0,68	0,65	0,64
45,0	**0,70**	0,53	0,45	0,49	0,68	0,61	0,68	0,65	0,64
50,0	0,68	0,51	0,43	0,48	**0,70**	0,60	0,69	0,65	0,63
60,0	0,64	0,47	0,41	0,47	**0,73**	0,57	0,68	0,64	0,63
70,0	0,61	0,45	0,40	0,45	**0,75**	0,56	0,67	0,63	0,62
80,0	0,57	0,43	0,38	0,42	**0,76**	0,53	0,67	0,62	0,61
90,0	0,53	0,40	0,36	0,41	**0,77**	0,52	0,67	0,62	0,61
100	0,51	0,38	0,36	0,39	**0,77**	0,51	0,67	0,62	0,60

Table 9.4: λ_1 for express multiple units and underground and for rail traffic with 25 t axles

	Express multiple units and underground		Rail traffic with 25 t axles				
L	Type 9	Type 10	Type 5	Type 6	Type 11	Type 12	25 t Mix
0,5	0,97	1,00	1,62	1,65	1,83	**1,79**	1,65
1,0	0,97	1,00	1,62	1,65	1,83	**1,79**	1,65
1,5	0,97	1,00	1,62	1,65	1,83	**1,79**	1,65
2,0	0,97	0,99	1,35	1,46	1,81	**1,78**	1,64
2,5	0,95	0,97	1,29	1,39	1,56	**1,74**	1,55
3,0	0,85	0,94	1,25	1,35	1,51	**1,69**	1,51
3,5	0,76	0,85	1,12	1,18	1,21	**1,57**	1,31
4,0	0,65	0,71	1,15	1,08	1,04	**1,30**	1,16
4,5	0,59	0,65	**1,14**	1,07	1,05	1,05	1,08
5,0	0,55	0,62	**1,16**	1,07	1,07	1,00	1,07
6,0	0,58	0,63	**1,12**	1,07	1,10	0,87	1,04
7,0	0,58	0,60	0,96	1,04	**1,15**	0,77	1,02
8,0	0,56	0,60	0,85	1,01	**1,14**	0,71	0,99
9,0	0,56	0,55	0,77	0,96	**1,13**	0,67	0,96
10,0	0,56	0,51	0,66	0,91	**1,12**	0,64	0,93
12,5	0,55	0,47	0,52	0,89	**1,07**	0,60	0,90
15,0	0,50	0,44	0,51	0,81	**0,99**	0,59	0,92
17,5	0,46	0,44	0,53	0,72	**0,85**	0,58	0,73
20,0	0,44	0,43	0,55	0,72	**0,76**	0,58	0,68
25,0	0,40	0,41	0,59	0,69	0,67	0,59	0,65
30,0	0,37	0,42	0,60	0,65	0,68	0,62	0,64
35,0	0,36	0,44	0,63	0,62	0,68	0,65	0,65
40,0	0,35	0,46	0,66	0,62	0,68	0,65	0,65
45,0	0,35	0,47	0,68	0,61	0,69	0,65	0,65
50,0	0,36	0,48	**0,70**	0,60	0,70	0,65	0,66
60,0	0,39	0,48	**0,73**	0,57	0,69	0,65	0,66
70,0	0,40	0,49	**0,75**	0,56	0,69	0,65	0,66
80,0	0,39	0,49	**0,76**	0,53	0,70	0,65	0,66
90,0	0,39	0,48	**0,77**	0,52	0,70	0,65	0,66
100,0	0,40	0,48	**0,77**	0,51	0,70	0,65	0,66

(4) In determining λ_1 the critical length of the influence line or area should be taken as follows, unless a more accurate determination is made:

a) for moments:

- for a simply supported span, the span length, L_i;
- for continuous spans, in midspan sections, see Figure 9.7, the span length L_i of the span under consideration;
- for continuous spans in support sections, see Figure 9.7, the mean of the two spans L_i and L_j adjacent to that support;
- for cross-girders supporting rail bearers (or stringers), the sum of the two adjacent spans of the rail-bearers (or stringers) immediately adjacent to the cross-girder;
- for a deck plate supported by only cross-girders or cross-ribs (no longitudinal members) and for those supporting cross-members, the length of the influence line for deflection (ignoring any part indicating upward deflection), taking due account of the stiffness of the rails in load distribution. For cross-members spaced not more than 750 mm apart, this may be taken as 2 × cross-member-spacing + 3 m.

b) for shear for a simply-supported span (and, as an approximation, for a continuous span):

- for the support section, see Figure 9.7 the span under consideration L_i ;
- for the midspan section, see Figure 9.7, 0,4 × the span under consideration L_i ;

c) in other cases:

- the same as for moments.

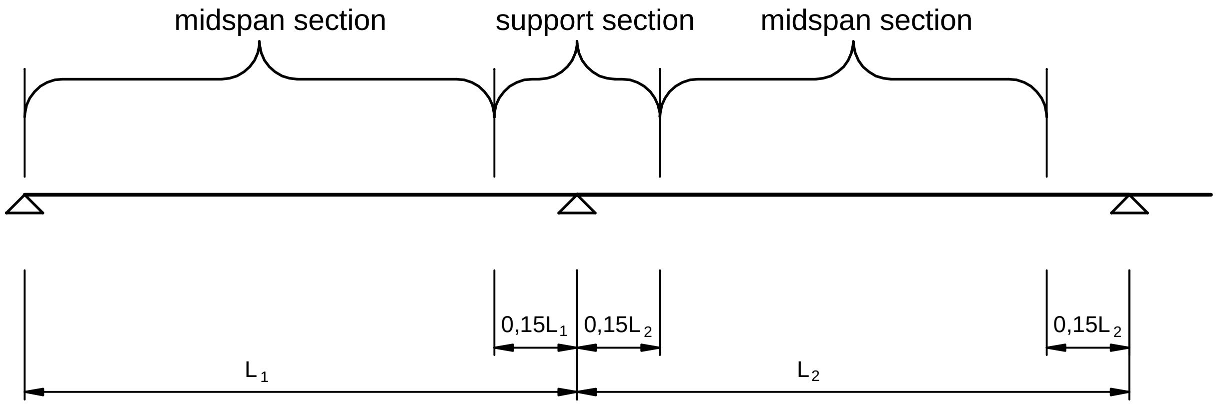


Figure 9.7: Location of midspan or support section

(5) λ_2 should be obtained from Table 9.5.

Table 9.5: λ_2

Traffic per year [10^6 t/track]	5	10	15	20	25	30	35	40	50
λ_2	0,72	0,83	0,90	0,96	1,00	1,04	1,07	1,10	1,15

(6) λ_3 should be obtained from Table 9.6.

Table 9.6: λ_3

Design life [years]	50	60	70	80	90	100	120
λ_3	0,87	0,90	0,93	0,96	0,98	1,00	1,04

(7) λ_4 should be obtained from Table 9.7.

Table 9.7: λ_4

$\Delta\sigma_1/\Delta\sigma_{1+2}$	1,00	0,90	0,80	0,70	0,60	0,50
λ_4	1,00	0,91	0,84	0,77	0,72	0,71

$\Delta\sigma_1$ is the stress range at the section to be checked due to load model 71 on one track;

$\Delta\sigma_{1+2}$ is the stress range at the same section due to load model 71 according to EN 1991-2 on any two tracks.

NOTE Table 9.7 is only valid if $\Delta\sigma_1$ and $\Delta\sigma_{1+2}$ have the same sign.

(8) The values of λ_4 in Table 9.7 assume that 12 % of the total traffic crosses the bridge whilst there is traffic on the other track. The value of λ_4 may be adapted for different proportions of crossing traffic by using:

$$\lambda_4 = \sqrt[5]{n + [1-n]\left[a^5 + (1-a)^5\right]} \tag{9.14}$$

where $a = \Delta\sigma_1/\Delta\sigma_{1+2}$

n is the proportion of the traffic that crosses the bridge with traffic on the other track.

(9) The value of λ should not exceed λ_{max} given by:

$$\lambda_{max} = 1{,}4 \tag{9.15}$$

9.5.4 Combination of damage from local and global stress ranges

(1) Where the stress verification in a member is due to the combined effects of flexure of the bridge (global effects) and flexure of the internal elements (local effects), the combined effects $\Delta\sigma_{E2}$ should be as follows:

$$\Delta\sigma_{E2} = \lambda_{loc} \times \Phi_{loc} \times \Delta\sigma_{loc} + \lambda_{glo} \times \Phi_{glo} \times \Delta\sigma_{glo} \tag{9.16}$$

in which the suffix “loc” refers to local effects and “glo” refers to global effects.

9.6 Fatigue strength

(1) EN 1993-1-9 should be used for the fatigue strength assessment of bridges.

NOTE The National Annex may exclude particular details in EN 1993-1-9 from the design of bridges.

(2) For the critical regions of steel decks, the fatigue categories according to Table 9.8 may be used.

Table 9.8: Detail categories for fatigue assessments

Critical region	Detail	Detail to EN 1993-1-9	Detail category
1	Deckplate stressed longitudinally at transverse fillet welds, see Figure 9.1	Table 8.4 detail 8	71
2	Deckplate stressed longitudinally at welded stringer-to-deckplate connection, see Figure 9.1	Table 8.2 detail 6	100
		Table 8.3 detail 9	80
3	Hollow section stiffener at stiffener-crossbeam connection, see Figure 9.1	Table 8.8 detail 1	80
4	Splice of stiffeners with splice plates and metallic backing strips, see Figure 9.2	Table 8.8 detail 4	71
5	Free edges of cope holes in webs of webs of crossbeams around soffits of stiffeners, see Figure 9.4	Table 8.8 detail 6	112

9.7 Post weld treatment

(1) Where appropriate, weld improvement techniques such as weld toe grinding, TIG remelting of weld toe region, hammer peening, shot peening, may be specified in the project specification.

NOTE The National Annex may give provisions for post weld treatment.

10 Design assisted by testing

10.1 General

(1) Design assisted by testing should be in accordance with EN 1990, supplemented by the additional provisions given in 10.2 and 10.3.

10.2 Types of tests

(1) A distinction should be made between the following types of tests:

a) tests to establish directly the ultimate resistance or serviceability properties of structural parts, for instance tests to develop standardised temporary bridge systems;

b) tests to obtain specific material properties, for instance soil testing in situ or in the laboratory, testing of new materials for coating;

c) tests to reduce uncertainties in parameters in load or resistance models, for instance wind tunnel testing, testing of full size prototypes, testing of small scale models;

d) control tests to check the quality of the delivered products or the consistency of the production characteristics, for instance tests of cables or sockets;

e) tests during execution in order to take account of actual conditions experienced, for instance for measurements of frequencies, or damping;

f) control tests to check the behaviour of the actual structure or of structural elements after completion, for instance proof load tests at the ultimate or serviceability limit states.

(2) For test types (a), (b) and (c), the design values should be obtained from the test results, if these are available at the time of design.

(3) For test types (d), (e) and (f) or cases where the test results are not available at the time of design, the design values should be taken as those that are expected to satisfy the acceptance criteria at a later stage.

10.3 Verification of aerodynamic effects on bridges by testing

(1) Testing should be used to verify the design of a bridge under action of wind where calculation or the use of established results are unable to provide sufficient assurance of the structural safety during either the erection stage or the service life.

(2) Testing should be used to establish:

a) the overall wind environment at the bridge site and at the local wind recording station;

b) the quasi-static drag and lift forces and twisting moments on a bridge or its elements resulting from the flow of wind past them;

c) the amplitude of oscillation developed by the bridge or its elements resulting from its response to the excitation from vortex shedding from alternate sides of the bridge or element in the wind flow (limited amplitude response);

d) the wind speed at which the bridge or an element of it may be liable to a divergent amplitude response (galloping, stall flutter, classical flutter, rain-wind-induced vibration, non-oscillatory divergence, etc);

e) the response of the bridge or its element due to the turbulence in the natural wind;

f) the inherent damping of the structure.

(3) Tests to establish 2(a) to (e) above should normally be carried out in a wind tunnel. Where a design is subject to wind tunnel testing, the models should accurately simulate the external cross-sectional details including non-structural fittings, such as parapets, and should be provided with a representative range of natural frequencies and damping appropriate to the predicted modes of vibration of the bridge. Due consideration should be given to the influence of turbulence and to the effect of wind which is inclined to the horizontal.

(4) Care should be taken that any potential changes in cross-section (for example icing or rivulets of water on a cable) are taken into account in testing.

NOTE The structural damping may be estimated by mechanically exciting the bridge (using, for example, reciprocating machinery, out of balance rotating machinery, rockers, etc.) and either measuring the necessary energy input to generate a particular amplitude of oscillation or measuring the decay of oscillation after the excitation is stopped.

Annex A [normative] – Technical specifications for bearings

A.1 Scope

(1) This annex gives guidance for preparation of technical specifications for bearings, that comply with EN 1337.

NOTE 1 According to EN 1337-1, 3.1.1 bearings are elements allowing rotation between two members of a structure and transmitting the loads defined in the relevant requirements as well as preventing displacements (fixed bearings), allowing displacements in only one direction (guided bearings) or in all directions of a plane (free bearings) as required.

NOTE 2 EN 1337 consists of the following 11 parts dealing with the following types and characteristics:

Part 1: General

- General design rules

Part 2: Sliding elements

- Vertical bearing capacity
- Reaction forces due to friction
- Translation capability
- Eccentricity

Part 3: Elastomeric bearings

- Vertical bearing capacity
- Reaction forces due to horizontal deformations
- Reaction moments due to rotation about the horizontal axes
- Eccentricity

Part 4: Roller bearings

- Vertical bearing capacity
- Reaction forces due to “rolling” friction
- Reaction moment in vertical plane with roller axis
- Horizontal bearing capacity due to friction in direction of roller axis
- Rotation about roller axis
- Eccentricity of roller with respect to top plate and bottom plate 0,5 times the relative eccentricity between the main structures

Part 5: Pot bearings

- Vertical bearing capacity
- Reaction moment in vertical plane
- Wear of seal
- Rotation capacity

Part 6: Rocker bearings

- Vertical bearing capacity
- Horizontal bearing capacity due to friction
- Rotation capacity about one axis

Part 7: Spherical and cylindrical PTFE bearings

- Vertical bearing capacity
- Reaction moment(s) due to friction
- Rotation capacity about all (spherical) axes or one (cylindrical) axis

Part 8: Guided bearings and restraint bearings

- Restraint of movements in one or more directions

Part 9: Protection

Part 10: Inspection and maintenance

Part 11: Transport, Storage and installation

NOTE 3 EN 1337 does not cover (EN 1337-1, 1):

a) bearings that transmit moments as a primary function

b) bearings that resist uplift

c) bearings for moving bridges

d) concrete hinges

e) seismic devices

(2) Technical specifications for bearings include vertical and horizontal forces, translational and rotational movements and other geometrical and performance characteristics, see A.3.1 (3).

A.2 Symbols

(1) Symbols for the most common types of bearings may be taken from EN 1337-1, Table 1.

A.3 General

A.3.1 Support plan

(1) The support plan shall be designed to permit the specified movement of a structure with the minimum possible resistance to such movements.

(2) The arrangement of bearings for a structure shall be considered in conjunction with the design of the structure as a whole. The bearing forces and movements resulting from such consideration shall then be given to the bearing manufacturer to ensure that the bearings provided meet the requirements as closely as possible.

(3) A drawing of the support plan shall be provided as follows using the symbols and nomenclature given in EN 1337-1, Table 1:

a) a simplified general arrangement of the bridge showing the bearings in plan;

b) details at the bearing location (e.g. recess and reinforcement) ;

c) a clear indication of the type of bearing at each location;

d) a table giving the detailed requirements for each bearing;

e) bedding and fixing details.

(4) Bearings should not generally be expected to resist moments due to rotational movement. Where such rotational movement occurs provision should be made to accommodate it by means of the bearing or within the structure. Where bearings are required to resist such rotational movement an analysis should be carried to ensure that the bearings will not be affected adversely, see A.3.2.

(5) Uplift may cause excessive wear in bearings if such conditions occur frequently enough. Where uplift is unavoidable prestressing may be used to provide the necessary additional vertical force.

(6) Bearings and supports shall be designed such that they can be inspected, maintained and replaced if necessary.

> **NOTE 1** For inspection purposes bearings are provided with movement indicators with marking showing the maximum allowable movements.

> **NOTE 2** Resetting or replacement of bearings or parts of bearings require jacking of the structure. The required lift to allow replacement is no more than 10 mm.

(7) Presetting should not be carried out on site. If presetting is required it shall be carried out at the factory. If adjustment on site is unavoidable it shall be carried out in accordance with the manufacturers detailed instructions.

A.3.2 Effects of continuity of deformation

(1) In the design of line rocker and single roller bearings the full implications of uneven pressure along the length of the roller or rocker should be taken into account in the design of the structure and the bearing. Particular care should be taken in the design of the following:

a) structures curved in plan;

b) structures with slender piers;

c) structures without transverse beams;

d) structures with transverse beams where the line rocker or single roller could effectively act as a built-in support for the transverse beam;

e) structures with transverse temperature gradient.

A.3.3 Anchorage of bearings

(1) The design of anchorages of bridge bearings should be carried out using the following criterion. Where the position of a bearing or part of a bearing is maintained completely or partially by friction its safety against sliding shall be checked at the ultimate limit state in accordance with the following

$$V_{Ed} \leq V_{Rd} \qquad \text{(A.1)}$$

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

$$V_{Rd} = \frac{\mu_K}{\gamma_\mu} N_{Ed} + V_{pd}$$

with N_{Ed} minimum design force acting normal to the joint in conjunction with V_{Ed}

V_{pd} design value of shear resistance of any fixing device in accordance with the Eurocodes

μ_K characteristic value of the friction coefficient, see Table A.1

γ_μ partial factor for friction

> **NOTE** γ_μ may be chosen in the National Annex. The following values are recommended.
>
> $\gamma_\mu = 2{,}0$ for steel on steel
>
> $\gamma_\mu = 1{,}2$ for steel on concrete

Table A.1: Characteristic values of the friction coefficient μ_K

Surface treatment of steel components	Steel on steel	Steel on concrete
Uncoated and free from grease Metal-sprayed Coated with fully hardened zinc silicate	0,4	0,6
Other treatment	From test	From test

(2) For dynamically loaded structures the value of N_{Ed} should be determined taking into account any dynamic variations in the load.

(3) For railway bridges and structures subjected to earthquake friction should be taken into account ($N_{Ed} = 0$).

(4) Where holding down bolts or other similar devices are used to provide some of the resistance to horizontal movement it should be demonstrated that this resistance is provided before any movement can take place. If bolts are provided in holes with normal tolerances, movement will inevitably take place before the full resistance to movement is achieved. This is unacceptable at serviceability.

A.3.4 Conditions of installation

(1) Conditions of installation taking account of the construction sequence and other time dependent effects shall be determined and agreed with the manufacturer.

NOTE It is normally difficult to predict the conditions on site at the time of installation and hence to estimate precisely the movement to be considered. It is better, therefore, to base the design on a range of possible assumptions, see A.4.2.

A.3.5 Bearing clearances

(1) Where the bearings are designed to resist to horizontal forces, some movements will take place before clearances are taken up.

(2) The total clearance between extremes of movements may be taken as up to 3 mm unless otherwise specified or agreed with the manufacturer.

(3) Clearance shall not be taken into account in allowing for horizontal movement unless it can be shown that they will be permanently available in the correct direction.

(4) If more than one bearing is required to resist horizontal forces, the bearings and their supports shall be designed to ensure that an adverse distribution of clearance will not prevent this happening and to accommodate the sharing of the load between the bearings caused by any distribution of clearance.

A.3.6 Resistance of bearings to rolling and sliding

(1) The values to be used for calculating the resistance to movement of the various types of bearings may be calculated in accirdance with EN 1337.

NOTE 1 The calculation should allow for the most adverse combination of the permitted variation in material properties, environmental conditions and manufacturing and installation tolerances.

NOTE 2 The properties of some materials (e.g. wear or friction coefficient of PTFE or stress-strain behaviour of elastomers) are only valid for the specified temperature range and the movement speeds as normally occurring in structures. also they are valid when the bearings are protected from harmful substances and sufficiently maintained.

NOTE 3 The actual resistance to movement is likely to be considerably less than the calculated maximum and therefore should not be considered when favourable in the design except as given in (2) below.

(2) Where a number of bearings are so arranged that the adverse forces, resulting from resistance to movement by some are partly relieved by the forces resulting from the resistance to movement by others unless a more precise investigation is carried out, the respective coefficients of friction μ_a and μ_r shall be calculated as follows:

$$\mu_a = 0{,}5\ \mu_{max}\ (1 + \alpha) \tag{A.2}$$

$$\mu_r = 0{,}5\ \mu_{max}\ (1 - \alpha) \tag{A.3}$$

where μ_a is the adverse coefficient of friction

μ_r is the relieving coefficient of friction

μ_{max} is the maximum coefficient of friction for the bearing as given in the relevant Parts of EN 1337

α is a factor dependent on the type of bearing and the number of bearings which are exerting either an adverse or relieving force as appropriate

NOTE The value for α may be chosen in the National Annex. Recommended values are given in Table A.2.

Table A.2: Factors α

n	α
≤ 4	1
$4 < n < 10$	$\frac{16-n}{12}$
≥ 10	0,5

(3) Clause (2) may also be applied to elastomeric bearings where for a bridge these are from different productions. In that case the coefficients of friction in equation (A.2) and (a.3) may be substituted by the respective shear moduli.

A.4 Preparation of the bearing schedule

A.4.1 General

(1) The bearing schedule should ensure that bearings are designed and constructed so that under the influence of all possible actions unfavourable effects of the bearing on the structure are avoided.

(2) The bearing schedule should contain

- a list of forces on the bearings from each action
- a list of movements of the bearings from each action
- other performance characteristics of the bearings

NOTE 1 Forces and movements from the various actions during construction should be appropriate to the construction and inspection scheme including time dependent effects.

NOTE 2 Forces and movements from variable actions should be given as extreme minimum and maximum values corresponding to the relevant load positions

NOTE 3 All forces and movements from actions other than temperature should be given for a specified temperature T_0. The effects of temperature should be determined such that the effects of deviation from the specified temperature T_0 can be identified.

(3) For structures with elastic behaviour all forces and movements should be based on characteristic values of actions to which the relevant partial factors and combination rules should be applied at serviceability, ultimate or durability limit states.

NOTE 1 Guidance for a bearing schedule with characteristic values of bearing reactions and displacements is given in Table A.3. Design values representing the technical specifications for bearing should be derived from this table.

NOTE 2 Normally the most adverse combination of action effects is sufficient for the design of bearings. In special cases greater economy may be achieved by considering the actual coexistent values of action effects.

(4) For structures in which the deformations are significant for action effects second order analysis may be performed in two stages:

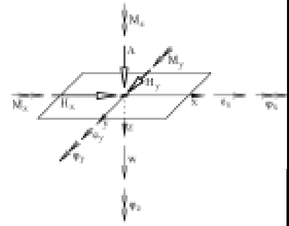
a) for the actions during the various construction phases up to the achievement of the final form of the structure that is required after construction for a specified temperature.

b) for all variable actions imposed on the final form of the structure

NOTE In general there is a requirement for the final geometrical form of the bridge including its bearings after completion of construction for a specified temperature. This is used as reference for determining the necessary measures during construction to achieve this requirement and also for determining forces and movements from variable actions during service taking account of any uncertainties.

Table A.3: Typical bearing schedule

This list comprises all reactions and movements in the final stage. When the bearings are installed during erection, they should be readjusted after reaching the final stage and reactions and movements exceeding those of the final stage shall be give separately.



				Bearing reactions and displacements									**Bearing No.**		
			reaction *)	max A	min A	max H_x	min H_x	max H_y	min H_y	max M_z	min M_z	max M_x	min M_x	max M_y	min M_y
				[kN]	[kN]	[kN]	[kN]	[kN]	[kN]	[kNm]	[kNm]	[kNm]	[kNm]	[kNm]	[kNm]
			displace-ment *)	max w	min w	max e_x	min e_x	max e_y	min e_y	max f_z	min f_z	max f_x	min f_x	max f_y	min f_y
actions (characteristic values)				[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mrad]	[mrad]	[mrad]	[mrad]	[mrad]	[mrad]
1.1	perma-nent G, P	self weight													
1.2		dead load													
1.3		prestressing													
1.4		creep and shrinkage													
2.1	vari-able Q	traffic loads													
2.2		special vehicles	and/or 2.1												
2.3		centrifugal force													
2.4		braking and acceleration forces													
2.5		nosing forces													
2.6		footpath loading													
2.7		wind on structure	w/o 2.1 to 2.6/or 2.8												
2.8		wind on structure and traffic	or 2.7												
2.9		temperature													
2.10		vertical temperature gradient													
2.11		horizontal temperature gradient													
2.12		settlement substructure													
2.13		restraint / friction force													
3.1	seismic	non collapse rupture (ULS)													
3.2		minimisation of damage (SLS)													
4.1	acci-dental A	derailment													
4.2		collision													
4.3		rupture of overhead line													
5.1	combi-nations														
5.2															
5.3															
5.4															
5.5															
...															

*) delete if not applicable	(light grey)	given by the designer of the bridge	(dark grey)	given by the producer of the bearing

A.4.2 Determination of design values of actions on the bearings and movements of the bearings

A.4.2.1 General

(1) In determining the actions on bearings and the movements of the bearing the following reference situation should be used:

a) the construction of the bridge is completed with its final geometrical form for the reference temperature T_0,

b) the fixed bearings are installed and the free bearings have a specified position at their location for the reference temperature T_0,

c) for elastomeric bearings, the position and movements of the bearings at their location comply with the assumptions made for the reference temperature T_0,

d) any uncertainty of position of the bearings at the reference temperature T_0 that may give rise to enlarged movements or restraints to such movements is included in the assumptions for the design values of the reference temperature T_0 and consequently for the design values of the temperature differences ΔT_d^*.

(2) The uncertainty of position of the free bearings in relation to the position of the fixed bearings or in case of elastomeric bearings in relation to the neutral point of movement for both permanent actions at the time of completion of the bridge and a given reference temperature T_0 depends on:

a) the method of installing the bearing;

b) the mean temperature of the bridge when the bearing are installed;

c) the accuracy of measurement of the mean temperature of the bridge, see Figure A.1.

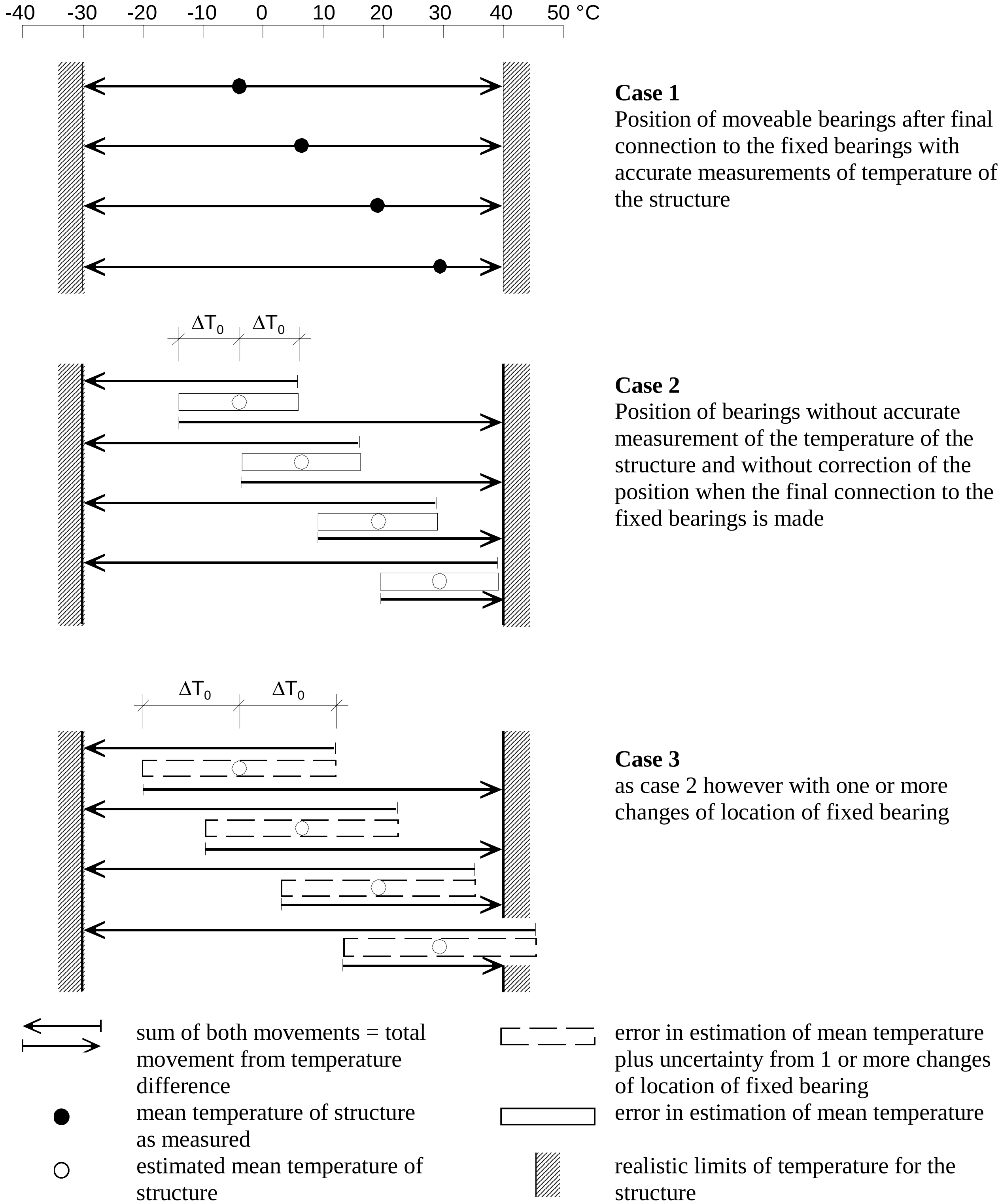


Figure A.1: Determination of ΔT_0 to take uncertainties of position of bearings into account

NOTE The National Annex may give informations on temperature measurements.

(3) The uncertainty of the position of free bearings should be taken into account by assuming an appropriate upper value T_{0max} and a lower value T_{omin} for the installation, that should be determined from

$$T_{omax} = T_0 + \Delta T_0 \tag{A.4}$$

$$T_{omin} = T_0 - \Delta T_0 \tag{A.5}$$

NOTE Δ may be determined in the National Annex. Numerical values for steel bridges as given in Table A.4 are recommended.

Table A.4: Numerical values for ΔT_0

Case	Installation of bearings	ΔT_0 [°C]
1	Installation with measured temperature and with correction by resetting	0
2	Installation with estimated temperature and without correction by resetting with bridge set at $T_0 \pm 10$ °C	15
3	Installation with estimated temperature and without correction by resetting and also one or more changes in the position of the fixed bearing	30

(4) The design values of the temperature difference ΔT_d^* including any uncertainty of the position of the bearings should be determined from

$$\Delta T_d^* = \Delta T_K + \Delta_\gamma + \Delta \tag{A.6}$$

where ΔT_K is the characteristic value of the temperature difference in the bridge according to EN 1991-1-5 relative to the mid point of the temperature range

ΔT_γ is the additional safety term to allow for the temperature difference in the bridge

ΔT_0 is the safety term to take into account the uncertainty of the position of the bearing at the reference temperature.

NOTE 1 The National Annex may specify ΔT_γ and ΔT_0.

NOTE 2 A numerical example for determining ΔT_d^* for case 2 in Table A.4 is:

$T_{Kmin} = -25°C$

$T_{Kmax} = +45°C$

$\Delta T_K = \pm 35$ °C

$T_0 = +10$ °C

$\Delta T_0 = \pm 15$ °C

$\Delta T_\gamma = \pm 5$ °C

$\Delta T_d^* = 35 + 5 + 15 = \pm 55$ °C

NOTE 3 In using ΔT_d^* for bearings with sliding elements or rollers and for elastomeric bearings the design criteria should be appropriate to ultimate limit states and not to serviceability limit state.

(5) Where actions on bearings and movements of bearings are obtained from a non linear global analysis of the structure with the bearings being structural components and incremental calculations are required, the design value of the temperature difference ΔT_d^* may also be expressed in terms of

$$\Delta T_d^* = \gamma_T \, \Delta T_K \tag{A.7}$$

where γ_T is the partial factor for the temperature difference.

NOTE In the case of the example in NOTE 2 of (4) γ_T would take the following values:

case 1 in Table A.4 $\gamma_T = \frac{40}{35} = 1{,}15$

case 2 in Table A.4 $\gamma_T = \frac{55}{35} = 1{,}60$

case 3 in Table A.4 $\gamma_T = \frac{70}{35} = 2{,}00$

(6) For determining the design values of actions on bearings and of movements of bearings the relevant loading combination for the persistent, transient and accidental load combinations shall be taken into account.

A.4.2.2 Actions for persistent design situations

(1) Persistent design situations apply to the bridge after its construction with the required form under permanent actions at the reference temperature T_0.

NOTE For construction see A.4.2.3.1

(2) Where time dependent actions have to be considered these should apply after construction.

(3) The characteristic values of the actions may be taken from the Eurocodes listed in Table A.5, see also Table A.3.

Table A.5: Characteristic values of actions

No.	Action	Eurocode
01 02	reference temperature T_0 temperature difference ΔT_0	EN 1991-1-5, Annex A
1.4	creep $\varepsilon_{K\varphi_K}$ for $\varphi_K = 1{,}35\ \varphi_m$ shrinkage $\varepsilon_{SK} = 1{,}6\ \varepsilon_{sm}$	EN 1992-1 EN 1992-1
2.1	traffic loads	EN 1991-2
2.2	special vehicles	EN 1991-2
2.3	centrifugal forces	EN 1991-2
2.4	brake and acceleration forces	EN 1991-2
2.5	nosing forces	EN 1991-2
2.6	foot path loading	EN 1991-2
2.7	wind on structures	EN 1991-1-4
2.8	wind on structures and traffic	EN 1991-2
2.9	temperature	EN 1991-1-5 6.13 and 6.15
2.10	vertical temperature gradient	EN 1991-1-5 6.14 and 6.15
2.11	horizontal temperature gradient	EN 1991-1-5 6.14 and 6.2
2.12	settlement of substructure	EN 1997-1
2.13	restraint, friction forces	EN 1337 relevant Part i

(4) For the combination of actions see A.4.2.7.

A.4.2.3 Actions for transient design situations

A.4.2.3.1 Design situations during construction

(1) Where bearings are installed before the construction is completed, all relevant construction phases after the instalment of the bearings including any changes of the boundary conditions of the system and all actions during construction should be taken into account in the calculation of movements.

(2) Time dependent actions that develop during the construction phase should be taken into account.

(3) The form of the bridge required at the time of installation of the bearings may be determined from the form required for the bridge after construction at the reference temperature T_0.

(4) The characteristic values of actions may be taken from the Eurocodes listed in Table A.6, see also Table A.3.

Table A.6: Characteristic values of actions

No.	Action	Eurocode
01	reference temperature T_0	EN 1991-1-5 Annex A
02	temperature difference ΔT_0	
1.1	self weight	EN 1991-1-7
1.2	dead load	EN 1991-1-7
1.3	prestressing	
1.4	creep	EN 1992-1
	shrinkage	EN 1992-1
2.2	erection loads	EN 1991-1-7
2.6	variable loads	EN 1991-1-7
2.7	wind on structure	EN 1991-1-4
2.8	wind during works	EN 1991-1-4
2.9	temperature	EN 1991-1-5
2.10	vertical temperature gradient	EN 1991-1-5
2.11	horizontal temperature gradient	EN 1991-1-5
2.12	settlement of substructure	EN 1997-1
2.13	restraint, friction forces	EN 1337 Part i

(5) During construction with launching technique friction forces, effects of the longitudinal slope of the bridge and sway of the piers should be taken into account.

(6) For the combination of actions see A.4.2.7.

A.4.2.3.2 Replacement of bearings and other transient design situations

(1) For transient design situations, the representative values of actions may be reduced according to the limited duration of the situation.

NOTE For transient design situation for traffic see also EN 1991-2.

(2) For the combination of actions see A.4.2.7.

A.4.2.4 Actions for accidental design situations

(1) Accidental design situations may be caused by a number of factors including the following:

- failure of guidance during launching of a bridge,
- failure of a guided or restraint bearing,
- failure of the foundation or pier.

(2) For actions arising from the above failures or for other accidental situations without defined causes the movements and displacements of the bridge should be limited by suitable stops at the abutments or on the piers so that damages are limited and slippages of the bridge or piers are prevented.

NOTE The National Annex may give further informations.

(3) For the design for accidental design situations see EN 1992 to EN 1999.

(4) For the combination of actions see A.4.2.7.

A.4.2.5 Seismic design situations

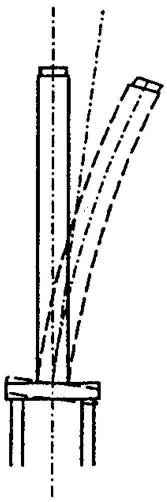
(1) For seismic design situations to determine actions and movements of bearings see EN 1998-1 and EN 1998-2.

(2) For the combination of actions see A.4.2.7.

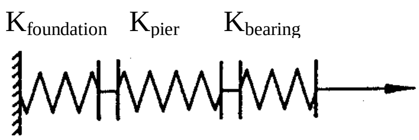
A.4.2.6 Analysis models for determining the movements of bearings

(1) Where the deformations of the foundation, the piers and the bearings have a significant influence on the forces on bearings and the movements of bearings, these elements of the structure should be included in the analysis model.

(2) For linear behaviour the elastic horizontal stiffnesses of the foundations, piers and bearings may be modelled by individual springs, which may be combined to a global spring stiffness at the location of a bearing for the calculation of the movements and restraints to movements for the various actions, see Figure A.2.



spring model



spring stiffness K [MN/m]

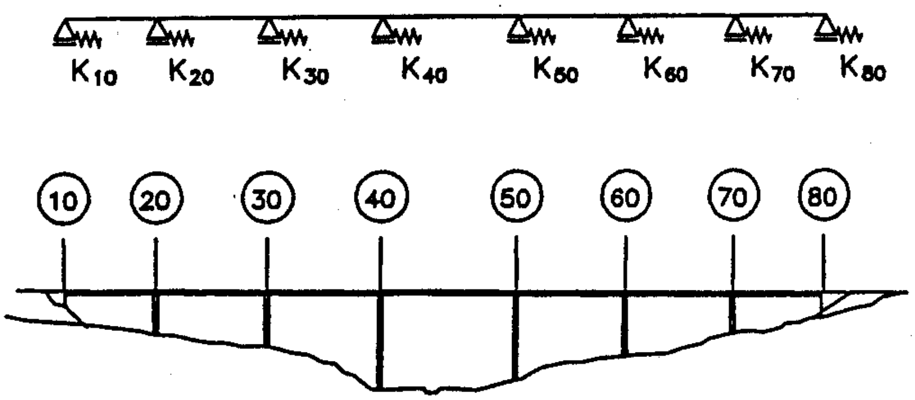
$$\frac{1}{K_{total}} = \frac{1}{K_{foundation}} + \frac{1}{K_{pier}} + \frac{1}{K_{bearing}}$$

displacements of springs v [m/MN]

$$v_{total} = v_{foundation} + v_{pier} + v_{bearing}$$

Figure A.2: Global spring stiffness of pier

(3) The global spring stiffness from all pier stiffnesses in the longitudinal direction of the bridge may be determined from the sum of the stiffnesses of the piers, see Figure A.3.



total spring stiffness K [MN/m]

$$K_{total} = K_{10} + K_{20} + K_{30} + K_{40} + K_{50} + K_{60} + K_{70} + K_{80}$$

Figure A.3: Horizontal spring stiffness from the piers

(4) The effects of eccentricity of springs on the distribution of forces should be taken into account.

A.4.2.7 Combinations of actions

(1) For the combination of actions to determine the design values of forces on bearings and movements of bearings in persistent and transient design situations see 6.4.3.2 of EN 1990.

(2) For the partial factors γ_G, γ_P and γ_Q for permanent and variable actions see Annex A2 of EN 1090.

(3) Where bearings are installed before the construction of the bridge is completed and also the movements of the bearings are checked during construction by measurements the following procedure may be adopted:

1. Actions on and movements of bearings should be determined for all relevant construction phases according to A.4.2.3.1 for the characteristic combination of actions according to 6.5.3(2) of EN 1990. When second order analysis is used the deformations calculated should be referred to the initial form of the structure (form as fabricated without stresses at reference temperature T_0). A comparison of the measured values and the values as calculated should be reported and corrections undertaken where appropriate.

   Ultimate limit state verifications for the bearings and the bridge structure at the points of load introduction from the bearings should follow (1) and (2) with movements of bearings calculated for the characteristic combination of actions.

2. The calculation of forces on and movement of bearings for design values of variable actions that occur after the completion of the bridge should be referred to the geometrical form of the bridge and the geometrical position of the bearings as required and checked after construction of the bridge and for the reference temperature T_0.

   When second order theory is applied, the γ-factors for permanent actions should be combined with the action effects from permanent actions and associated with the required final form of the bridge.

(4) Ultimate limit state verifications for the bearings and the bridge structure at the points of load introduction from the bearings should be performed for the combination of actions according to 6.4.3.2 of EN 1990 and eccentricities based on the calculations in (3).

A.4.3 Determination of the position of bearings at reference temperature T_0

(1) The bearing should be installed so that the temperature expansion and contraction are not markedly different.

(2) Deformation due to creep and shrinkage may be considered to be equivalent to an additional thermal contraction (cooling down).

A.5 Supplementary rules for particular types of bearings

A.5.1 Sliding elements

(1) The load introduction to the bearing should be proportioned such that the deformation limits of the backing plates of sliding elements see EN 1337-2, 6.9 are not exceeded.

A.5.2 Elastomeric bearings

(1) Forces, moments and deformations exerted on the structure from elastomeric bearings can be determined using the stiffness parameters given in EN 1337-3, 5.3.3.7.

A.5.3 Roller bearings

(1) The eccentricity due to relative movement of top and bottom roller plates may be increased by eccentricities due to roller friction and from rotational elements in case of multiple rollers.

(2) For eccentricity in the transverse direction see A.3.2 and A.4.3(2).

A.5.4 Pot bearings

(1) Unless the relevant class of accumulated slide path of internal seal systems is otherwise specified, the following procedure may be used to determine the class.

NOTE The class of accumulated slide path of internal seal is related to testing for durability.

(2) It shall be verified that

$$S_d \leq S_T \tag{A.8}$$

where S_d is the required accumulated slide path due to variable loads

S_T is the accumulated slide path capacity in accordance with EN 1337-5, 5.4 or from testing according to Annex E of EN 1337-5.

(3) S_d may be determined from

$$S_d = \frac{D}{2\,c} \sum_i n_i \Delta\varphi_{2i} \tag{A.9}$$

where n_i is the number of load events associated with the effects $\Delta\varphi_{2i}$

$\Delta\varphi_2 = \varphi_{2max} - \varphi_{2min}$ is the range of rotation angles from extreme positions of the characteristic loads

D is the internal diameter of pot in millimeters

c is a factor to correct for the difference between constant amplitude slide path used in tests and the actual effects of variable amplitude movements.

NOTE Unless otherwise moments determined c may be taken as c = 5.

(4) For restraint moments due to rotation of elastomeric pad and internal seal friction see EN 1337-5, 6.13.

A.5.5 Rocker bearings

(1) For line and point rocker rotational eccentricities see EN 1337-6, 6.6.

A.5.6 Spherical and cylindrical PTFE bearings

(1) For maximum deformations of backing plates see A.5.1.

(2) For eccentricities due to friction, rotation and lateral forces see EN 1337-7, Annex A.

A.5.7 Details of installation

(1) Where structural components for load introduction from the bearings are not cast in situ directly on the bearing subsequent to its installation e.g. in case of precast concrete or steel members, appropriate measures shall be taken:

- to ensure their uniform contact with the bearing,
- to avoid areas of variable rigidity on or underneath the bearing.

(2) Level corrections should be effected by grouting or suitable packing by plates with machined surfaces.

(3) More details are given in EN 1337-11.

Annex B [normative] – Technical specifications for expansion joints for road bridges

B.1 Scope

(1) This annex gives guidance for the preparation of technical specifications for a selected expansion joint for road bridges in order to verify the suitability for the purpose.

(2) Technical specifications for the manufacturer of expansion joints include

- special movements (translational and rotational) from temperature, creeping, shrinkage, traffic, and setting if relevant;
- traffic categories, other actions and environmental influences;
- the type of expansion joint and the related ETA;
- dimensions in sections and plan and categories of use (vehicles, cycles, pedestrians);
- particular requirements concerning durability, maintenance, accessibility and replacement, drainage, water tightness, noise emission.

(3) Data needed for the design of the connection of the expansion joint to the supporting structure of the bridge are those given in the relevant ETA and as supplied by the manufacturer for the specific project. They include:

- dimensions including tolerances, movement capacities and other requirements for connections, the anchorage and for installation;
- minimum requirements for stiffness of the main structure supporting the expansion joint;
- recommended detailing of the connection to the bridge;
- forces and moments from imposed movements to be taken into account in the bridge design.

NOTE 1 The following families of expansion joints are distinguished in the “Guideline for European Technical Approval of Expansion Joints for Road Bridges” (ETAG).

Table B.1: Types of expansion joints

ETAG -Part	Type
2	**Buried expansion joint:** This expansion joint is formed in situ using components such as waterproofing membranes or an elastomeric pad, to distribute the deformations to a greater width and to support the surfacing which is continuous over the deck joint gap. The components of the expansion joint are non flush with the running surface.
3	**Flexible expansion joint:** An in situ poured joint comprising a band of specially formulated flexible material (binder and aggregates), which also forms the surfacing, supported over the deck joint gap by thin metal plates or other suitable components. The joint material is flush with the running surface.
4	**Nosing expansion joint:** This expansion joint has lips or edges prepared with concrete, resin mortar or elastomeric. The gap between the edges is filled by a prefabricated flexible profile, which is non flush with the running surface.
5	**Mat expansion joint:** This expansion joint uses the elastic properties of a prefabricated elastomeric strip or pad to allow the expected movements of the structure. The strip is fixed by e.g. bolts to the structure. The joint element is flush with the running surface.
6	**Cantilever expansion joint:** This expansion joint consists of cantilever symmetrical and non-symmetrical elements (such as comb or saw tooth plates), which are anchored on one side of the deck joint gap and interpenetrated to bridge the deck joint gap. The elements are flush with the running surface.
7	**Supported expansion joints:** This expansion joint consists of one element flush with the running surface, which is fixed by hinges on one side and sliding supports on the other side (by a second element), and which spans the deck joint gap. The expected structural movement is allowed through sliding on the non fixed side of the hinged element i.e. on the supporting element, that is anchored to the substructure.
8	**Modular expansion joint:** This expansion joint consists in a succession of watertight elements (in the traffic direction) comprising movement controlled metal beams supported by moveable substructures bridging that structural gap (i.e. cross beams, cantilevers, pantographs etc.). The metal beams are flush with the running surface.

NOTE 2 The ETAG on Expansion joints for road bridges does not cover movable bridges.

NOTE 3 Expansion joints are normally installed by the manufacturer of the expansion joints or under the supervision of the manufacturer.

B.2 Technical specifications

B.2.1 General

(1) Expansion joints for bridges shall be approved in accordance with the “Guideline for the Technical Approval of Expansion Joints for Road Bridges”.

(2) The technical specifications for expansion joints for a specific bridge project should be determined for the actions on the bridge and from the bridge response to these actions.

NOTE For the actions, combinations of actions and the modelling of the bridge structure to determine bridge responses relevant for expansion joints see also Annex A – Technical specification for bearings.

(3) For drafting technical specifications the guideline for the preparation of an expansion joint schedule in B.2.2 should be used.

B.2.2 Expansion joint schedule

(1) An expansion joint schedule shall give all the relevant informations necessary for the design of the expansion joint, including the following:

1. Geometric data for the bridge deck surface and arrangement of the expansion joint in plan and section views. The data should include provisions for final alignment and for the durability of the load carrying connection between the expansion joint and the bridge structure (e.g. infill of fibre reinforced concrete). They should demonstrate the accessibility of movable parts from below and their protection against corrosion and dirt.
2. User categories:

– vehicle

– cyclist

– pedestrian

Pedestrian paths should include maintenance vehicles, snow-ploughs etc. Gaps and voids should be covered in a way that no accidents may occur.

3. Arrangement of the expansion joints in conjunction with the geometry of the bridge, e.g. longitudinal and transverse slope and curvature and the arrangement of bearings and directions of their displacements.
4. List of actions on expansion joints (including standard and possibly optional actions as accidental actions) comprising:

– imposed displacements and rotations from the bridge movements related to the installation temperature in all directions from the individual characteristic values of any transient, accidental and seismic actions on the bridge. For accidental and seismic actions the limit relating to opening or closing movements should be indicated;

– imposed direct loads from user categories, vertical loads, horizontal loads for ultimate, serviceability and fatigue limit states;

– environmental conditions that may affect the properties of the constituent materials.

5. Installation plan containing:

– information about the prefixing (gap) of the expansion joint and its marking (considering the deformation of the structure at the time of installation from creeping, shrinkage, setting and the temperature assumed, e.g. + 10 °C);

– requirements for adjustment measures to cope for differences to assumptions (e.g. movements for ΔT = 1 °C) in form of a diagram,

– temporary abutments and final abutments;

– time of unfastening;

– time of concreting.

6. Other requirements such as

– sectioning for erection (site connections) and for maintenance and repair;

– provisions for anchorage and connections and also colliding elements (road restraint systems);

– tightness for debris, dust, water;

– accessibility to the joint structure or the drainage system;

– design life according to traffic categories in Table 4.5 of EN 1991-2;

– connection to the waterproofing system of the deck;

– noise emission.

B.2.2 Actions for the design of the joint anchorage and connections

(1) The information needed for the design of the anchorage or connections of the expansion joint from the joint manufacturer is as follows:

1. Geometric data for the bearing surfaces for the expansion joint components incl. tolerances and types of connections foreseen for installation;
2. Minimum stiffness of the bearing surfaces (e.g. due to maximum deflections);
3. Characteristic values of forces and moments to be transmitted to the bridge structure.

B.3 Imposed loads and displacements and rotations from bridge movements

(1) The design values of displacements and rotations at the location of the expansion joints should be prepared according to the rules specified in A.4.2 of Annex A – Technical specifications for bearings.

(2) In the determination of displacements and rotations the following aspects should be taken into account:

1. relative displacements and rotations at both ends of the joint;
2. angles between longitudinal slope and transverse slope of bridge surface and direction of movement of the movable bearings;
3. effects of eccentricities;
4. allowance for lifting the bridge for replacing the bearings (e.g. by 10 mm).

Annex C [informative] – Recommendations for the structural detailing of steel bridge decks

C.1 Highway bridges

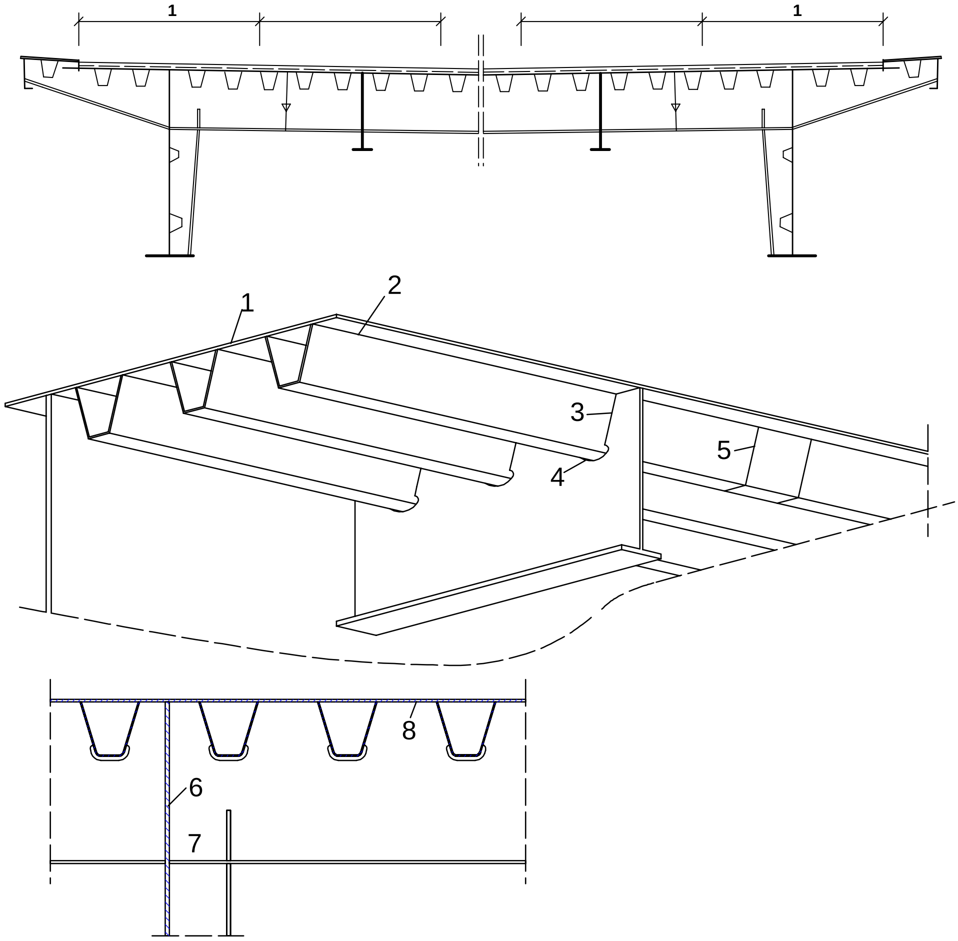
C.1.1 General

(1) This annex gives recommendations for the structural detailing and execution for road bridges to achieve a minimum quality standard as assumed in EN 1993-1-9,

NOTE This annex applies to the types of details described in the following figures only.

(2) The recommendations are based on a standard design as given in Figure C.1 aiming at both durability of the steel structure and of the surfacing. It is assumed that for the surfacing appropriate requirements for bonding, make up of the surfacing (material), plate preparation and waterproofing are met.

NOTE For technical information see National Annex.



1 heavy traffic lane

1 deckplate
2 welded connection of stiffener to deck plate
3 welded connection of stiffener to web of cross beam
4 cut out in web of cross beam
5 splice of stiffener
6 splice of cross beam
7 welded connection of cross beam to main girder or transverse frame
8 welded connection of the web of cross beam to the deck plate

Figure C.1: Structural details in steel decks of highway bridges

NOTE The recommendations do not apply to decks provided with transverse stiffeners.

(3) The recommendations are based on the lanes in the carriageway carrying heavy traffic and cover:

1. the deck plate,
2. the welded connections of the stiffeners to the deck plate,
3. the welded connections of the stiffeners to the web of the cross beam,

4. the detail of the cut out in the web of the cross beam,
5. the splice of the stiffeners,
6. splices of cross beams,
7. the connection between cross beams and main girders.

(4) Details of tolerances, testing methods and amount of testing and test results required are given in Tables C.3, Table C.4 and Table C.5.

C.1.2 Deck plate

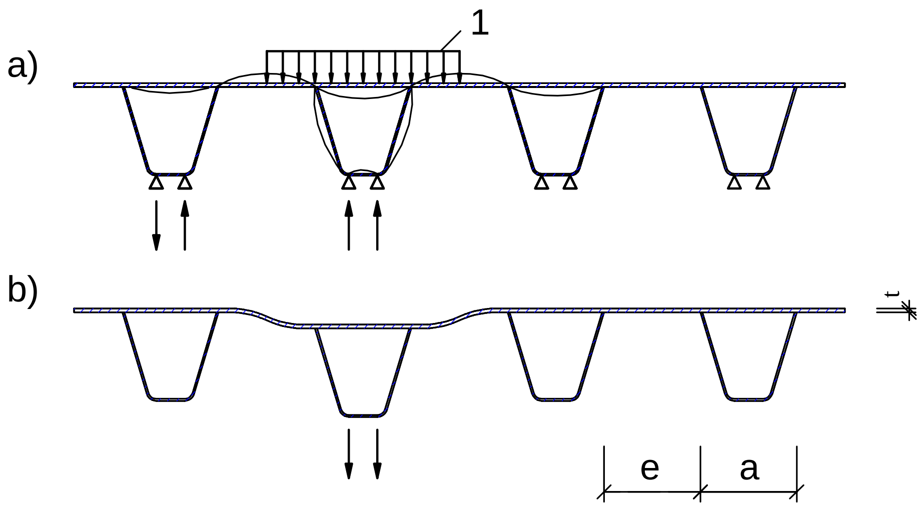
C.1.2.1 General

(1) Fatigue actions originate from bending of the deck plate due to wheel loads and tyre pressures, see Figure C.2.

(2) Figure C.2 a) shows the bending assuming the stiffeners would not deflect. Figure C.2 b) shows the effect of differential deflections of stiffeners.

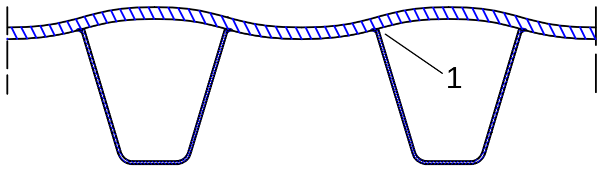
(3) The combination of the deck plate with the surfacing leads to an increase of the stiffness of the plate due to composite actions.

(4) Fatigue cracks may occur in the welds between the stiffeners and the plate, see Figure C.3, and in the surfacing.

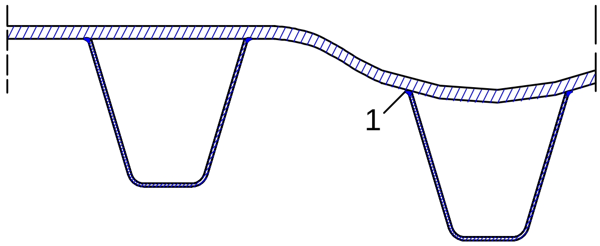


1 wheel load

Figure C.2: Effect of a) local wheel loads and b) differential deflections of stiffeners



a) crack initiation starting at weld root inside the stiffeners



b) crack initiation starting at weld toe outside the stiffeners

1 crack initiation

Figure C.3: Fatigue cracks in deck plate

(5) The recommendations refer to

1. the minimum thickness of the deck plate and the minimum stiffness of stiffeners
2. the splices of the deck plate
3. the connections between the deck plate and webs of main girders, webs of open section stiffeners and webs of cross beams.

(6) The connection between the deck plate and the webs of the stiffeners is treated in section C.1.3.

(7) In order to achieve the tolerances for the assembly of the deck plate as specified in Table C.4 tolerances given in Table C.3 (1) should be met.

C.1.2.2 Deck plate and minimum stiffness of stiffeners

(1) The thickness of deck plate should be selected according to the traffic category, the effects of composite action of the deckplate with the surfacing and the spacing of the supports of the deckplate by webs of stiffeners.

NOTE 1 The National Annex may give informations on the plate thickness to be used. Recommended plate dimensions are as follows, see Figure C.2:

1. Deck plate thickness in the carriage way in the heavy vehicle lane

   $t \geq 14$ mm for asphalt layer ≥ 70 mm,

   $t \geq 16$ mm for asphalt layer > 40 mm.

2. Spacing of the supports of the deck plate by webs of stiffeners in the carriageway

   $e/t \leq 25$, recommended $e \leq 300$ mm.

   Locally e may be increased by 5 % where required, e.g. for adaptation to bridge curvature in plan.

3. Deck plate thickness for pedestrian bridges:

   $t \geq 10$ mm and $e/t \leq 40$

   $e \leq 600$ mm.

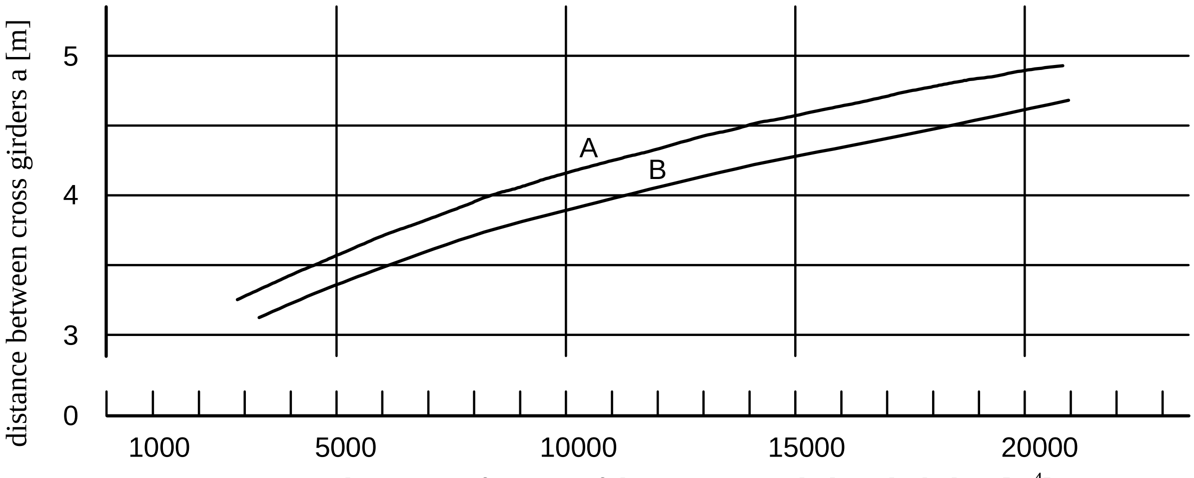
4. Thickness of stiffener:

   $t_{stiff} \geq 6$ mm

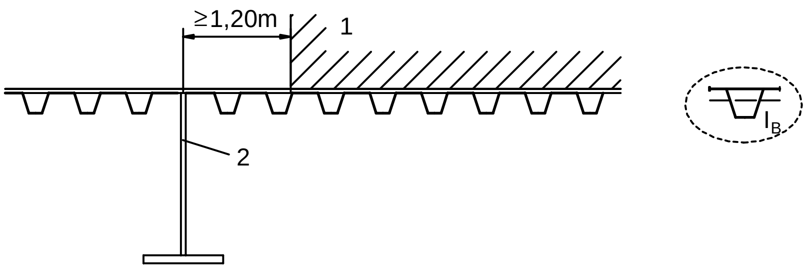
NOTE 2 When the recommendations 1, 2, 3, 4, are satisfied, the bending moments in the deck plate need not be verified.

(2) The minimum stiffness of stiffeners should be selected according to the traffic category and the location of the stiff bearing from webs of main girders or longitudinal girders in relation to the lane carrying heavy traffic to prevent cracking of the surfacing due to differential deflections.

NOTE The National Annex may give informations on the minimum stiffness of stiffeners. The minimum stiffnesses in Figure C.4 are recommended.



Condition for curve A



1 heavy traffic lane

2 web of main girder or longitudinal girder

NOTE
a) Curve A applies to all stiffeners, that are not covered by b).
b) Curve B applies to stiffeners that are located under the most heavily loaded traffic lane within 1,20 m of a web of a main girder
c) The figure applies to all types of stiffeners

Figure C.4: Minimum stiffness of longitudinal stiffeners

C.1.2.3 Splices of deck plates

(1) Transverse splices (with weld running across the traffic lane) should be double V-welds or single V-weld with root run or capping run or single V-weld with ceramic backing strips. Splices with metallic backing strips, see Figure C.6, are not recommended because of the crossing with the stiffeners.

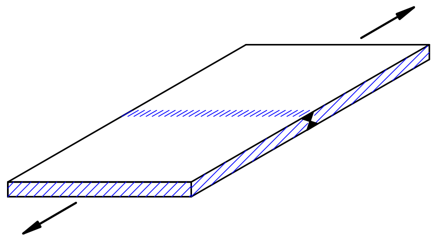


Figure C.5: Splice of deck plate transverse to traffic lane without metallic backing strip

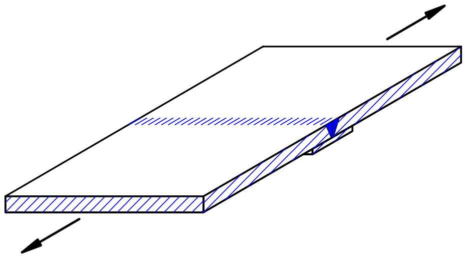
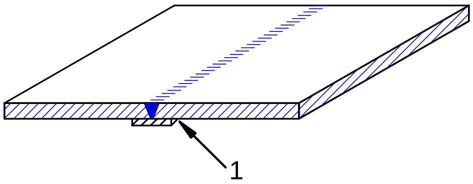


Figure C.6: Splice of deck plate transverse to traffic lane with ceramic backing strip

(2) For tolerances and inspections of splices of deck plate without backing strips see Table C.4 (1).

(3) Longitudinal splices (with welds running along the traffic lane) should be designed as transverse splices.



1 no sealing weld

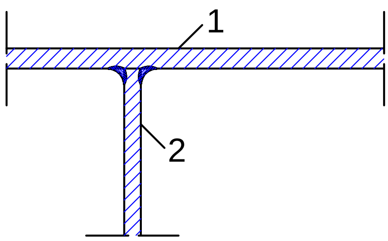
Figure C.7: Splice of deck plate in the direction of traffic lane with metallic backing strip

(4) V-welds with metallic backing strips may be used for longitudinal splices with the following requirements:

1. execution according to Figure C.7
2. tolerances and inspections according to Table C.4 (2).

C.1.2.4 Connection between deck plate and webs of main girders, webs of open section stiffeners and webs of crossbeams

(1) The welds connecting the deck plate with the webs should be designed as fillet welds according to Figure C.8.



1 deck plate

2 web of main girder

Figure C.8: Connection between deck plate and web of main girder

(2) For the connection of hollow section stiffeners to the deck plate see C.1.3.

C.1.3 Stiffeners

C.1.3.1 Fatigue actions

(1) Fatigue actions originate from

1. bending in the webs imposed from the deformations of the deck plate by the rigid welded connections between the stiffener and the deck plate,
2. shear in the welds between stiffeners and deck plate from shear forces in the stiffeners,
3. direct stresses in the stiffeners from bending moments in the stiffeners and from axial forces due to cooperation of the stiffeners in the top flange of the main girders,
4. local bending at the connection between stiffeners and the webs in the cross beams.

C.1.3.2 Type of stiffeners

(1) Stiffeners may be closed section stiffeners, whether trapezoidal, V-shape, round or open stiffeners.

(2) For closed section stiffeners see recommendations in Table C.3 (2).

(3) For open stiffeners under traffic lanes see recommendations in Table C.3 (3).

(4) In case of change of plate thickness of stiffeners, the misalignment at the surface of plates should not exceed 2 mm.

C.1.3.3 Stiffener-deck plate connection

(1) For closed section stiffeners under the carriageway the weld between the stiffener and the deck plate should be a butt weld.

(2) The throat thickness a should not be less than the thickness t of the stiffener, see Table C.4 (3) and (4).

(3) For stiffener to deck plate connections outside the carriageway Table C.4 (5) applies.

(4) For tolerances and tests see Table C.4 (3), (4) and (5).

C.1.3.4 Stiffener splice connection

(1) The stiffener splice connection should have splice plates in accordance with Table C.4 (6).

(2) The splice should be located close to the point of contraflexure of the stiffener (at a distance of $0{,}2\ell$ from cross beam, where ℓ = span of stiffener).

(3) The welding sequence should be such that residual stresses are small and that the bottom flange of the stiffener receives residual compression. The welding sequence specified in Table C.4 (6) is

1. First weld between stiffener and splice plate.
2. Second weld between stiffener and splice plate; at [1] and [2] according to Table C.4 (6) the bottom flange then the web should be welded.
3. Deck plate weld.

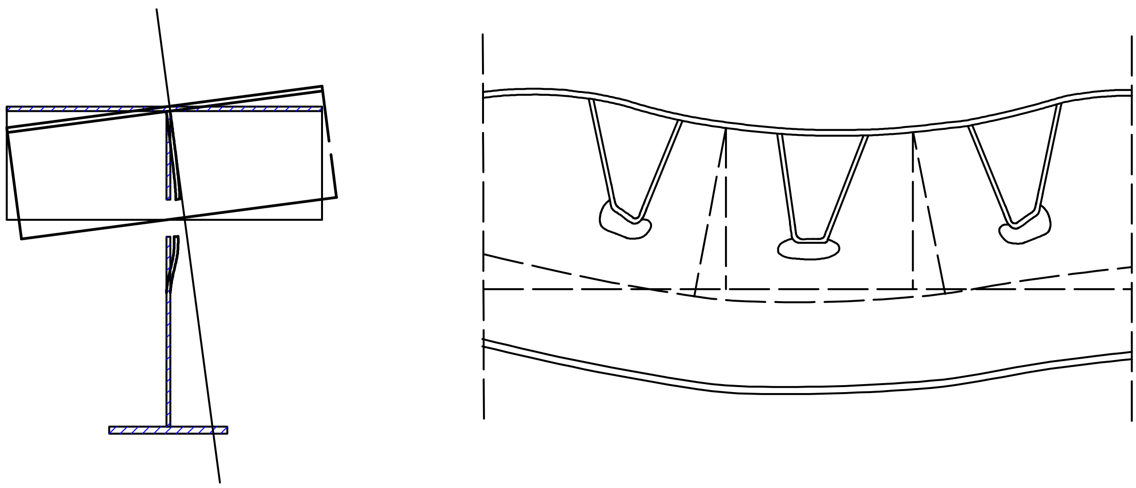
(4) For the butt welds between the stiffeners and the splice plate the tolerances and inspections according to Table C.4 (7) should apply.

C.1.3.5 Connection of stiffeners to the web of the cross beam

C.1.3.5.1 General

(1) Fatigue actions at the connection of the stiffeners to the web of the cross beam originate from the following, see Figure C.9:

1. Shear forces, torsional moments and stresses due to distortional deformations of the stiffeners induce stresses in the fillet welds between the stiffeners and the web of the cross beam.
2. Rotations of the stiffeners due to deflections of the stiffeners induce bending stresses in the web. Poisson effects result in transverse deformations of the stiffeners restrained at the web of the cross beam.
3. In plane stresses and strains in the web of the cross beam may cause stress concentration at the edges of the cope holes and deformations on the stiffeners.



rotation of the stiffener at its connection to web of cross beam, see C.1.3.5.1 (1) 2

imposed deformations to stiffener from strain distribution in the web of the cross beam, see C.1.3.5.1(1) 3

Figure C.9: Connection of stiffeners to the web of the cross beam

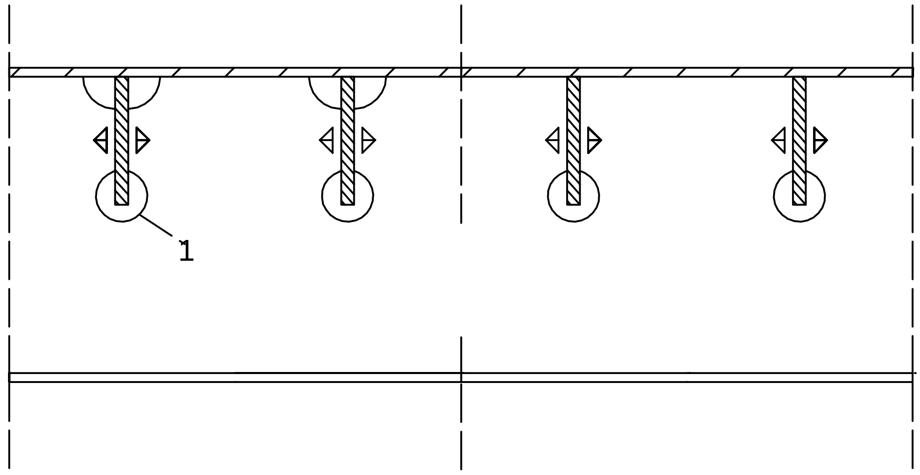
(2) The magnitude of these effects depends on whether stiffeners are passing through the web and the shapes of the cut out and cope hole, or stiffeners are fitted between the webs of cross beams including the shape and fit up.

(3) It is recommended that stiffeners should pass through the webs of the cross beam.

(4) Where it is not possible to provide stiffeners through the webs, e.g. for bridges with extremely small depths of cross beams or small spacing of cross beams, stiffeners fitted between the webs may be used following the recommendations in C.1.3.5.3.

(5) For flat stiffeners, see Figure C.10, the fatigue actions (see C.1.3.5.1 (1)) are similar to closed section stiffeners; however the effects of C.1.3.5.1 (1) 3. are smaller.

Open section stiffeners with longitudinal welds passing through the web of the cross beam
with cope holes without cope holes



1 cope hole at bottom of flat to prevent melting of sharp edges

Figure C.10: Connections of flat stiffeners with webs of cross beams

C.1.3.5.2 Cut outs in the webs of cross beams

(1) For closed section stiffeners cut outs should be designed as follows, see Figure C.11, either

1. with cope holes around the soffit of the stiffener, see Figure C.11 a, with partial welding of the stiffener to the web, or
2. without cope holes, see Figure C.11 b, with welding all around.

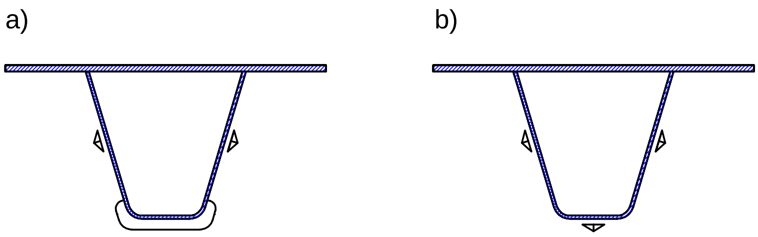
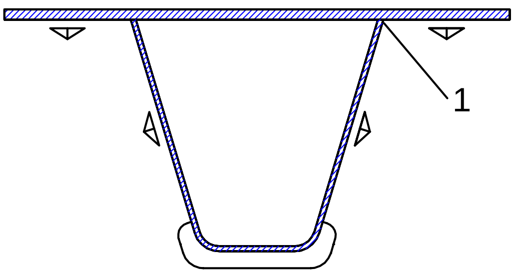


Figure C.11: Cut outs of webs of cross beams with or without cope holes

(2) Cope holes in the web of the cross beam at the stiffener deck plate connections should be avoided, see Figure C.12.

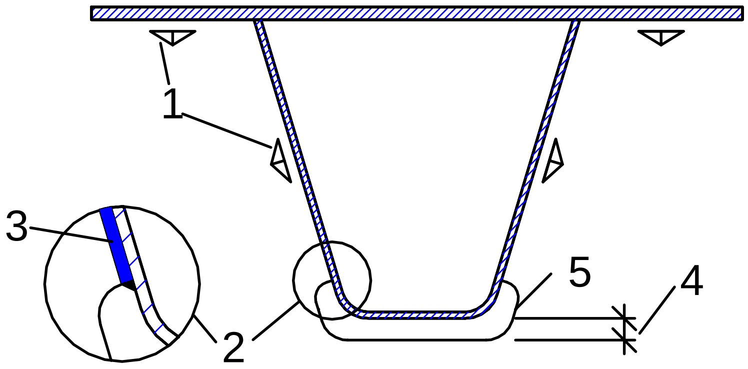


1 no cope holes
dimension according to Table C.4 (3), (4) and (5)

Figure C.12: Welded connections of closed section stiffeners with web of cross beam with cope holes

(3) The shape of the cut outs in the web of the cross beam, see Figure C.13, should be such that:

1. The welds between stiffeners and the web have adequate strength and the returns are without notches, see Figure C.13 a.
2. The dimensions of the cut out are sufficient to cope for stiffener profile tolerances and to allow surface preparation, application and inspection of the corrosion protection, see Figure C.13 b,
3. Stress ranges $\Delta\sigma$ at the edge of the cut outs from in plane bending and out of plane bending of the web are within acceptable limits, see Figure C.13, 5.



1 fillet welds

2 detail a)

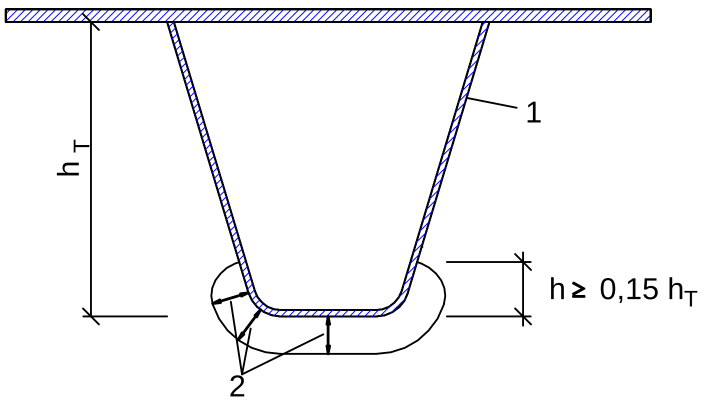
3 weld around the edge without notches, ground where necessary

4 detail b)

5 detail c)

Figure C.13: Critical details for the shape of cope holes

(4) The minimum size of the cut out should comply with ISO 12944-3 and Figure C.14.



1 plate thickness of web of cross beam $t_{w,crossb}$

2 constant value of clearance $b \geq 2t_{w,crossb} \geq 25$ mm

Figure C.14: Minimum dimensions of cope holes

(5) The requirements for tolerance and inspection are given in Table C.4 (9).

(6) For the connection of the stiffeners to the end-cross beam, see C.1.3.5.3.

(7) For the connection of stiffeners without cope holes the requirements are given in Table C.4 (8).

C.1.3.5.3 Stiffeners fitted between cross beams

(1) Stiffeners may only be fitted between cross beams, where the following conditions apply:

1. bridge is designed for light traffic only, or the stiffeners are not located under the traffic;
2. the spacing between cross beams is ≤ 2,75 m;
3. steels for the webs of the cross beam comply with the requirements for Z-quality according to EN 1993-1-10;
4. an assembly and welding sequence from field to field is provided that reduces shrinkage effects.

(2) The connection of the stiffeners to the web should be made by butt welds with a weld preparation according to the requirements in Table C.4 (10).

C.1.3.5.4 Stiffeners made of flat plates

(1) Flats passing through webs of cross beams should have continuous fillet welds to the deck plate and should be welded to the web of the cross beams at either sides, see Figure C.11.

(2) Gap width should be such that damage from shrinkage is avoided.

(3) The requirements for detailing and inspection should be taken from Table C.4 (11).

C.1.4 Cross beams

C.1.4.1 General

(1) The requirements for the cross beam comprise:

1. the plate thickness of the web and for the connections of the stiffeners to the web;
2. the web to deck plate connection;
3. the connection of the web of the cross beam to the web of the main girder;
4. web to bottom flange connection of the cross beam;
5. the connections of the bottom flange of cross beam to the web of main girder or to the bottom flange of maingirder where both flanges are on equal level;
6. the connection of cross beams to either transverse stiffeners, frames or diaphragms which are positioned in the same plane as the cross beams.

(2) Any corners of free edges of cut outs or cope holes should be radiused.

(3) The following detailed requirements apply.

C.1.4.2 Connections of the web of cross beam

(1) The requirements for detailing and inspection of welded connections of webs of cross beams to the deck plate and to the web of the main girder should be taken from Table C.4 (12) and Table C.4 (13) respectively.

(2) Splices of webs in cross beams should be according to Table C.4 (14).

C.1.4.3 Connections of the flange of cross beams

(1) The connection of the bottom flange of the cross beam to the web of the main girder should be a butt weld complying with Table C.4 (15).

(2) Where the bottom flanges of cross beams and of main girders are in the same plane, the connections should comply with the requirements in Table C.4 (16).

(3) Splices of flange of cross beams should comply with Table C.4 (14).

C.1.4.4 Transverse stiffeners, frames or diaphragms

(1) In order to reduce stress concentrations at connections between cross beams, transverse stiffeners and diaphragms appropriate local stiffening should be provided at all connections and joints.

(2) Connections of components of transverse frames to cross beams should be detailed according to Figure C.15. The details should be verified for fatigue.

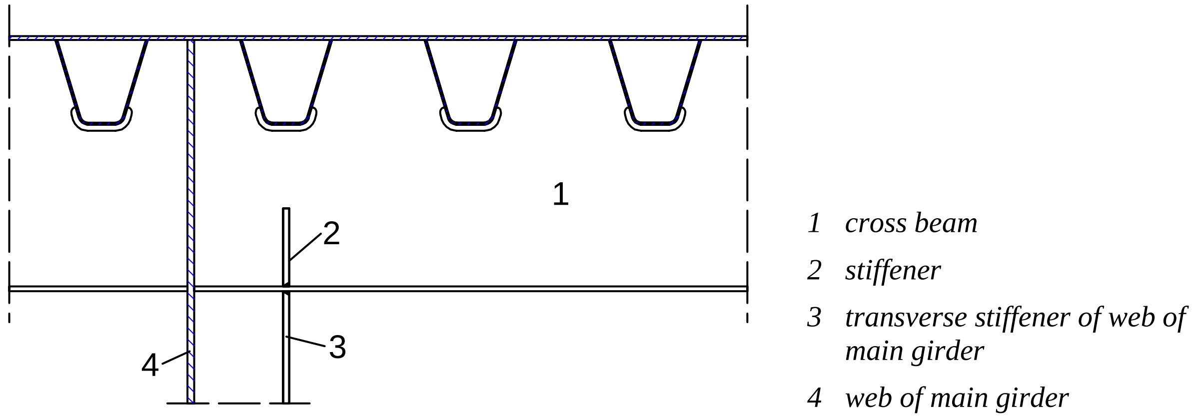


Figure C.15: Typical connection of cross beam to transverse stiffener of web of main girders

C.2 Railway bridges

C.2.1 General

(1) This annex gives recommendations for the design and structural detailing of orthotropic decks of railway bridges. It covers provisions for execution complying with the quality standard as assumed in EN 1993-1-9.

(2) Bridge decks of railway bridges may consist of the following:

1. longitudinal stiffeners and cross beams;
2. transverse stiffeners only.

(3) For bridge decks with longitudinal stiffeners, either open section stiffeners made of flats or closed section stiffeners with trapezoidal profiles should be used.

(4) For bridge decks with longitudinal closed section stiffeners cross beams should be designed with bottom flanges. For bridge decks with longitudinal stiffeners made of flats cross beams may be designed without bottom flanges. For bridge decks with transverse stiffeners only, flat stiffeners may be used without bottom flanges.

C.2.2 Plate thickness and dimensions

(1) For bridge decks with longitudinal stiffeners and cross beams, see Figure C.16, the requirements for plate thicknesses and dimensions in Table C.1 apply.

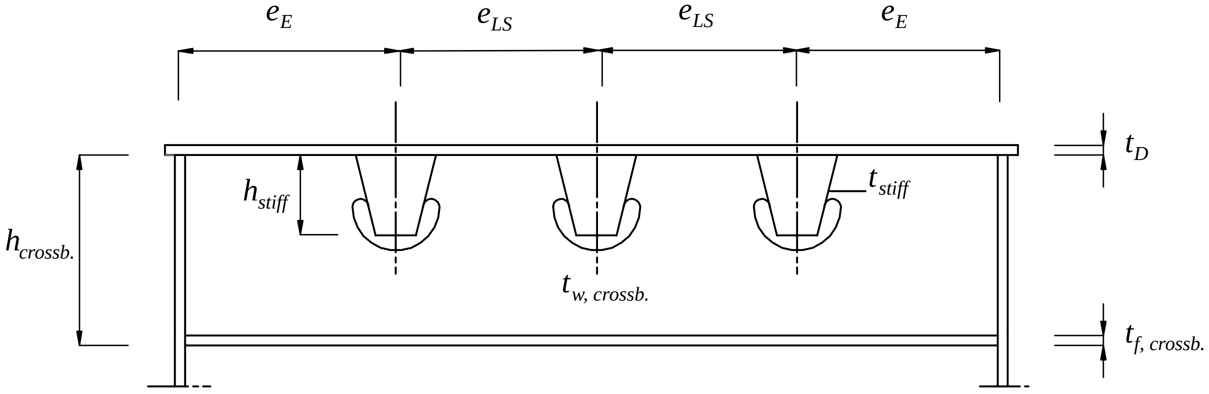


Figure C.16: Typical cross beam details

Table C.1: Dimensions of bridge deck with longitudinal stiffeners

	open section stiffeners	hollow section stiffeners
thickness of deck plate t_D	$t_D \geq 14$ mm	$t_D \geq 14$ mm
spacing e_{LS} between stiffeners	$e_{LS} \sim 400$ mm	600 mm $\leq e_{LS} \leq$ 900 mm
edge distance e_E of first stiffener	$e_E \geq e_{LS}$	$e_E \geq e_{LS}$
spacing of cross beams e_{crossb}	$e_{crossb} \leq 2700$ mm	2500 mm $\leq e_{crossb} \leq$ 3500 mm
ratio of depth of stiffener to depth of crossbeam h_{stiff}/h_{crossb}	$h_{stiff}/h_{crossb} \leq 0{,}5$	$h_{stiff}/h_{crossb} \leq 0{,}4$
plate thickness t_{stiff}	$t_{stiff} \geq 10$ mm	6 mm $\leq t_{stiff} \leq$ 10 mm
plate thickness of web of cross beam $t_{w,crossb}$	$t_{w,crossb} \geq 10$ mm	10 mm $\leq t_{w,crossb} \leq$ 20 mm
plate thickness of flange of cross beam $t_{f,crossb}$	$t_{f,crossb} \geq 10$ mm	$t_{f,crossb} \geq 10$ mm

(2) For bridge decks with transverse stiffeners only the requirements for plate thickness and dimensions in Table C.2 apply.

Table C.2: Dimensions of bridge deck with transverse stiffeners only

thickness of deck plate t_D	$t_D \geq 14$ mm
spacing of cross beams e_{crossb}	$e_{crossb} \sim 700$ mm
edge distance of cross beams e_E	$e_E \geq 400$ mm
plate thickness of web cross beam $t_{w,crossb}$	$t_{w,crossb} \geq 10$ mm
plate thickness of flange of cross beam $t_{f,crossb}$	(where flanges are provided) $t_{f,crossb} \geq 10$ mm

C.2.3 Stiffener to crossbeam connection

(1) Longitudinal stiffeners should normally pass through the webs of cross beams.

(2) The connections of open section stiffeners to the webs of cross beams should be detailed as shown in Figure C.17.

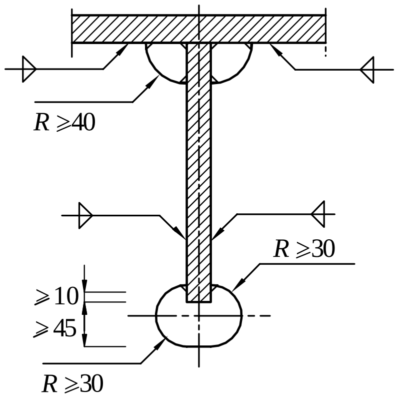


Figure C.17: Connection between flat stiffener and web of cross beam

(3) The connection of hollow section stiffeners to the webs of cross beams should be detailed as shown in Figure C.18.

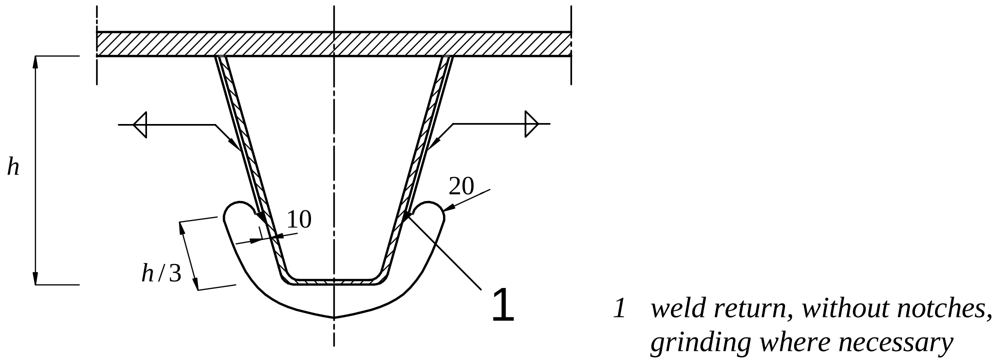


Figure C.18: Connection between closed stiffener and web of cross beam

C.2.4 Weld preparation tolerances and inspections

C.2.4.1 General

(1) Unless specified otherwise Table C.3 and C.4 should be used for the structural detailing, weld preparation, tolerances and inspections of the bridge.

C.2.4.2 Stiffener to deckplate connections

C.2.4.2.1 Weld preparation of stiffeners

(1) For stiffener to deckplate connections, the edges of the formed plates (see Table C.4 (3) and (4)) should be chamfered, see Figure C.19.

(2) Such a chamfering may be dispensed with for plate thicknesses t < 8 mm where by welding tests it can be proved that the requirements for butt welds according to C.2.4.2.2 are met.

C.2.4.2.2 Requirements for butt welds

(1) The requirements for the butt welds shall be as below:

– seam thickness $a \geq 0{,}9\ t_{stiff}$, see Table C.4(7)
– unwelded gap at root ≤ 0,25 t or ≤ 2 mm whichever is the smallest

where a = size of the weld

t = thickness of the plate

t_{stiff} = thickness of the stiffener

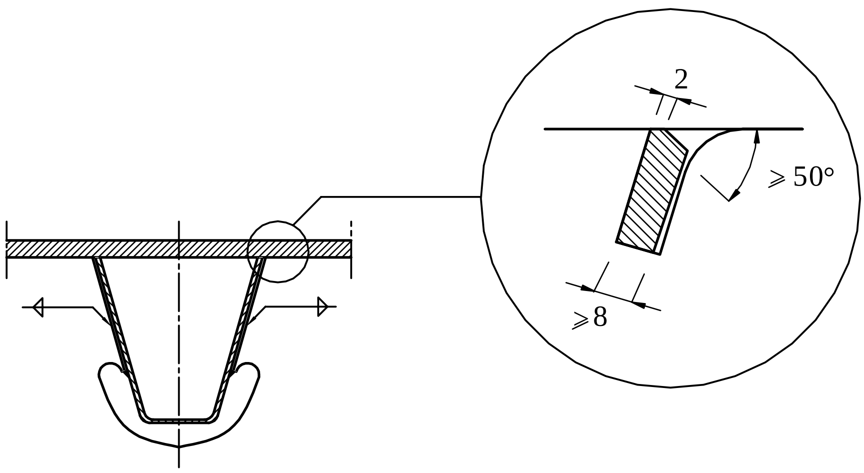


Figure C.19: Weld preparation of stiffener – deck plate connection

C.3 Tolerances for semi-finished products and fabrication

C.3.1 Tolerances for semi-finished products

(1) Irrespective of the fabrication methods for the delivery of the deck plate or formed profiles for stiffeners the tolerances for fabrication as specified in Table C.4 should be met.

(2) In Table C.3 recommendations for semi-finished products are given that may be used as a guidance for the procurement. These recommendations may be deviated from where the requirements of Table C.4 can be met by other measures.

C.3.2 Tolerances for fabrication

(1) The tolerances in Table C.4 apply for design, fabrication and execution of bridge decks.

(2) In Table C.4 the following abbreviations are used

- Requirement 1: External test results according to EN 25817 B
- Requirement 2: Internal test results according to EN 25817 B
- Requirement 3: See C.3.3
- Requirement 4: Steels to EN 10164 as required from EN 1993-1-10.

C.3.3 Particular requirements for welded connections

(1) Where required in Table C.4 by referring to this section the conditions specified in Table C.5 apply in supplement to EN 25817 B.

Table C.3: Tolerances of semifinished products

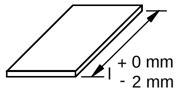
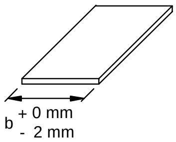
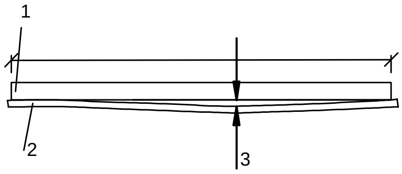
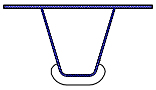
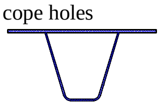
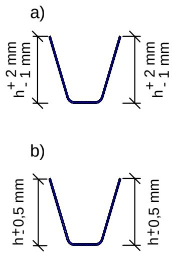
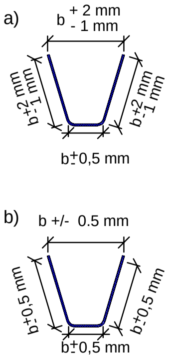
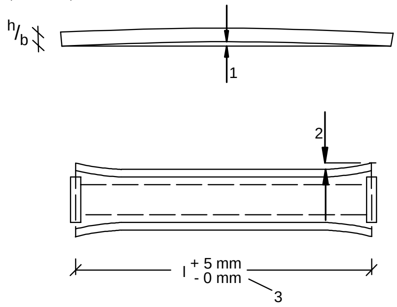
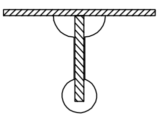
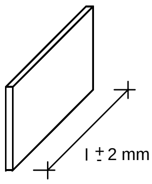
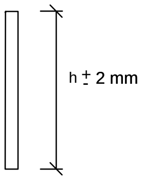
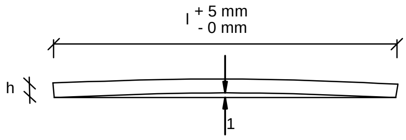
Product	Thickness	Length / depth	Width	Straightness	Remarks
1) Plate for deck after cutting and straightening by rolling	EN 10029, class C			 *1 measure length 2000 mm* *2 plate* *3 fit up gap max. 2,0 mm*	Length and widths inclusive provisions for shrinkage and after applying the final weld preparation.
2) Formed profile a) for passing through cross beams with cope holes  b) for passing through cross beams without cope holes 	EN 10029, class C			a) and b)  *1 max. gap L/1000* *2 max widening + 1 mm* *3 for stiffener splices with splice plates* *radius r = r ± 2 mm* *rotation 1° on 4 m length* *parallelism 2 mm*	Plate thickness t ≥ 6 mm For cold forming only material suitable for cold forming to be used. R/t ≥4 for welding quality in cold forming region. Ends of profiles to be inspected visually for cracks; in case of doubts by PT. ad b) In case of exceedance of the tolerances the cut outs in the cross beams to be adapted to meet maximum gap width,
3) Flat profile for welding on both sides 	EN 10029, class C			 *1 max. gap L/1000*	Plate thickness t ≥ 10 mm Choice of Z-quality according to EN 10164 from EN 1993-1-10 required.

Table C.4: Fabrication

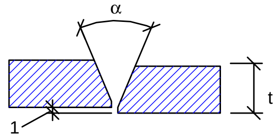
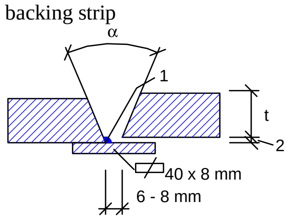
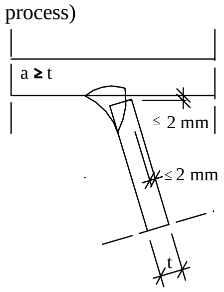
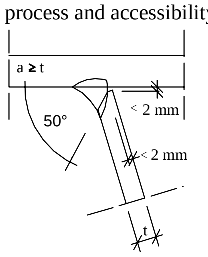
Structural detail	Stress level σ_{Ed}	Testing method and amount of testing	Test results required	Remarks
1) Splices of deck plate without backing strip  *1 misalignment ≤ 2 mm*	tensile stress $\sigma_{Ed} \leq 0,90\ f_{yk}$ and $\sigma_{Ed} > 0,75\ f_{yk}$	1a Inspection of weld preparation before welding 1b 100 % visual inspection after welding 2 100 % ultrasonic (UT) or radiographic (RT) testing	ad 1a Tolerances for weld preparation to be met, maximum misalignment ≤ 2 mm ad 1b Requirement 1 and 3 ad 2 Requirement 2 and 3	Testing requirement, see C.3.3.
	tensile stress $\sigma_{Ed} \leq 0,75\ f_{yk}$ and $\sigma_{Ed} > 0,60\ f_{yk}$	1a Inspection of weld preparation before welding 1b 100 % visual inspection after welding 2 100 % ultrasonic (UT) or radiographic (RT) testing	ad 1a Tolerances for weld preparation to be met, maximum misalignment ≤ 2 mm ad 1b Requirement 1 and 3 ad 2 Requirement 2 and 3	Testing requirement, see C.3.3.
	tensile stress $\sigma_{Ed} \leq 0,60\ f_{yk}$ or compression stress	1a Inspection of weld preparation before welding 1b 100 % visual inspection after welding	ad 1a Tolerances for weld preparation to be met, maximum misalignment ≤ 2 mm ad 1b Requirement 1 and 3	Testing requirement, see C.3.3.
2) Splices of deck plate with backing strip  *1 tack weld* *2 misalignment ≤ 2 mm* Weld preparation and weld preparation angle α in dependence of the welding process. Splices of metallic backing strips to be made of butt welds with grooved root and capping run. All work on splices to be finished before tack welding of deck plate. No sealing welds.	tensile stress $\sigma_{Ed} \leq 0,90\ f_{yk}$ and $\sigma_{Ed} > 0,75\ f_{yk}$	1a Inspection of weld preparation before welding; the melting of tack welds by subsequent weld beads to be verified by procedure tests 1b 100 % visual inspection after welding 2 100 % radiographic (RT) testing	ad 1a Tolerances for weld preparation to be met, tack welds of backing strips: Requirement 1 misalignment ≤ 2 mm ad 1b Requirement 1 fit up gaps between plate and backing strip ≤ 1 mm ad 2 Requirement 2 and 3	ad 1a Tack weld in the final butt weld, tack welds with cracks to be removed
	tensile stress $\sigma_{Ed} \leq 0,75\ f_{yk}$ and $\sigma_{Ed} > 0,60\ f_{yk}$	1a Inspection of weld preparation before welding 1b ≥ 50 % visual inspection after welding 2 10 % radiographic (RT) testing	ad 1a Tolerances for weld preparation to be met, tack welds of backing strips: Requirement 1 misalignment ≤ 2 mm ad 1b Requirement 1 and 3 ad 2 Requirement 2 and 3	ad 1a Tack weld in the final butt weld, tack welds with cracks to be removed
	tensile stress $\sigma_{Ed} \leq 0,60\ f_{yk}$ or compression stress	1a Inspection of weld preparation before welding 1b 100 % visual inspection after welding	ad 1a Tolerances for weld preparation to be met, misalignment ≤ 2 mm ad 1b Requirement 1 and 3	
3) Stiffener-deckplate connection (fully mechanized welding process)  	independent on stress level in deck plate	1a Inspection of weld preparation before welding 1b 100 % visual inspection after welding 2 Before fabrication: welding procedure tests to EN 288-3 or when this is available, to EN 288-8 with all welding heads. 3 During fabrication for each 120 m bridge 1 production test, however 1 production test for a bridge as a minimum, with all welding heads, checking by macro section tests	ad 1 Tolerances for weld preparations to be met ad 1b Requirement 1 ad 2 Fusion ratio to be met / Requirement 2 by preparing macro section tests (1 time at start or stop and one time at middle of weld) ad 3 see ad 2: however macro section tests only from middle of weld of the welding test	Starts and stops to be removed ad 2 Welding procedure tests under supervision of a recognized body, checking of welding parameters during fabrication ad 3 Execution, evaluation and documentation by fabricators production control, supervision by fabricators production control
4) Stiffener-deck plate connection (manual and partially mechanized welding process), weld preparation angle α in dependence of the welding process and accessibility  	independent on stress level in deck plate	1a Inspection of weld preparation before welding 1b 100 % visual inspection after welding	ad 1 Tolerances for weld preparations to be met ad 1b Requirement 1	Starts and stops to be removed This requirement also applied to local welds, e.g. at stiffener-stiffener connections with splice plates, see 16).

Table C.4 (continued): Fabrication

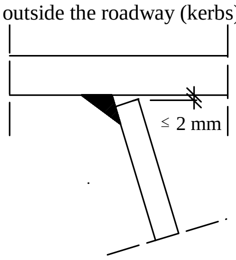
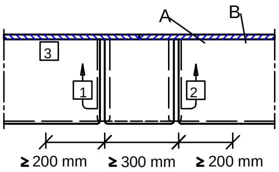
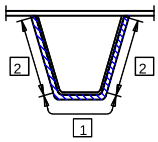
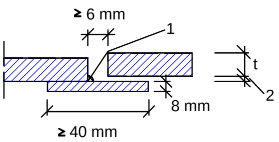
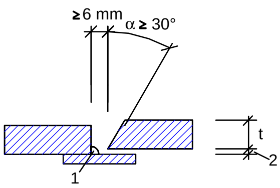
Structural detail	Stress level σ_{Ed}	Testing method and amount of testing	Test results required	Remarks
5) Stiffener-deck plate connection outside the roadway (kerbs)  throat thickness of fillet weld a as required by analysis	pedestrian loading without loading by vehicles except errant vehicles	1a Inspection of weld preparation before welding 1b ≥ 25 % visual inspection after welding 2 Measuring of throat thickness	ad 1a Tolerance of gap to be met ad 1b Requirement 1 ad 2 Requirement of throat thickness to be met and requirement 1	Starts and stops to be removed
6) Stiffener-stiffener connection with splice plates   *A site weld* *B shop weld*	independent on stress level	1a Inspection of weld preparation before welding 1b = 100 % visual inspection after welding	ad 1a Tolerance of gap to be met, misalignment between stiffener and splice plate ≤ 2 mm ad 1b Requirement 1 and 3	The non welded length of the seam on site between stiffeners and deck plate may also be provided at one side of the splice only. ad 1a For the root gaps see detail 7), for the site weld see details 3), 4) and 5)
7) Stiffener to stiffener connection with splice plates a) for plate thicknesses t = 6 - 8 mm  *1 continuous tack weld* *2 misalignment ≤ 2 mm* b) for plate thicknesses t ≥ 8 mm  *1 continuous tack weld* *2 misalignment ≤ 2 mm* weld preparation angle α dependant on welding process and gap width dependant on plate thickness	independent on stress level	1a Inspection of weld preparation before welding 1b = 100 % visual inspection after welding 2 Test of weld by 1 production test	ad 1a Tolerance of weld preparation to be met, misalignment ≤ 2 mm ad 1b Requirement 1 ad 2 Requirement 1 and 2	

Table C.4 (continued): Fabrication

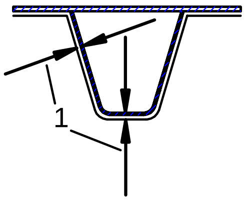
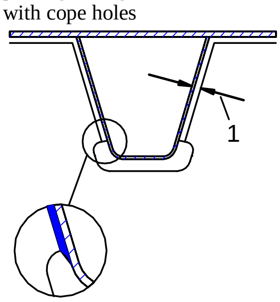
Structural detail	Stress level σ_{Ed}, τ_{Ed}	Testing method and amount of testing	Test results required	Remarks
8) Stiffener-cross beam connection with stiffeners passing through the cross beam without cope holes  *1 gap ≤3 mm*	throat thickness $a = a_{nom}$ according to analysis for gap width $s \leq 2$ mm, for greater gap widths s: $a = a_{nom} + (s-2)$ minimum throat thickness a = 4 mm	1a Inspection of weld preparation before welding 1b 100 % visual inspection after welding	ad 1a Tolerance of weld preparation to be met, required throat thickness a available ad 1b Requirement 1 and 3	1. It is assumed, that first the stiffeners are welded to the deck plate (with jigs and fixtures) and the cross beams are subsequently assembled and welded. 2. The tolerances for the cut outs of cross beams follow the tolerances of the formed profiles for the stiffeners, see Table C.3, detail 2)b). 3. The cut edges of the webs of cross beams should be without notches, in case there are they should be ground. For flame cutting EN ISO 9013 – Quality 1 applies.
9) Stiffener-cross beam connection with stiffeners passing through the cross beam with cope holes  *1 gap ≤3 mm* welds around edges of cope holes without notches	throat thickness $a = a_{nom}$ according to analysis for gap width ≤ 2 mm, for greater gap widths s: $a = a_{nom} + (s-2)$ minimum throat thickness a = 4 mm	1a Inspection of weld preparation before welding 1b 100 % visual inspection after welding	ad 1a Tolerance of weld preparation to be met, required throat thickness a available ad 1b Requirement 1 and 3	1. It is assumed, that first the stiffeners are welded to the deckplate (with jigs and fixtures) and the cross beams are subsequently assembled and welded. 2. The tolerances for the cut outs of cross beams follow the tolerances of the formed profiles for the stiffeners, see Table C.3, detail 2)a). 3. The cut edges of the webs of cross beams including the cope holes should be without notches, in case there are they should be ground. For flame cutting EN ISO 9013 – Quality 1 applies.

Table C.4 (continued): Fabrication

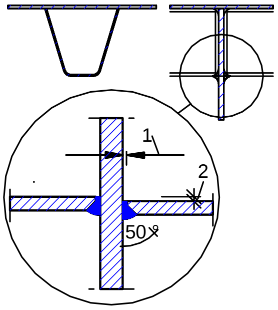
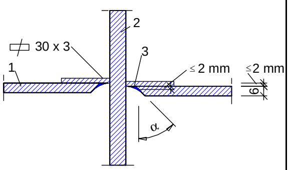
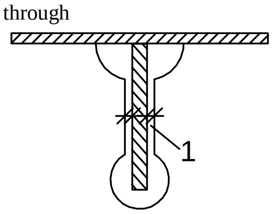
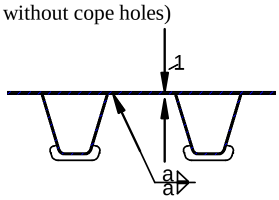
Structural detail	Stress level σ_{Ed}, τ_{Ed}	Testing method and amount of testing	Test results required	Remarks
10) Stiffener-cross beam connection with stiffeners fitted between cross beams (not passing through)  *1 gap ≤ 2 mm* *2 misalignment ≤ 2 mm* single sided full penetration weld (single V-weld) without backing strip  *1 stiffener* *2 web of cross beam* *3 tack weld* single sided full penetration weld with backing strip	throat thickness $a > t_{stiffener}$	1a Inspection of weld preparation before welding 1b ≥ 50 % visual inspection after welding	ad 1a Tolerance of weld preparation to be met, misalignment ≤ 2 mm ad 1b Requirement 1 and 3	1. This solution is only permitted for bridges with light traffic and for cross beam spacing ≤ 2,75 m. 2. Webs of cross beams see requirement 4. 3. The sequence of assembly and welding of stiffeners and cross beams should be decided to prevent harmful shrinkage effects. 4. Backing strips in one part, see 7). 5. Tack welds only inside final welds.
11) Stiffener-cross beam connection with flats passing through  *1 gap ≤ 1 mm*	throat thickness of fillet welds according to analysis	1a Inspection of weld preparation before welding 1b 100 % visual inspection after welding	ad 1a Tolerance of weld preparation to be met ad 1b Requirement 1 and 2	The cut edges of the cross beam should be prepared without notches and hardening, else they should be ground. For flame cutting EN ISO 9019 – quality 1 applies.
12) Connection of web of cross beam to deck plate (with or without cope holes)  *1 gap ≤ 1 mm*	throat thickness of fillet welds according to analysis	1a Inspection of weld preparation before welding 1b 100 % visual inspection after welding	ad 1a Tolerance of weld preparation to be met, requirement 1 and 2 ad 1b Requirement 1	The flame cut edges should be prepared in accordance with EN ISO 9019 – quality 1.

Table C.4 (continued): Fabrication

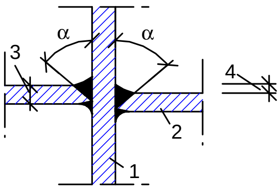
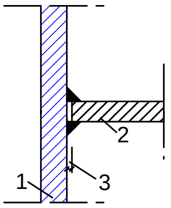
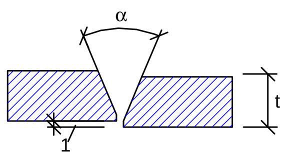
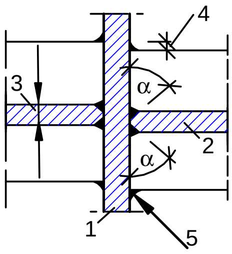
Structural detail	Stress level σ_{Ed}	Testing method and amount of testing	Test results required	Remarks
13) Connection of webs of cross beams to web of main girder a) for continuous cross beams  *1 web of main girder* *2 web of cross beam* *3 $t_{w,crossb}$* *4 misalignment $\leq 0,5\ t_{w,crossb}$*	independent on stress level	1a Inspection of weld preparation before welding 1b 100 % visual inspection after welding	ad 1a Tolerance of weld preparation to be met, requirement 1 for a), misalignment $\leq$ 0,5 $t_{web,cross\ beam}$ ad 1b Requirement 1	Execution with full penetration welds, weld preparation angle α and weld preparation in accordance with welding process and plate thickness.
b) for non continuous cross beams  *1 web of main girder* *2 web of cross beam* *3 gap ≤ 2 mm*	throat thickness of fillet weld according to analysis	see above	ad 1a see above ad 1b see above	Execution with fillet welds, see detail 12)
14) Splice of lower flange or web of cross beam  *1 misalignment 0 – 2 mm*	independent on stress level	1a Inspection of weld preparation before welding 1b 100 % visual inspection after welding 2 ≥ 10 % ultrasonic (UT) or radiographic (RT) testing	ad 1a Tolerance of weld preparation to be met, requirement 1, misalignment ≤ 2 mm ad 1b Requirement 1 and 3 2 Requirement 2	
15) Connection of cross beam flanges to web of main girder  *1 web of main girder* *2 web of cross beam* *3 $t_{w,crossb}$* *4 misalignment $\leq 0,5\ t_{w,cross}$* *5* $r \geq \frac{t_{w,maingirder}}{2} \geq 8$	independent on stress level	1a Inspection of weld preparation before welding 1b 100 % visual inspection after welding	ad 1a Tolerance of weld preparation to be met, misalignment 0,5 ≤ $t_{web,cross\ beam}$ ad 1b Requirement 1 and 3	1. Webs of main girders, requirement 4. 2. For smaller plate thicknesses also single V-welds with root run and capping run may be used, see 13). 3. Only full penetration butt welds with root run and capping run should be used.

Table C.4 (continued): Fabrication

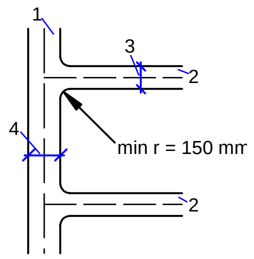
Structural detail	Stress level σ_{Ed}	Testing method and amount of testing	Test results required	Remarks
16) In plane connection of flanges of cross beams and main girders  *1 main girder* *2 cross beam* *3* b_{crossb} *4* $b_{main\ girder}$	minimum radius at connection min r = 150 mm, all plate thicknesses are equal otherwise a fatigue assessment is necessary			Transitions to be ground.

Table C.5: Conditions in supplement to EN 25817 B

to No.	Discontinuity	Supplementary requirement
3	Porosity and gas pores	only singular small pores acceptable
4	Localized (clustered) porosity	max. sum of pores: 2 %
5	Gas canal, long pores	no larger long pores
10	Bad fit up, fillet welds	transverse welds to be tested totally, small root reset only locally acceptable
		$b \leq 0{,}3 + 0{,}10\ a$, however $b \leq 1$ mm b = root gap or root reset resp.
11	Undercut	a) butt welds only locally acceptable $h \leq 0{,}5$ mm b) fillet welds not acceptable where transverse to stress direction, undercuts have to be removed by grinding.
18	Linear misalignment of edges	maximum 2 mm sharp edges to be removed
24	Stray flash or arc strike	not acceptable outside fusion zone
26	Multiple discontinuities in a cross section	not allowed
6	solid inclusions	not allowed
25	welding spatter	spatter and their heat affected zones to be removed

Annex D [informative] – Buckling lengths of members in bridges and assumptions for geometrical imperfections

D.1 General

(1) This annex gives buckling length factors β that may be used for the design of compression members in bridges in the expression:

$$\ell_K = \beta \ L \tag{D.1}$$

(2) This Annex also gives guidance for the application of imperfections in case second order analysis is carried out, see 5.3.2 of EN 1993-1-1.

(3) Imperfections may either be determined from the relevant buckling mode, see 5.3.2(10) of EN 1993-1-1 or from simplified assumptions for member imperfections, see 5.3.2(3) of EN 1993-1-1.

D.2 Trusses

D.2.1 Vertical and Diagonal elements with fixed ends

(1) Unless a more accurate verification is used with regard to the relative stiffnesses and the nature of connections then

- for in plane buckling: β = 0,9
- for out of plane buckling: β = 1,0

D.2.2 Vertical elements being part of a frame, see Figure D.1a or D.1b

(1) The buckling length factor β may be taken from Table D.1

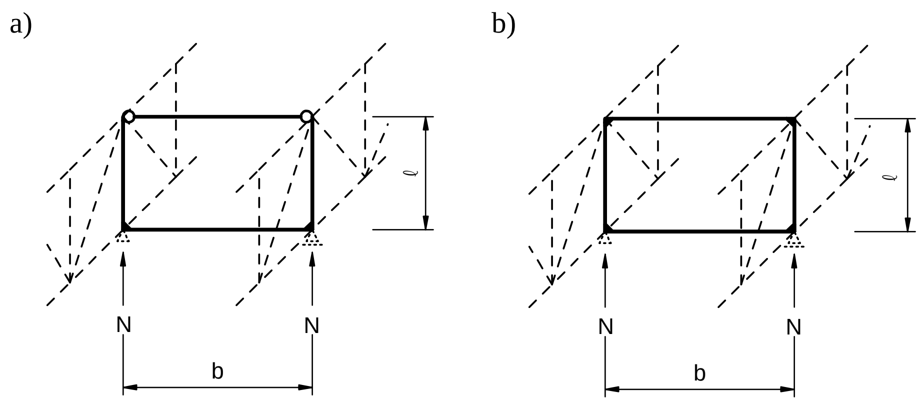
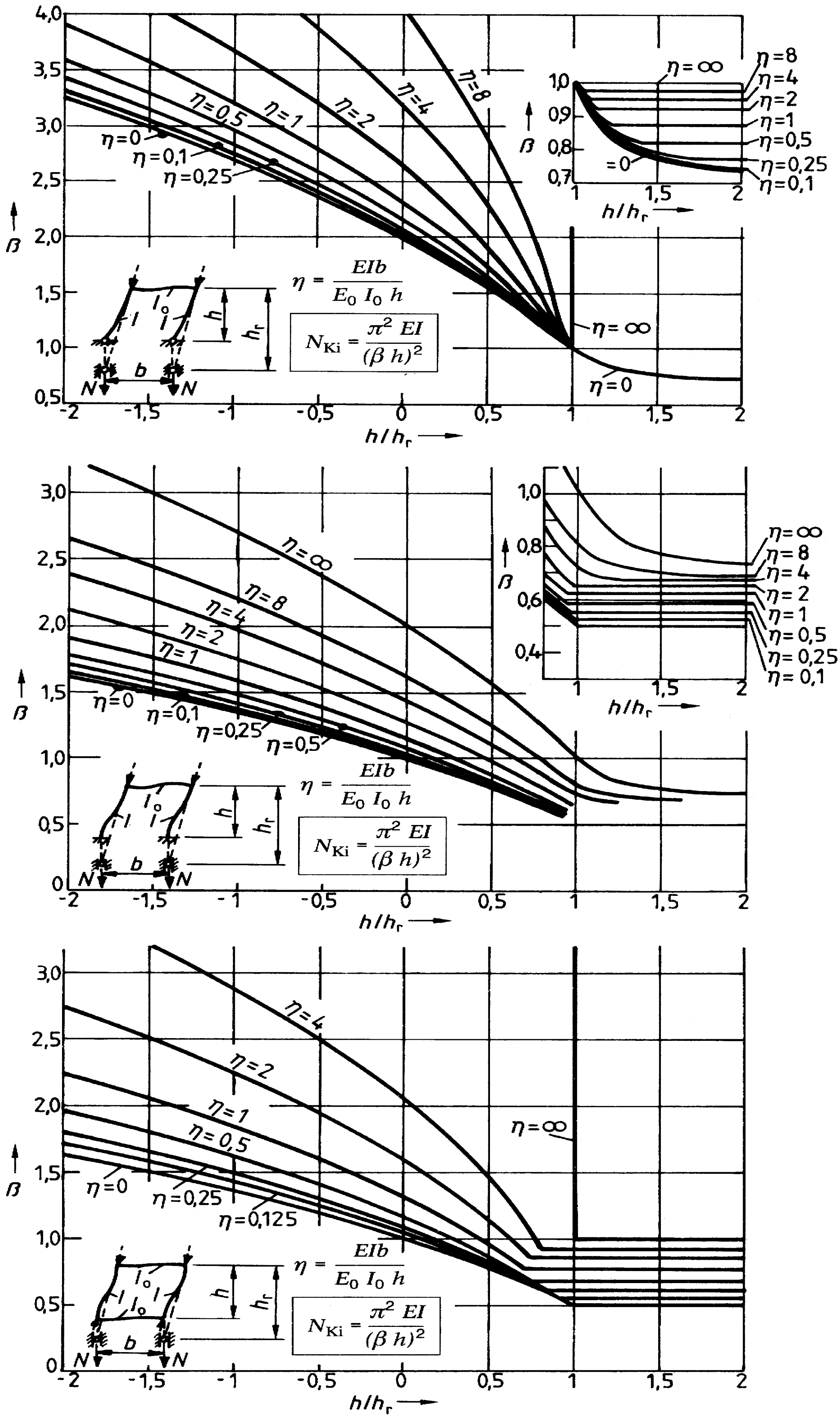


Figure D.1: Vertical elements being part of a frame

Table D.1: Buckling length factors β

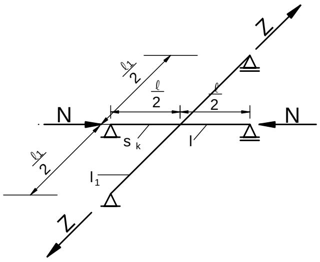
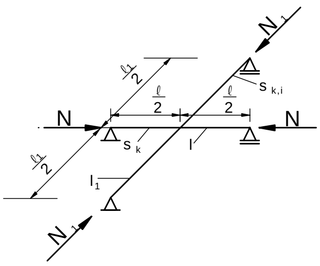
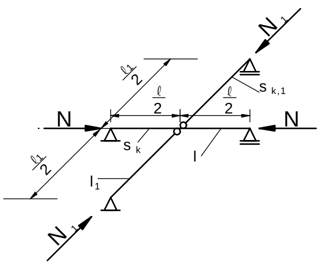
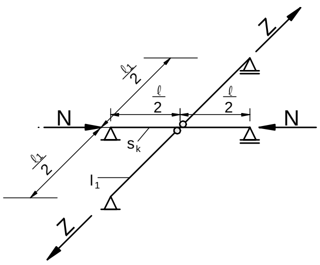
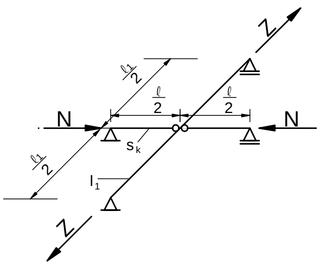
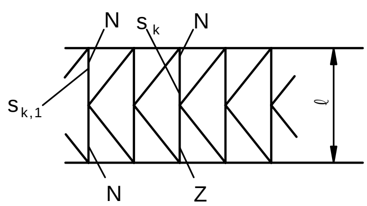


D.2.3 Out of plane buckling of diagonals

(1) The buckling lengths of diagonals of trusses may be taken from Table D.2.

(2) For achieving continuity of diagonals as given in Table D.2 connections should be effective in stiffness and strength in bending.

Table D.2: Buckling lengths

	1	2	3
1		$\beta = \sqrt{\dfrac{1-\dfrac{3}{4}\dfrac{Z\ell}{N\ell_1}}{1+\dfrac{I_1\ell^3}{I\ell_1^3}}}$ but $\beta \geq 0{,}5$	
2		$\beta = \sqrt{\dfrac{1+\dfrac{N_1\ell}{N\ell_1}}{1+\dfrac{I_1\ell^3}{I\ell_1^3}}}$ but $\beta \geq 0{,}5$	$\beta_1 = \sqrt{\dfrac{1+\dfrac{N\ell_1}{N_1\ell}}{1+\dfrac{I\ell_1^3}{I_1\ell^3}}}$ but $\beta_1 \geq 0{,}5$
3		continuous compression members $\beta = \sqrt{1+\dfrac{\pi^2}{12}\dfrac{N_1\ell}{N\ell_1}}$	hinged compression members $\beta_1 = 0{,}5$ when $EI \geq \dfrac{N_1\ell^3}{\pi^2\ell_1}\left(\dfrac{\pi^2}{12}+\dfrac{N\ell_1}{N_1\ell}\right)$
4		$\beta = \sqrt{1-0{,}75\dfrac{Z\ell}{N\ell_1}}$ but $\beta \geq 0{,}5$	
5		$\beta = 0{,}5$ when $\dfrac{N\ell_1}{Z\ell} \leq 1$ or when $EI_1 \geq \dfrac{3Z\ell_1^2}{4\pi^2}\left(\dfrac{N\ell_1}{Z\ell}-1\right)$	
6		$\beta = \left(0{,}75-0{,}25\left\|\dfrac{Z}{N}\right\|\right)$ but $\beta \geq 0{,}5$	$\beta_1 = \left(0{,}75-0{,}25\dfrac{N_1}{N}\right)$ $N_1 < N$

(3) For diagonals which are elastically supported at midspan, see Figure D.2:

$$\beta = \sqrt{1 - \frac{3}{16}\frac{C\,L}{N}} \tag{D.2}$$

where L is the system length

N is the maximum of N_1 or N_2

C is the lateral support stiffness but $C \leq \frac{4N}{\ell}$

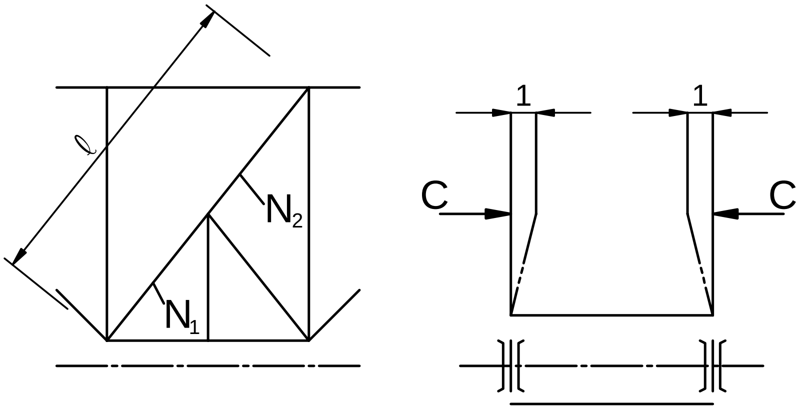


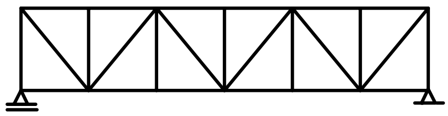
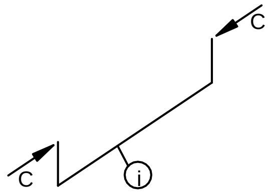
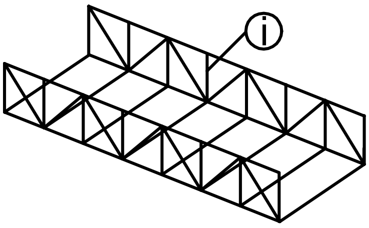
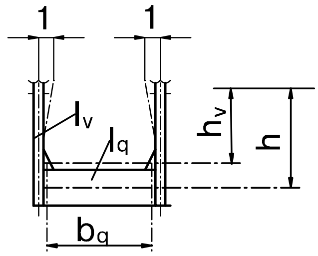
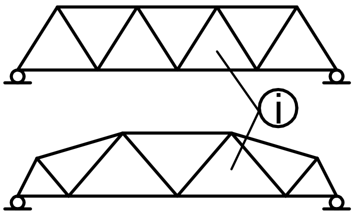
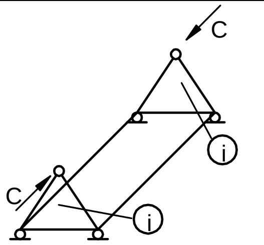
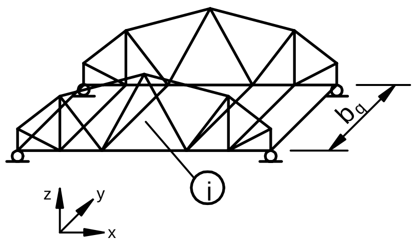
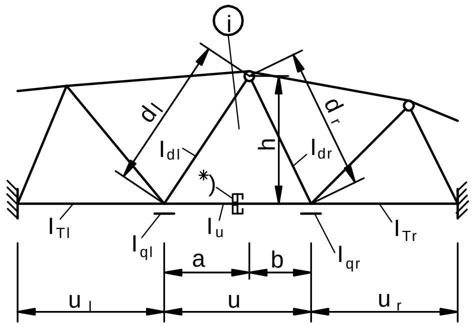
Figure D.2: Diagonal with elastical support at midspan

D.2.4 Compression chords of open bridges

(1) Compression chords may be modelled as columns with lateral supports.

(2) The stiffness of the lateral supports may be determined according to Table D.3.

Table D.3: Lateral stiffnesses C_d for trusses without posts

	1	2
1	 Example of truss bridges with posts	
1a	 Modelling	 $C = \dfrac{EI_v}{\dfrac{h_v^3}{3} + \dfrac{h^2 b_q I_v}{2I_q}}$
2	 Example of truss bridges without posts	 2U-frame in truss bridges without posts
2a	 Modelling	 *) torsional hinge Modelling: bottom chord of the U-frame with flexural stiffness I_ℓ only, adjacent bottom chords with torsional stiffness I_T
2b	spring stiffness $C = \dfrac{A + B - 2D}{AB - D^2} EI_u$	
2c	$A = \dfrac{h^2 I_u}{n_\ell} + \dfrac{d_\ell^3 I_u}{3I_{d\ell}} + \dfrac{a^2 u}{3}$ $\quad n_\ell = \dfrac{2}{b_q} I_{q\ell} + \dfrac{GI_{T\ell}}{Eu_\ell}$ $B = \dfrac{h^2 I_u}{n_r} + \dfrac{d_r^3 I_u}{3I_{dr}} + \dfrac{b^2 u}{3}$ $\quad n_r = \dfrac{2}{b_q} I_{qr} + \dfrac{GI_{Tr}}{Eu_r}$ $D = \dfrac{1}{6} abu$ The length d_ℓ, d_r, a, b, u and b_q may be reduced in case of rigid ends. u_ℓ and u_r may be reduced in case of end that are torsionally rigid. $EI_{d\ell}$, EI_{dr}, EI_u = bending stiffness of diagonals and bottom chords for out-of-plane bending $EI_{q\ell}$, EI_{qr} = bending stiffness of the cross beam $GI_{T\ell}$, GI_{Tr} = St. Venant torsional stiffness of the adjacent chords	

D.3 Arched Bridges

D.3.1 General

(1) In the following buckling length factors β are given for in plane and out of plane buckling of arches.

(2) The critical buckling force N_{cr} in the arch for in plane buckling is expressed by

$$N_{cr} = \left(\frac{\pi}{\beta s}\right)^2 EI_y \tag{D.3}$$

where N_{cr} relates to the force at the supports

s is the half length of the arch

EI_y is the in plane flexural stiffness of the arch

β is the buckling length factor

(3) The critical buckling force in free standing arches for out of plane buckling is expressed by

$$N_{cr} = \left(\frac{\pi}{\beta s}\right)^2 EI_z \tag{D.4}$$

where N_{cr} relates to the force at the supports

ℓ is the projection length of the arch

EI_z is the out of plane flexural stiffness of the arch

β is the buckling length factor

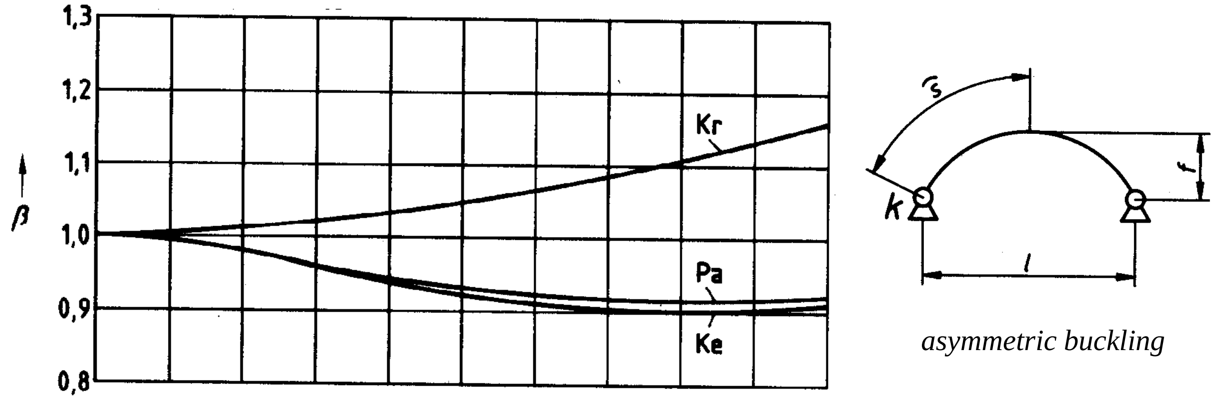
(4) The out of plane buckling of arches with wind bracing and portals may be verified by a stability check of the end portals.

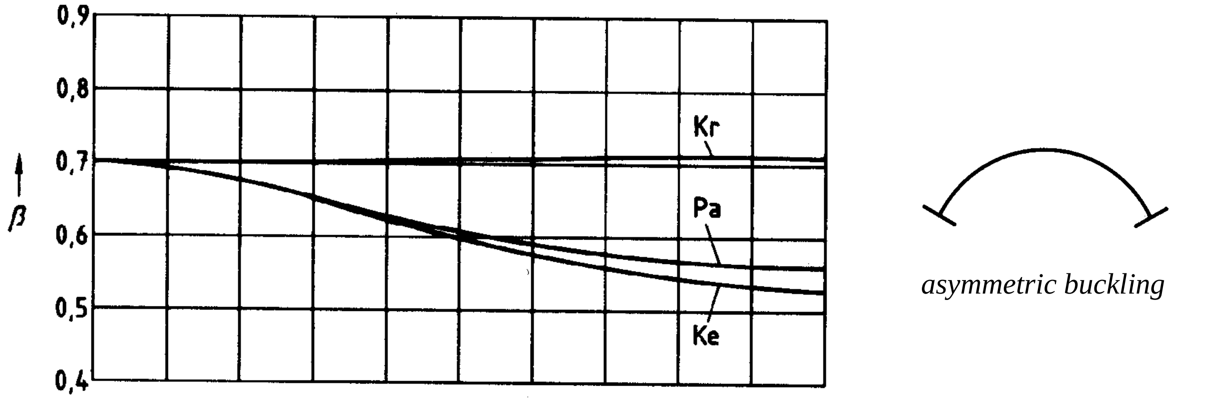
D.3.2 In plane buckling factors for arches

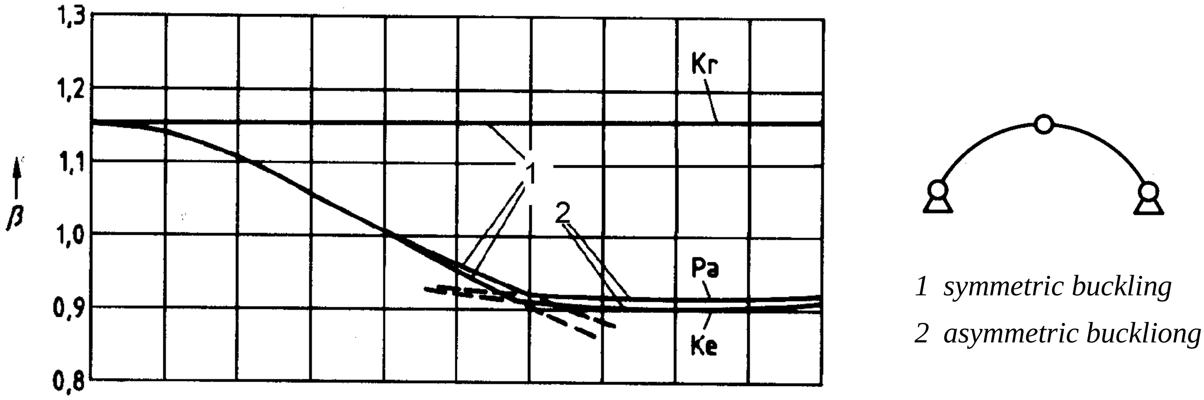
(1) For arches with rigid supports buckling factors β are given in Table D.4.

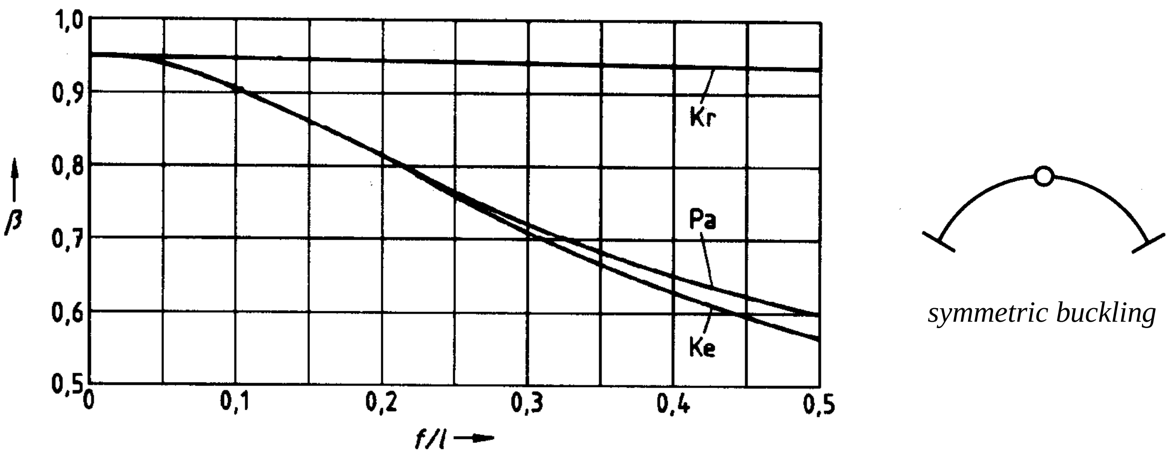
(2) For arches with a tension tie and hangers buckling factors β are given in Figure D.4.

Table D.4: Buckling length factor β for arches









Pa: parabolic form Ke: chain form Kr: circular form
for Pa and Ke the loading is vertical

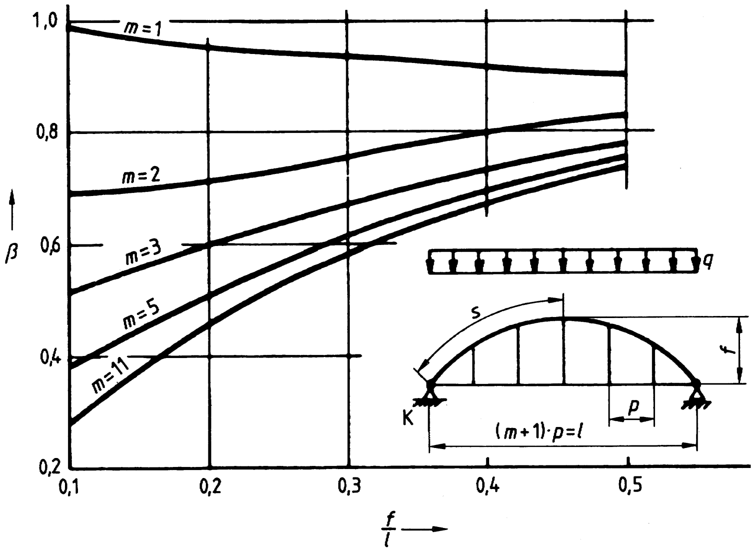


Figure D.4: Buckling factor β

(3) Snap through of arches may be assumed to be prevented, if the following criterion is satisfied:

$$\ell \sqrt{\frac{EA}{12EI_y}} > K \tag{D.5}$$

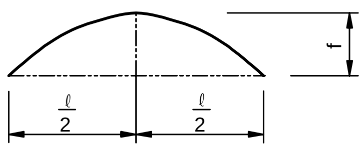
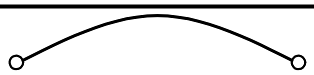
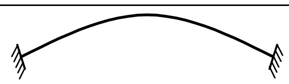
where A is the cross sectional area

I_y is the moment of inertia

K is a factor

(4) The factor K may be taken from Table D.5.

Table D.5: Factor K

	f/ℓ	0,05	0,075	0,10	0,15	0,20
	K	35	23	17	10	8
		319	97	42	13	6

D.3.3 Out of plane buckling factors for free standing arches

(1) For out of plane buckling of free standing arches the buckling factors may be taken as

$$\beta = \beta_1 \beta_2 \tag{D.6}$$

where β_1 is given in Table D.6 and β_2 is given in Table D.7

Table D.6: β_1 - values

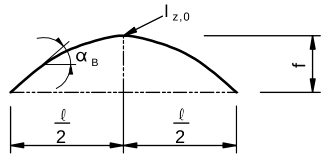
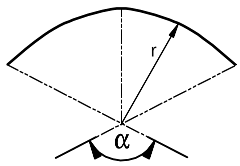
f/ℓ	0,05	0,10	0,20	0,30	0,40	
I_z constant	0,50	0,54	0,65	0,82	1,07	
I_z varies $I_z(\alpha_B) = \frac{I_{z,0}}{\cos\alpha_B}$	0,50	0,52	0,59	0,71	0,86	

Table D.7: β_2 - values

Loading	β_2	Comments
conservative (The deck is fixed to the top of the arch)	1	
by hangers	$1 - 0{,}35\frac{q_H}{q}$	
by posts	$1 - 0{,}45\frac{q_{St}}{q}$	q total load q_H load part transmitted by hangers q_{St} load part transmitted by posts

(2) For out of plane buckling of free standing circular arches with radial loading the buckling factor β may be taken as

$$\beta = \pi r \alpha \frac{\sqrt{\pi^2 + \alpha^2 K}}{\ell(\pi^2 - \alpha^2)} \tag{D.7}$$

where r is the radius of the circle

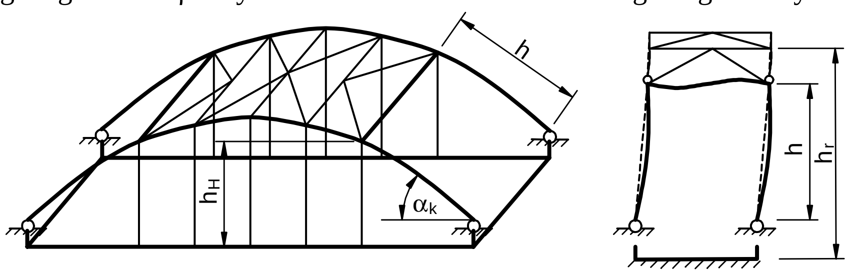
α is the section angle of the arch $0 < \alpha < \pi$

$K = \frac{EI_z}{GI_T}$

D.3.4 Out of plane buckling of arches with wind bracing and end portals

(1) The out of plane buckling may be verified by a stability check of the end portals according to D.2.2.

(2) The buckling length factor β may be taken from Table D.1 in using the geometry in Figure D.5.



bridge deck

Figure D.5: Buckling of portals for arches

(3) The value h_r in Table D.1 may be taken as the mean of all lengths $h_H \frac{1}{\sin\alpha_k}$ of the hangers.

D.3.5 Imperfections

(1) Unless the relevant buckling modes are used for imperfection, see 5.3.2(10) of EN 1993-1-1, the bow imperfections given in Table D.8 for in plane buckling of arches and in Table D.9 for out of plane buckling of arches may be used.

Table D.8: Shape and amplitudes of imperfections for in plane buckling of arches

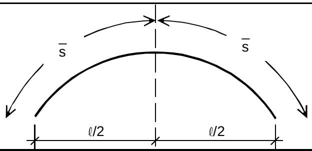
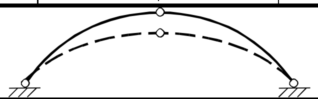
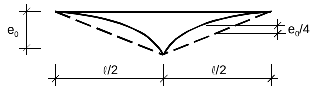
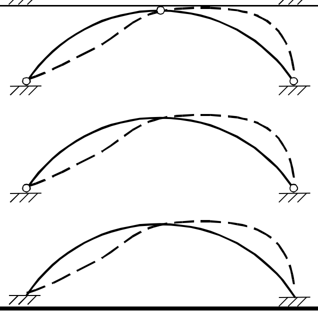
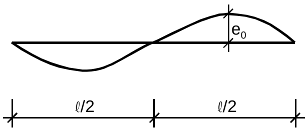
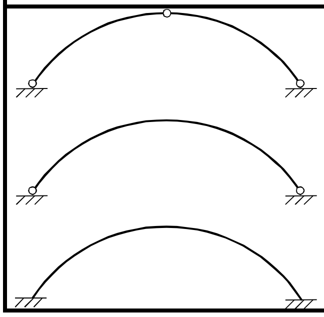
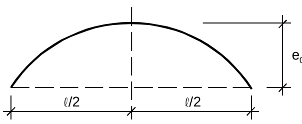
	1	2	3			
		shape of imperfection (sinus or parabola)	e_0 according to classification of cross section to buckling curve			
			a	b	c	d
1			$\frac{s}{300}$	$\frac{s}{250}$	$\frac{s}{200}$	$\frac{s}{150}$
2			$\frac{\ell}{600}$	$\frac{\ell}{500}$	$\frac{\ell}{400}$	$\frac{\ell}{300}$

Table D.9: Shape and amplitudes of imperfections for out of plane buckling of arches

	shape of imperfection (sinus or parabola)	e_0 according to classification of cross section to buckling curve				
			a	b	c	d
		$\ell \leq 20$ m	$\frac{\ell}{300}$	$\frac{\ell}{250}$	$\frac{\ell}{200}$	$\frac{\ell}{150}$
		$\ell > 20$ m $\ell_1 = \sqrt{20\,\ell\,[m]}$	$\frac{\ell_1}{300}$	$\frac{\ell_1}{250}$	$\frac{\ell_1}{200}$	$\frac{\ell_1}{150}$

Annex E [informative] – Combination of effects from local wheel and tyre loads and from global traffic loads on road bridges

E.1 Combination rule for global and local load effects

(1) When considering the local strength of stiffeners of orthotropic decks effects from local wheel and tyre loads acting on the stiffener and from global traffic loads acting on the bridge should be taken into account, see Figure E.1.

(2) To take account the different sources of these loads the following combination rule may be applied to determine the design values:

$$\sigma_{Ed} = \sigma_{loc,d} + \psi\sigma_{glob,d} \tag{E.1}$$

$$\sigma_{Ed} = \psi\sigma_{loc,d} + \sigma_{glob,d} \tag{E.2}$$

where σ_d design value of stress in the stringer due to combined effects of local load σ_{loc} and global load σ_{glob}

$\sigma_{loc.d}$ design value of stress in the stringer due to local wheel or tyre load from a single heavy vehicle

$\sigma_{glob.d}$ design value of stress in the stringer due to bridge loads comprising one or more heavy verhicles

ψ combination factor

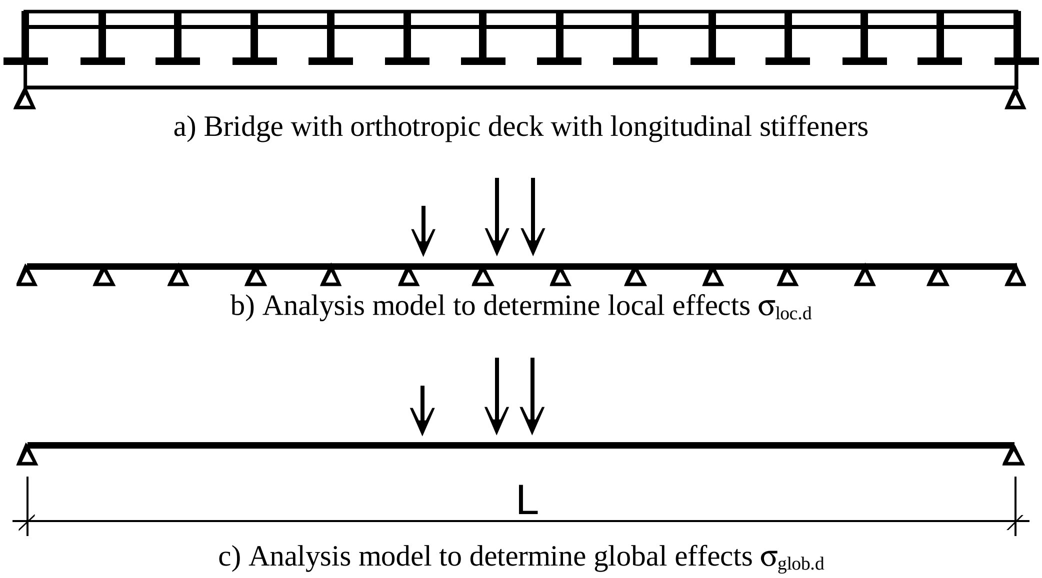


Figure E.1: Modelling of structure with local and global effects

E.2 Combination factor

(1) The combination factor ψ may be determined on the basis of the weight distributions of several lorries acting on an influence line for combined action effects.

NOTE The National Annex may give information on the combination factor. The factor in Figure E.2 is recommended.

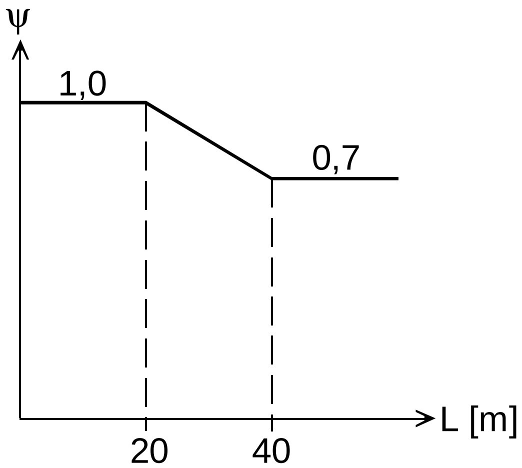


Figure E.2: Combination factor dependent on span length L

EUROPEAN PRESTANDARD
PRÉNORME EUROPÉENNE
EUROPÄISCHE VORNORM

prEN 1993-3 : 20xx

8 October 2001

UDC

Descriptors:

English version

Eurocode 3 : Design of steel structures

Part 3 : Buildings

Calcul des structures en acier

Partie 3 :

Ingènierie du bâtiment

Bemessung und Konstruktion von Stahlbauten

Teil 3 :

Hochbau

CEN

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

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

Ref. No. EN 1993-3 : 20xx. E

Content

Page

Foreword

This European Standard EN 1993-3, Design of Steel Structures : Buildings, has been prepared on behalf of Technical Committee CEN/TC250 « Structural Eurocodes », the Secretariat of which is held by BSI. CEN/TC250 is responsible for all Structural Eurocodes.

The text of the draft standard was submitted to the formal vote and was approved by CEN as EN 1993-2 on YYYY-MM-DD.

No existing European Standard is superseded.

Background of the Eurocode programme

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

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

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

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

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

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

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

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

Status and field of application of Eurocodes

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

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

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

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

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

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

National Standards implementing Eurocodes

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

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

– values for partial factors and/or classes where alternatives are given in the Eurocode,

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

– geographical and climatic data specific to the Member State, e.g. snow map,

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

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

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

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

---

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

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

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

Additional information specific to EN 1993-3

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

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

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

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

EN 1993-3 is intended for use by

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

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

National annex for EN 1993-3

This standard gives alternative procedures, values and recommendations for classes with notes indicating where national choices may have to be made. Therefore the National Standard implementing EN 1993-3 should have a National Annex containing all Nationally Determined Parameters to be used for the design of steel structures to be constructed in the relevant country.

National choice is allowed in EN 1993-3 through clauses:

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

1 General

1.1 Scope

1.1.1 Scope of Eurocode 3

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

1.1.2 Scope of Part 3 of Eurocode 3

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

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

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

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

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

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

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

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

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

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

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

Section 1: Introduction

Section 2: Basis of design

Section 3: Materials

Section 4: Durability

Section 5: Structural analysis

Section 6: Ultimate limit states

Section 7: Serviceability limit states

Section 8: Connections

[5] EN xxx is the conversion of ENV 1090

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

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

(14) Section 4 gives general rules for durability.

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

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

(17) Section 7 gives rules for serviceability.

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

1.2 Normative references

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

EN xxx[5] Execution of steel structures:

1.3 Assumptions

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

1.4 Distinction between principles and application rules

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

1.5 Definitions

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

Draft note: ... to be inserted later.

1.6 Symbols

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

Draft note: ... to be inserted later.

2 Basis of design

2.1 Requirements

2.1.1 Basic requirements

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

2.1.2 Reliability management

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

2.1.3 Design working life, durability and robustness

2.1.3.1 General

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

2.1.3.2 Design working life

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

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

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

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

2.1.3.3 Durability

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

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

2.1.3.4 Robustness and structural integrity

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

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

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

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

2.2 Principles of limit state design

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

2.3 Basic variables

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

2.4 Verification by the partial factor method

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

2.5 Design assisted by testing

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

3 Materials

3.1 General

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

3.2 Structural steel

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

3.3 Connecting devices

3.3.1 Fasteners

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

3.3.2 Welding consumables

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

3.4 Other items

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

4 Durability

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

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

a) Members supporting lifting appliances or rolling loads

b) Members subject to repeated stress cycles from vibrating machinery

c) Members subject to wind-induced vibrations

d) Members subject to crowd-induced oscillations

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

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

5 Structural analysis

5.1 Structural modelling for analysis

5.1.1 Structural modelling and basic assumptions

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

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

5.1.2 Joint modelling

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

5.1.3 Ground structure interaction

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

5.2 Structural stability

5.2.1 Effects of deformed geometry of the structure

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

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

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

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

h is the storey height

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

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

5.2.2 Method of analysis

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

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

NOTE In general structural stability may be achieved by bracing.

5.3 Imperfections

5.3.1 Basis

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

5.3.2 Imperfections for global analysis

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

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

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

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

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

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

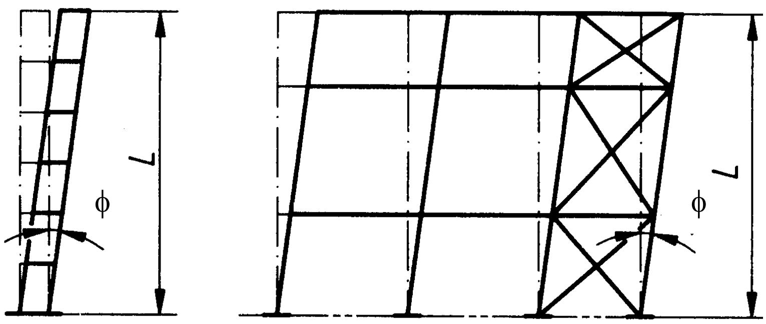


Figure 5.1: Equivalent sway imperfections for buildings

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

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

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

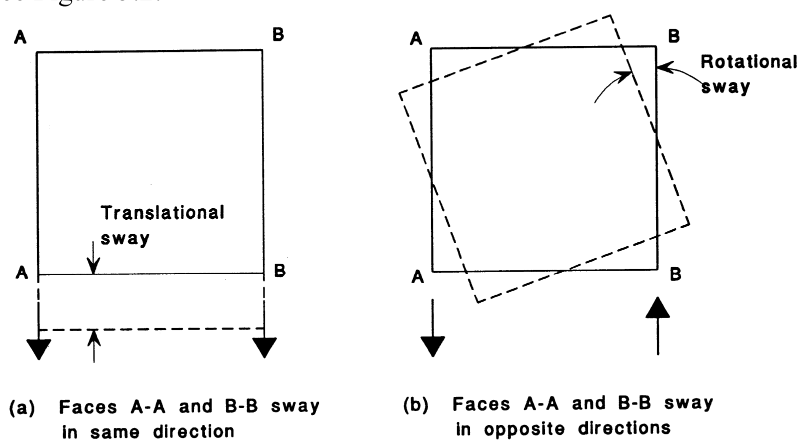


Figure 5.2: Transitional and torsional effects

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

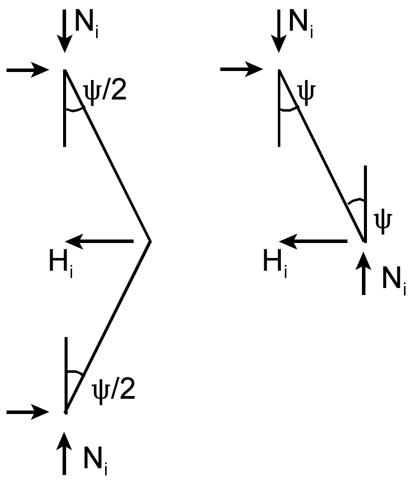


Figure 5.3: Configuration of imperfections for floor diaphragms

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

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

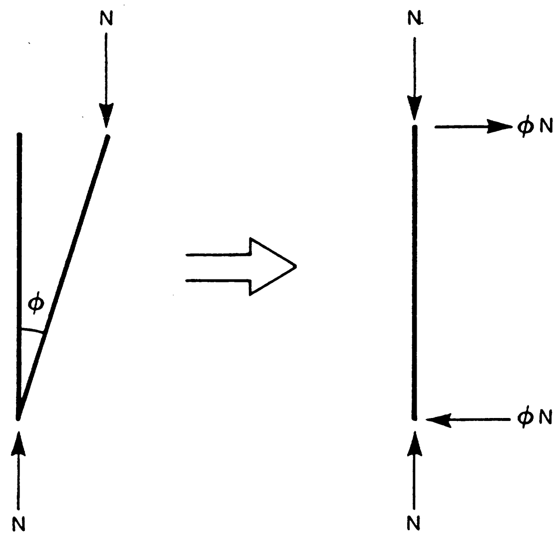


Figure 5.4: Replacement of initial sway imperfections by equivalent horizontal forces

5.3.3 Imperfection for analysis of bracing systems

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

5.3.4 Member imperfections

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

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

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

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

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

5.4 Calculation of action effects

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

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

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

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

5.5 Classification of cross sections

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

6 Ultimate limit states

6.1 General

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

6.2 Resistance of cross-sections

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

6.3 Buckling resistance of members

6.3.1 Compression members

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

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

6.3.2 Lateral-torsional buckling of beams

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

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

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

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

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

W_y is the appropriate section modulus of the compression flange

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

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

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

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

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

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

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

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

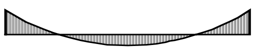
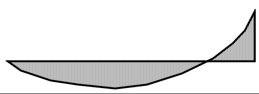
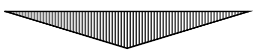
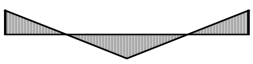
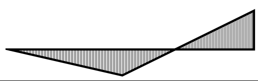
$A_{eff,f}$ is the effective areas of the flange

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

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

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

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

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

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

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

where χ is the reduction factor of the equivalent compression member

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

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

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

curve c for rolled sections

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

where h is the overall depth of the cross-section

t is the thickness of the compression flange

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

6.3.3 Lateral torsional buckling of frames

6.3.3.1 General method

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

6.3.3.2 Lateral torsional buckling of portal frames with plastic design

6.3.3.2.1 General

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

a) restraints at “rotated” plastic hinges

b) verification of stable length of segment between such restraints

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

6.3.3.2.2 Restraints at rotated plastic hinges

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

6.3.3.2.3 Verification of stable length of segment

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

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

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

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

6.3.4 Bending and axial compression

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

6.4 Built-up compression members

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

6.5 Buckling of plates

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

7 Serviceability limit states

7.1 General

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

7.2 Serviceability limit states for buildings

7.2.1 General

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

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

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

7.2.2 Recommendations for horizontal deflections

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

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

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

7.2.3 Dynamic effects

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

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

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

where δ is the deflection in [mm]

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

8 Fasteners, welds, connections and joints

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

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

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

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

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

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

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

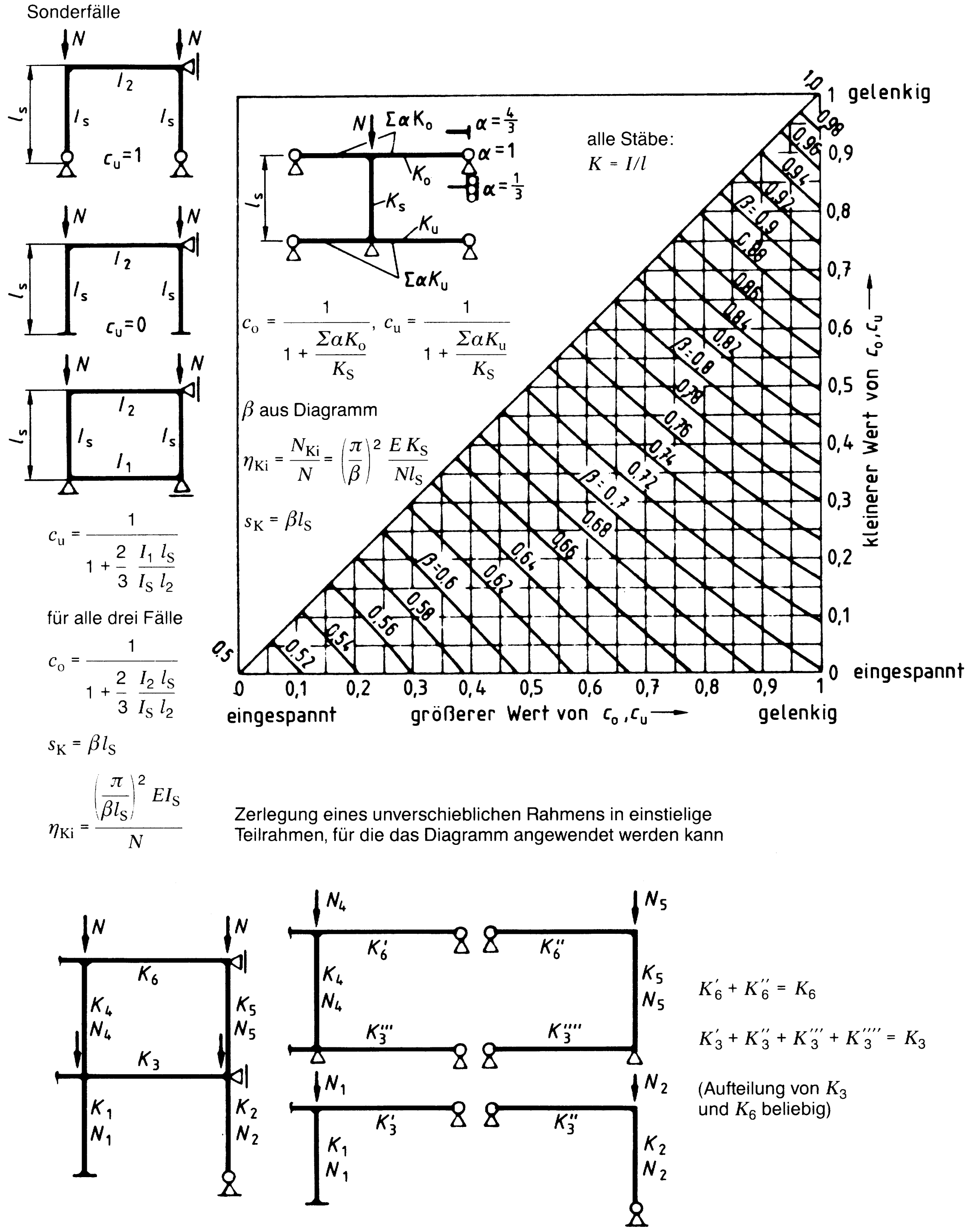


Bild 27. Diagramm zur Bestimmung des Verzweigungslastfaktors η_{Ki} und der Knicklängen s_K für Stiele unverschieblicher Rahmen mit $\varepsilon_{Riegel} \leq 0{,}3$

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

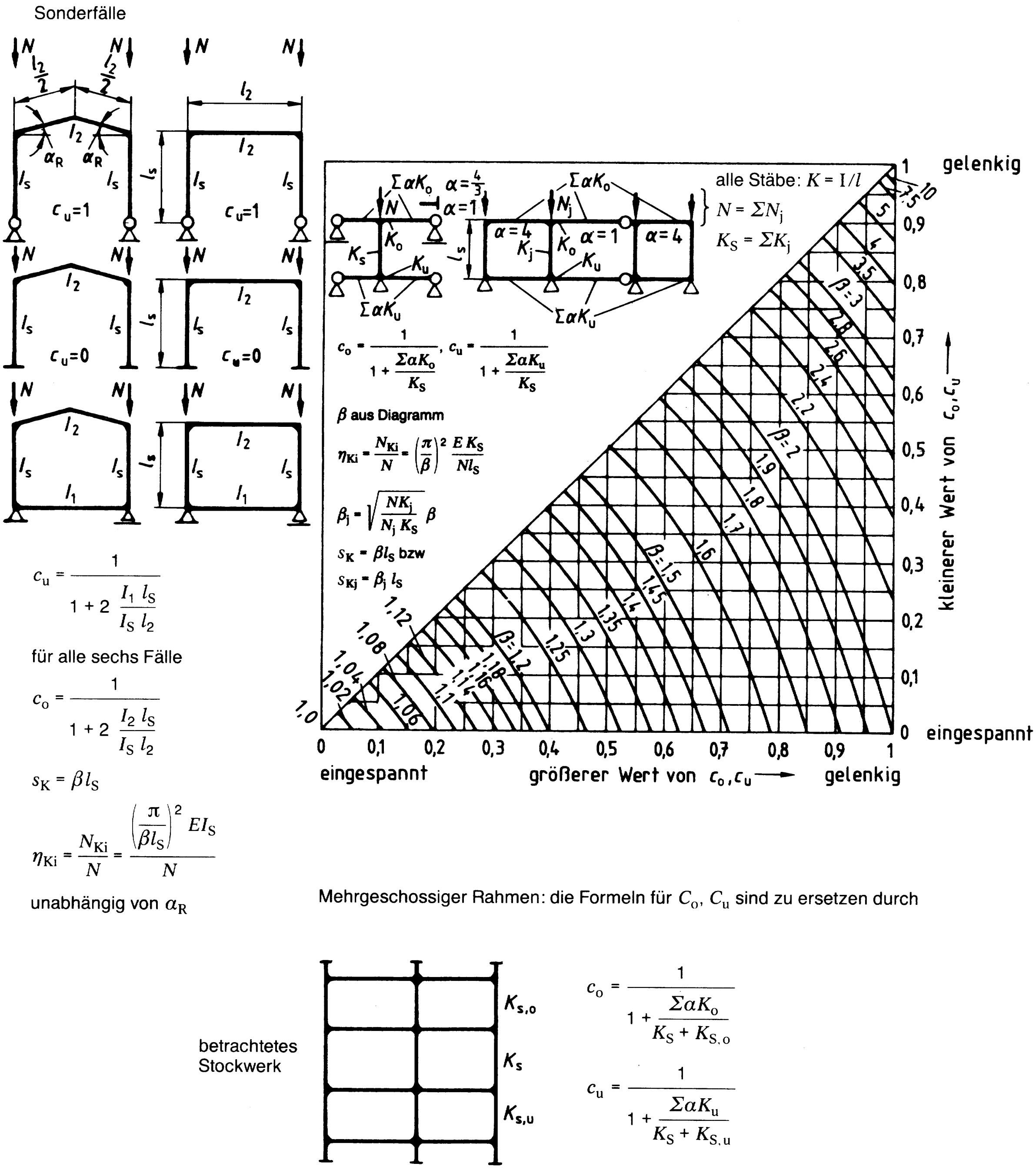


Bild 29. Diagramm zur Bestimmung des Verzweigungslastfaktors η_{Ki} und der Knicklänge s_K für Stiele verschieblicher Rahmen mit $\varepsilon_{Riegel} \leq 0,3$

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

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

B.2.1 General

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

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

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

B.2.2 Angles as web members

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

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

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

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

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

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

Draft note: Rules to be checked !.

B.2.3 Hollow sections as members

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

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

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

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

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

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

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

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

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

L is the system length of a brace member.

B.3 Torsional and torsional-flexural buckling

B.3.1 Elastic critical buckling load of compression members

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

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

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

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

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

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

G is the shear modulus

I_t is the torsion constant of the gross cross-section

I_w is the warping constant of the gross cross-section

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

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

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

ℓ_T is the effective elastic torsional buckling length.

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

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

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

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

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

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

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

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

B.3.2 Elastic critical moment for beams

B.3.2.1 Cross-sections symmetrical about the minor axis

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

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

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

I_t is the torsion constant

I_w is the warping constant

I_z is the second moment of area about the minor axis

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

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

k and k_w are effective length factors

$z_g = z_a - z_s$

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

z_a is the coordinate of the point of load application

z_s is the coordinate of the shear centre

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

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

$k = k_w = 1,0$

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

NOTE

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

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

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

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

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

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

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

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

(7) The sign convention for determining z_g is:

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

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

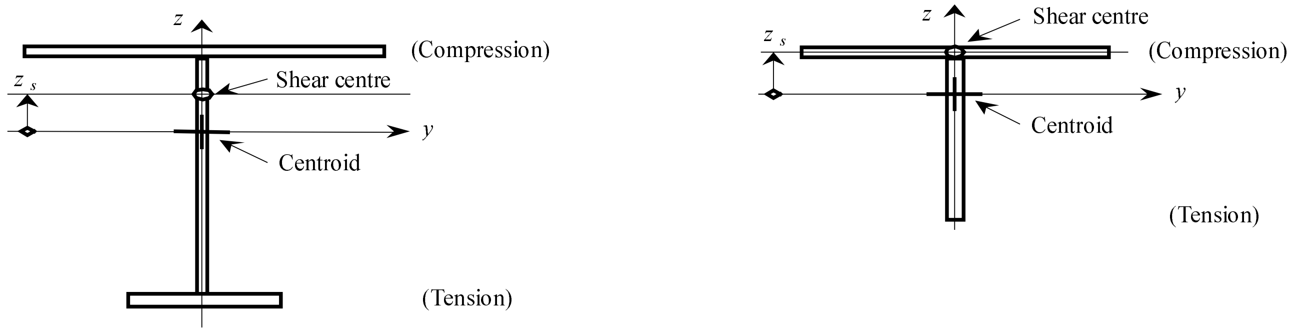


Figure B.3: Sign convention for determining z_j ((Sign convention should be changed))

Draft note: Modifications proposed by Prof. Ivan Balaz

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

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

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

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

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

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

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

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

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

for sections with a lipped compression flange:

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

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

where h_L is the depth of the lip.

B.3.2.2 Bi-symmetric cross sections

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

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

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

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

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

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

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

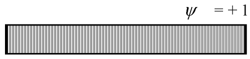
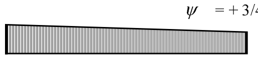
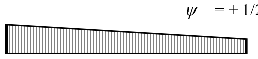
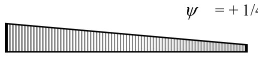
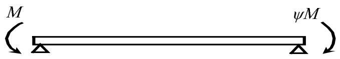
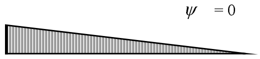
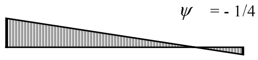
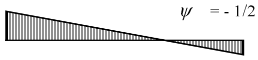
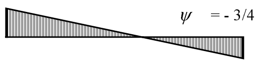
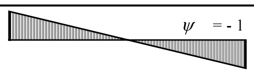
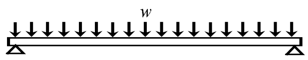
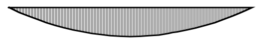
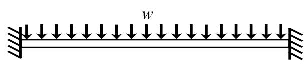
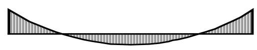
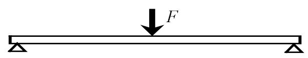
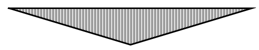
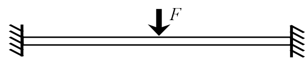
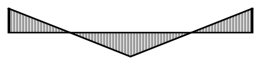
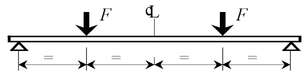
Loading and support conditions	Bending moment diagram	Values of k	Values of factors C_1	C_2	C_3
	ψ = + 1 	1,0	1,000		1,000
		0,7	1,000	-	1,113
		0,5	1,000		1,144
	ψ = + 3/4 	1,0	1,141		0,998
		0,7	1,270	-	1,565
		0,5	1,305		2,283
	ψ = + 1/2 	1,0	1,323		0,992
		0,7	1,473	-	1,556
		0,5	1,514		2,271
	ψ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

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

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

B.3.3 Minimum restraints along columns and beams

B.3.3.1 Lateral restraints

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

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

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

I_w is the warping constant

I_t is the torsion constant

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

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

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

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

B.3.3.2 Torsional restraint

Draft note: To be developed to identify product requirements.

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

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

B.4.1.1 Stable lengths between adjacent lateral restraints

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

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

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

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

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

B.4.1.2 Stable length between torsional restraints

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

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

where

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

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

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

where

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

C_m is the modification factor for linear moment gradient

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

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

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

where

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

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

B.4.2 Haunched or tapered members

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

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

where

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

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

c is the value defined in B.4.3.3

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

B.4.3.1 Linear moment gradients

(1) The modification factor Cm may be determined from

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

in which

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

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

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

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

L_t is the distance between the torsional restraints

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

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

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

where

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

B.4.3.2 Non linear moment gradients

(1) The modification factor C_n may be determined from

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

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

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

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

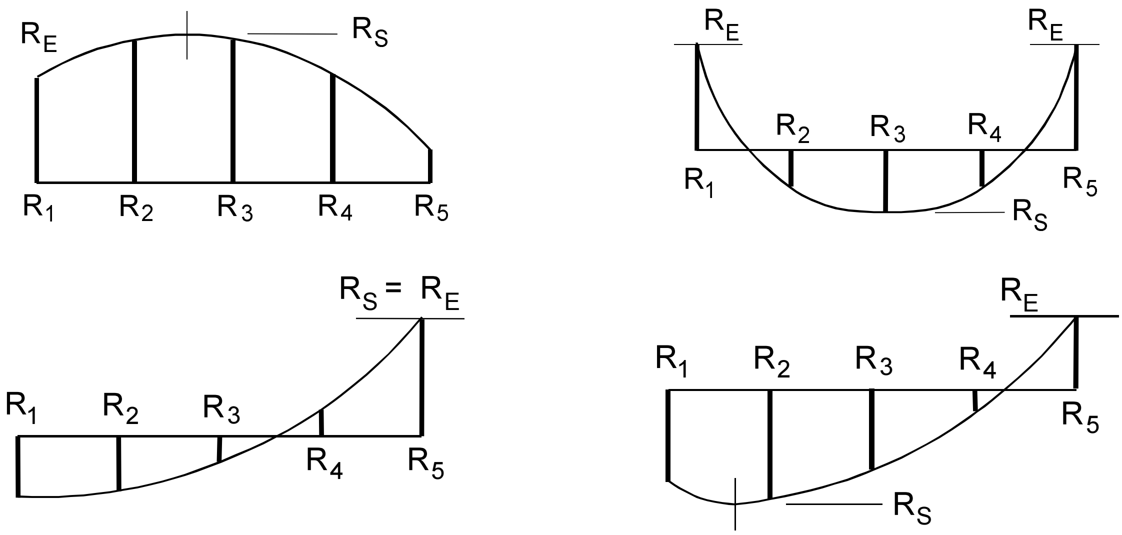


Figure B.4: Moment ratios

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

$$R = \frac{M_{y.Sd}}{f_y W_{y.el.c}} \tag{B.23}$$

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

B.4.3.3 Taper factor

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

– for tapered members or segments:

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

– for haunched members or segments:

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

where B is the breadth of the minimum depth cross-section;

D_h is the additional depth of the haunch or taper, see Figure B.5;

D_{max} is the maximum depth of cross-section within the length L_y , see Figure B.5;

D_{min} is the minimum depth of cross-section within the length L_y , see Figure B.5;

D_s is the vertical depth of the un-haunched section, see Figure B.5;

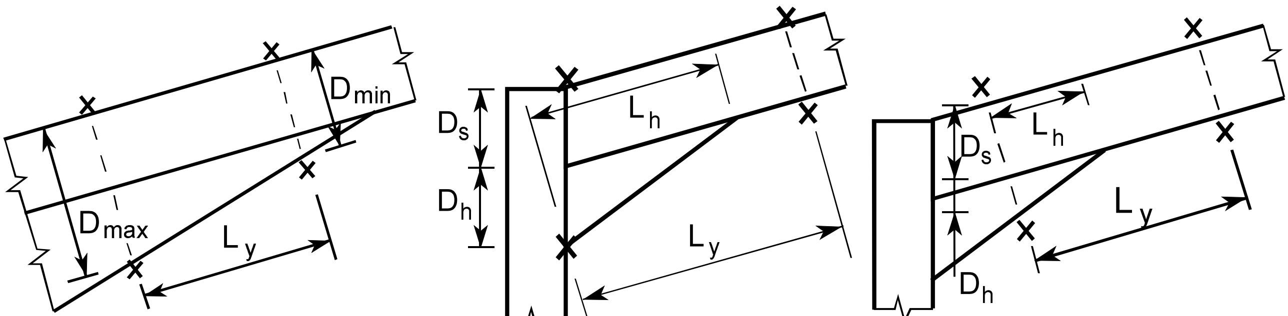
L_h is the length of haunch within the length L_y , see Figure B.5;

L_y is the length between points at which the compression flange is laterally restrained;

x is the torsional index of the minimum depth cross-section, see x.x.x.x.

Draft note: Torsional index to be defined.

x = restraint



(a) Tapered segment (b) Haunched segment (c) Haunched segment

Figure B.5: Dimensions defining taper factor

EUROPEAN STANDARD
NORME EUROPÉENNE
EUROPÄISCHE NORM

prEN 1993-5 : 2004

July 2004

UDC

Descriptors:

English version

Stage 49

Eurocode 3 : Design of steel structures

Part 5 : Piling

Calcul des structures en acier

Partie 5 :

Pieux et palplanches

Bemessung und Konstruktion von Stahlbauten

Teil 5 :

Pfähle und Spundwände

CEN

European Committee for Standardisation
Comité Européen de Normalisation
Europäisches Komitee für Normung

Central Secretariat: rue de Stassart 36, B-1050 Brussels

Ref. No. EN 1993-5 : 2004. E

Content

Page

Foreword

This European Standard EN 1993-5: Design of Steel Structures: Piling, has been prepared on behalf of Technical Committee CEN/TC250 "Structural Eurocodes", the Secretariat of which is held by BSI. CEN/TC250 is responsible for all Structural Eurocodes.
The text of the draft standard was submitted to the formal vote and was approved by CEN as EN 1993-5 on YYY-MM-DD.

No existing European Standard is superseded.

Background to the Eurocode programme

In 1975, the Commission of the European Community decided on an action programme in the field of construction, based on article 95 of the Treaty. The objective of the programme was the elimination of technical obstacles to trade and the harmonisation of technical specifications.

Within this action programme, the Commission took the initiative to establish a set of harmonised technical rules for the design of construction works which, in a first stage, would serve as an alternative to the national rules in force in the Member States and, ultimately, would replace them.

For fifteen years, the Commission, with the help of a Steering Committee with Representatives of Member States, conducted the development of the Eurocodes programme, which led to the first generation of European codes in the 1980's.

In 1989, the Commission and the Member States of the EU and EFTA decided, on the basis of an agreement[1] between the Commission and CEN, to transfer the preparation and the publication of the Eurocodes to the CEN through a series of Mandates, in order to provide them with a future status of European Standard (EN). This links de facto the Eurocodes with the provisions of all the Council's Directives and/or Commission's Decisions dealing with European standards (e.g. the Council Directive 89/106/EEC on construction products - CPD - and Council Directives 93/37/EEC, 92/50/EEC and 89/440/EEC on public works and services and equivalent EFTA Directives initiated in pursuit of setting up the internal market).

The Structural Eurocode programme comprises the following standards generally consisting of a number of Parts:

EN 1990	Eurocode:	Basis of structural design
EN 1991	Eurocode 1:	Actions on structures
EN 1992	Eurocode 2:	Design of concrete structures
EN 1993	Eurocode 3:	Design of steel structures
EN 1994	Eurocode 4:	Design of composite steel and concrete structures
EN 1995	Eurocode 5:	Design of timber structures
EN 1996	Eurocode 6:	Design of masonry structures
EN 1997	Eurocode 7:	Geotechnical design
EN 1998	Eurocode 8:	Design of structures for earthquake resistance
EN 1999	Eurocode 9:	Design of aluminium structures

1 Agreement between the Commission of the European Communities and the European Committee for Standardisation (CEN) concerning the work on EUROCODES for the design of building and civil engineering works (BC/CEN/03/89).

Eurocode standards recognise the responsibility of regulatory authorities in each Member State and have safeguarded their right to determine values related to regulatory safety matters at national level where these continue to vary from State to State.

Status and field of application of Eurocodes

The Member States of the EU and EFTA recognise that Eurocodes serve as reference documents for the following purposes:

- as a means to prove compliance of building and civil engineering works with the essential requirements of Council Directive 89/106/EEC, particularly Essential Requirement Nº1 – Mechanical resistance and stability - and Essential Requirement Nº2 - Safety in case of fire;
- as a basis for specifying contracts for construction works and related engineering services;
- as a framework for drawing up harmonised technical specifications for construction products (ENs and ETAs)

The Eurocodes, as far as they concern the construction works themselves, have a direct relationship with the Interpretative Documents[2] referred to in Article 12 of the CPD, although they are of a different nature from harmonised product standard[3]. Therefore, technical aspects arising from the Eurocodes work need to be adequately considered by CEN Technical Committees and/or EOTA Working Groups working on product standards with a view to achieving a full compatibility of these technical specifications with the Eurocodes. The Eurocode standards provide common structural design rules for everyday use for the design of whole structures and component products of both a traditional and an innovative nature. Unusual forms of construction or design conditions are not specifically covered and additional expert consideration will be required by the designer in such cases.

National Standards implementing Eurocodes

The National Standards implementing Eurocodes will comprise the full text of the Eurocode (including any annexes), as published by CEN, which may be preceded by a National title page and National foreword, and may be followed by a National annex (informative).

The National Annex (informative) may only contain information on those parameters which are left open in the Eurocode for national choice, known as Nationally Determined Parameters, to be used for the design of buildings and civil engineering works to be constructed in the country concerned, i.e.:

- values for partial factors and/or classes where alternatives are given in the Eurocode,
- values to be used where a symbol only is given in the Eurocode,
- geographical and climatic data specific to the Member State, e.g. snow map,

2 According to Art. 3.3 of the CPD, the essential requirements (ERs) shall be given concrete form in interpretative documents for the creation of the necessary links between the essential requirements and the mandates for hENs and ETAGs/ETAs.

3 According to Art. 12 of the CPD the interpretative documents shall:
(a) give concrete form to the essential requirements by harmonising the terminology and the technical bases and indicating classes or levels for each requirement where necessary;
(b) indicate methods of correlating these classes or levels of requirement with the technical specifications, e.g. methods of calculation and of proof, technical rules for project design, etc.;
(c) serve as a reference for the establishment of harmonised standards and guidelines for European technical approvals.
The Eurocodes, de facto, play a similar role in the field of the ER 1 and a part of ER 2.

- the procedure to be used where alternative procedures are given in the Eurocode,
- references to non-contradictory complementary information to assist the user to apply the Eurocode.

Links between Eurocodes and product harmonised technical specifications (ENs and ETAs)

There is a need for consistency between the harmonised technical specifications for construction products and the technical rules for works[4]. Furthermore, all the information accompanying the CE Marking of the construction products which refer to Eurocodes should clearly mention which Nationally Determined Parameters have been taken into account.

Additional information specific to EN 1993-5

EN 1993-5 gives design rules for steel sheet piling and bearing piles to supplement the generic rules in EN 1993-1.

EN 1993-5 is intended to be used with Eurocodes EN 1990 - Basis of design, EN 1991 - Actions on structures and Part 1 of EN 1997 Geotechnical Design.

Matters that are already covered in those documents are not repeated.

EN 1993-5 is intended for use by

- committees drafting design related product, testing and execution standards,
- clients (e.g. for the formulation of their specific requirements)
- designers and constructors
- relevant authorities.

Numerical values for partial factors and other parameters are recommended as basic values that provide an acceptable level of safety. They have been selected assuming that an appropriate level of workmanship and quality management applies.

Annex A and Annex B have been prepared to complement the provisions of EN 1993-1-3 for class 4 steel sheet piles.

Annex C gives guidance on the plastic design of steel sheet pile retaining structures.

Annex D gives one possible set of design rules for primary elements of combined walls.

Reference should be made to EN 1997 for geotechnical design which is not covered in this document.

National Annex for EN 1993-5

This standard gives alternative procedures, values and recommendations for classes with notes indicating where national choices may have to be made. Therefore the National Standard implementing EN 1993-5 should have a National Annex containing all Nationally Determined

4 See Art. 3.3 and Art. 12 of the CPD, as well as clauses 4.2, 4.3.1, 4.3.2 and 5.2 of ID 1.

Parameters to be used for the design of buildings and civil engineering works to be constructed in the relevant country.

National choice is allowed in EN 1993-5 through clauses:

1.1 (11)
3.7 (1)P
3.9 (1)P
4.4 (1)
5.1.1 (4)
5.2.2 (2)
5.2.2 (13)
5.2.5 (7)
5.5.4 (2)P
6.4 (3)
7.1 (4)
7.2.3 (2)
7.4.2 (4)
A.3.1 (3)
B.5.4 (1)
D.2.2 (5)

1 General

1.1 Scope

(1)P Part 5 of EN 1993 provides principles and application rules for the structural design of bearing piles and sheet piles made of steel.

(2) It also provides examples of detailing for foundation and retaining wall structures.

(3)P The field of application includes:

- steel piled foundations for civil engineering works on land and over water;
- temporary or permanent structures needed to carry out steel piling work;
- temporary or permanent retaining structures composed of steel sheet piles, including all kinds of combined walls.

(4)P The field of application excludes:

- offshore platforms;
- dolphins.

(5) Part 5 of EN 1993 also includes application rules for steel piles filled with concrete.

(6) Special requirements for seismic design are not covered. Where the effects of ground movements caused by earthquakes are relevant see EN 1998.

(7) Design provisions are also given for walings, bracing and anchorages, see section 7.

(8) The design of steel sheet piling using class 1, 2 and 3 cross-sections is covered in sections 5 and 6, whereas the design of class 4 cross-sections is covered in annex A.

NOTE: The testing of class 4 sheet piles is covered in annex B.

(9) The design procedures for crimped U-piles and straight web steel sheet piles utilise design resistances obtained by testing. Reference should be made to EN 10248 for testing procedures.

(10) Geotechnical aspects are not covered in this document. Reference is made to EN 1997.

(11) Provisions for taking into account the effects of corrosion in the design of piling are given in section 4.

NOTE: Numerical values for corrosion rates may be given in the National Annex.

(12) Allowance for plastic global analysis in accordance with 5.4.3 of EN 1993-1-1 is given in 5.2.

NOTE: Guidance for the design of steel sheet pile walls allowing for plastic global analysis is given in Annex C.

(13) The design of combined walls at ultimate limit states is covered in section 5 including general provisions for the design of primary elements.

> **NOTE:** Guidance for the design of both tubular piles and I-sections used as primary elements is given in Annex D.

1.2 Normative references

This European Prestandard incorporates by dated or undated reference, provisions from other publications. These normative references are cited at the appropriate places in the text and the publications are listed hereafter. For dated references, subsequent amendments to, or revisions of, any of these publications apply to this European Prestandard only when incorporated in it by amendment or revision. For undated references the latest edition of the publication referred to applies.

EN 1990 Eurocode: Basis of structural design

EN 1991 Eurocode 1: Actions on structures

EN 1992 Eurocode 2: Design of concrete structures

EN 1993 Eurocode 3: Design of steel structures

- Part 1.1: General rules: General rules and rules for buildings;
- Part 1.2: General rules: Structural fire design;
- Part 1.3: General rules: Supplementary rules for cold formed thin gauge members and sheeting;
- Part 1.5: General rules: Plated structural elements;
- Part 1.6: General rules: Strength and stability of shell structures
- Part 1.8: General rules: Design of joints
- Part 1.9: General rules: Fatigue
- Part 1.10: General rules: Material toughness and through-thickness properties
- Part 1.11: General rules: Design of structures with tension components made of steel

EN 1994 Eurocode 4: Design of composite steel and concrete structures

EN 1997 Eurocode 7: Geotechnical design

EN 1998 Eurocode 8: Earthquake resistant design of structures;

EN 10002 Metallic materials; tensile testing;

EN 10025 Hot rolled products of non-alloy structural steels - Technical delivery conditions;

EN 10027 Designation systems for steel;

EN 10113 Hot rolled products in weldable fine grain structural steels;

EN 10137	Plates and wide flats made of high yield strength structural steels in the quenched and tempered or precipitation hardened conditions;
EN 10210	Hot finished structural hollow sections of non-alloy fine grain structural steels;
EN 10219	Cold formed structural hollow sections of non-alloy fine grain structural steels;
EN 10248	Hot rolled sheet piling of non alloy steels;
EN 10249	Cold formed sheet piling of non alloy steels;
EN 1536	Execution of special geotechnical work - Bored piles;
EN 1537	Execution of special geotechnical work - Ground anchors;
EN 12063	Execution of special geotechnical work - Sheet-pile walls;
EN 12699	Execution of special geotechnical work - Displacement piles;
EN 14199	Execution of special geotechnical work - Micro piles;
EN 10045	Metallic materials; Charpy impact test;

1.3 Assumptions

(1)P In addition to the general assumptions in EN 1990 the following assumptions apply:

Installation and fabrication of steel piles and steel sheet piles are in accordance with EN 12699, EN 14199 and EN 12063.

1.4 Distinction between principles and application rules

(1)P Reference shall be made to 1.4 of EN 1993-1-1.

1.5 Definitions

For the purpose of this standard, the following definitions apply:

1.5.1 foundation: Part of a construction work including piles and possibly their pile cap.

1.5.2 retaining structure: A construction element including walls retaining soil, similar material and/or water, and, where relevant, their support systems (e.g. anchorages).

1.5.3 soil-structure interaction: The mutual influence of deformations on soil and a foundation or a retaining structure.

1.6 Symbols

(1) In addition to those given in EN 1993-1-1, the following main symbols are used:

c Slant height of the web of steel sheet piles, see Figure 5-1;

α Inclination of the web, see Figure 5-1.

(2) In addition to those given in EN 1993-1-1, the following subscripts are used:

red Reduced.

(3) In addition to those given in EN 1993-1-1, the following major symbols are used:

A_v Projected shear area, see Figure 5-1;

F_{Ed} Design value of the anchor force;

$F_{Q,Ed}$ Additional horizontal force resulting from global buckling to be resisted by the toe of a sheet pile to allow for the assumption of a non-sway buckling mode, see Figure 5-4;

$F_{t,Rd}$ Design tension resistance of an anchor;

$F_{t,Ed}$ Design value of the circumferential tensile force in a cellular cofferdam;

$F_{t,ser}$ Axial force in an anchor under characteristic loading;

$F_{ta,Ed}$ Design tensile force in the arc cell of a cellular cofferdam;

$F_{tc,Ed}$ Design tensile force in the common wall of a cellular cofferdam;

$F_{tg,Rd}$ Design tensile resistance of shafts of anchors;

$F_{tm,Ed}$ Design tensile force in the main cell of a cellular cofferdam;

$F_{ts,Rd}$ Design tensile resistance of simple straight web steel sheet piles;

$F_{tt,Rd}$ Design tensile resistance of threads of anchors;

$R_{c,Rd}$ Design resistance of a sheet pile to a local transverse force;

$R_{tw,Rd}$ Design tensile resistance of the webs of a sheet pile to the introduction of a local transverse force;

$R_{Vf,Rd}$ Design shear resistance of the flange of a sheet pile to the introduction of a local transverse force;

$p_{m,Ed}$ Design value of the internal pressure acting in the main cell of a cellular cofferdam;

r_a Initial radius of the arc cell in a cellular cofferdam;

r_m Initial radius of the main cell in a cellular cofferdam;

t_f Nominal flange thickness of a steel sheet pile;

t_w Nominal web thickness of steel sheet piles;

β_B Factor accounting for the possible reduction of the section modulus of U-piles due to insufficient shear force transmission in the interlocks;

β_D Factor accounting for the possible reduction of the bending stiffness of U-piles due to insufficient shear force transmission in the interlocks;

β_R Factor accounting for the interlock resistance of straight web steel sheet piles;

β_T Factor accounting for the behaviour of a welded junction pile at ultimate limit states;

$\beta_{o,I}$ Factor accounting for the reduction of the second moment of area about the wall axis due to the ovalisation of the tube

ρ_P Factor accounting for the effects of differential water pressure on transverse local plate bending.

(4) Further symbols are defined where they first occur.

1.7 Units

(1)P S.I. units shall be used in accordance with ISO 1000.

(2) The following units are recommended for use in calculations:

- forces and loads: kN, kN/m, kN/m^2;
- unit mass: kg/m^3;
- unit weight: kN/m^3;
- stresses and strengths: N/mm^2 (MN/m^2 or MPa);
- bending moments: kNm;
- torsional moments: kNm.

1.8 Terminology

For the purposes of this Prestandard, the following terminology is used:

NOTE: Figure 1-1 to Figure 1-10 are only examples and are provided in order to enhance the understanding of the wording of the terminology used. The examples are by no means exhaustive and they do not represent any preferred detailing.

1.8.1 Anchorage

The general expression used to describe the anchoring system at the back of a retaining wall, such as dead-man anchors, anchor plates or anchor screens, screw anchors, ground anchors, anchor piles and expanded bodies. Examples of connections between anchors and a sheet pile wall are shown in Figure 1-1.

1.8.2 Anchored wall

A wall whose stability depends upon penetration of the sheet piling into the ground and also upon one or more anchor levels.

1.8.3 Bearing piles

Structural elements (hollow type, H-type, cruciform or X-type cross-sections) incorporated into the foundations of building or civil engineering works and used for resisting axial compressive or tensile forces, moments and transverse (shear) forces (see Table 1-1). The bearing resistance is achieved by base resistance or shaft friction or a combination of both.

1.8.4 Bracing

Struts perpendicular or at an angle to the front face of a retaining wall, supporting the wall and usually connected to the walings (see Figure 1-2).

1.8.5 Cantilever wall

Wall whose stability depends solely upon the penetration of the sheet piling into the ground.

1.8.6 Cellular cofferdams

Cofferdams constructed of straight web profiles with interlock tensile strength sufficient to resist the circumferential tension developed in the cellular walls due to the radial pressure of the contained fill (see Figure 1-3). The stability of these cells is obtained by the self-weight of the fill. Two basic types of cellular cofferdams are:

- Cellular cofferdams involving circular cells: This type of cofferdam consists of individual cells of large diameter connected together by arcs of smaller diameter (see Figure 1-4a);

- Cellular cofferdams involving diaphragm cells: This type of cofferdam consists of two rows of circular arcs connected together by diaphragms perpendicular to the axis of the cofferdam (see Figure 1-4b).

1.8.7 Combined walls

Retaining walls composed of primary and secondary elements. The primary elements are normally steel tubular piles, I-sections or built up box types, spaced uniformly along the length of the wall. The secondary elements are generally steel sheet piles of various types installed in the spaces between the primary elements and connected to them by interlocks (see Figure 1-5).

1.8.8 Double U-pile

Two threaded single U sheet piles with a crimped or welded common interlock allowing for shear force transmission.

1.8.9 Driveability

The ability of a sheet pile or bearing pile to be driven through the ground strata to the required penetration depth without detrimental effects.

1.8.10 Driving
Any method for installing a pile into the ground to the required depth, such as impact driving, vibrating, pressing or screwing or by a combination of these or other methods.

1.8.11 High modulus wall
A high strength retaining wall formed by interlocking steel elements that have the same geometry. The elements may consist of fabricated profiles, see Figure 1-6, to obtain a high section modulus.

1.8.12 Interlock
The portion of a steel sheet pile or other sheeting that connects adjacent elements by means of a thumb and finger or similar configuration to make a continuous wall. Interlocks may be described as

- Free: Threaded interlocks that are neither crimped nor welded;
- Crimped: Interlocks of threaded single piles that have been mechanically connected by crimped points;
- Welded: Interlocks of threaded single piles that have been mechanically connected by continuous or intermittent welding.

1.8.13 Jagged wall
Special sheet pile wall configuration in which the single piles are arranged either to enhance the moment of inertia of the wall (see example in Figure 1-7) or to suit special applications (see example in Figure 1-8).

1.8.14 Pile coupler
A mechanical friction sleeve used to lengthen a steel tubular or X shaped pile.

1.8.15 Propped wall
A retaining wall whose stability depends upon penetration of the sheet piling into the ground and also upon one or more levels of bracing.

1.8.16 Soldier or king pile wall
Soldier or king pile walls consist of vertical piles (king, master or soldier piles) driven at intervals, supporting intermediate horizontal elements (boarding, planks or lagging), see Figure 1-9. The king or master piles may be rolled or welded I-sections, tubular or box sections.

1.8.17 Steel box piles
Piles with a non-circular hollow shape formed from two or more hot-rolled sections continuously or intermittently welded together in longitudinal direction (see Table 1-1).

1.8.18 Steel tubular piles
Piles of circular cross-section formed by the seamless, longitudinal or helical welding processes (see Table 1-1).

1.8.19 Steel sheet pile
The individual steel elements of which a sheet pile wall is composed. The types of steel sheet piles covered in this Part 5 are given in Table 1-2: Z-shaped, U-shaped and straight web profiles, and in Table A-1 of Annex A for cold formed sheet piling. The interlocks of the Z-piles are located on the extreme fibres of the wall, whereas the interlocks of U-shaped and straight web profiles are located on the axis of the retaining wall.

1.8.20 Steel sheet pile wall
The screen of sheet piles that forms a continuous wall by threading of the interlocks.

1.8.21 T-connection
Special element, see Figure 1-10, to connect two cellular cofferdams by arcs of smaller diameter, see Figure 1-3.

1.8.22 Triple U-pile
A sheet pile consisting of three threaded single U sheet piles with two crimped or welded common interlocks allowing for shear force transmission.

1.8.23 Waling
Horizontal beam, usually of steel or reinforced concrete, fixed to the retaining wall and used to transmit the design support force for the wall into the tie rods or struts.

Table 1-1: Examples of cross-sections of steel bearing piles

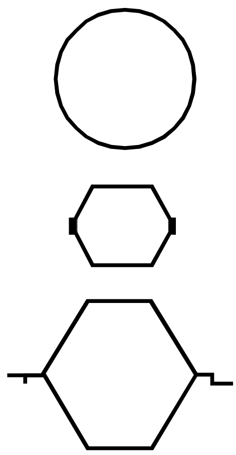
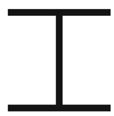
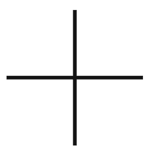
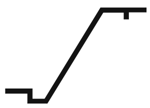
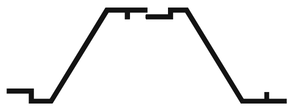
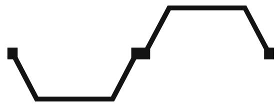
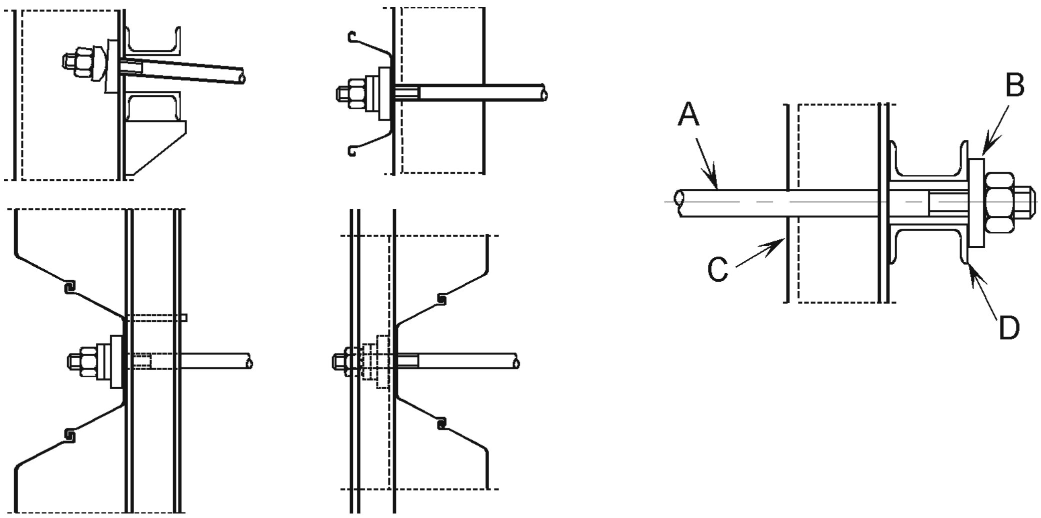
Type of cross-section	Representation
Hollow type (examples), *see Note*	
H type	
X type	
Note: Reference should be made to EN 12699 and EN 14199 for execution details.	

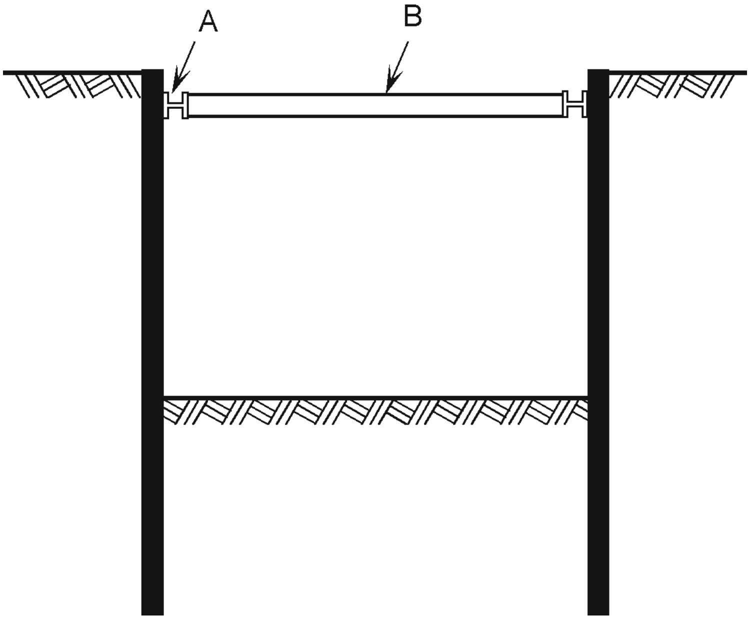
Table 1-2: Steel sheet piles

Type of cross-section	Single pile	Double pile
Z - profiles		
U - profiles		
Straight web profiles		
Note: Reference should be made to EN 10248 for details of the interlocks.		



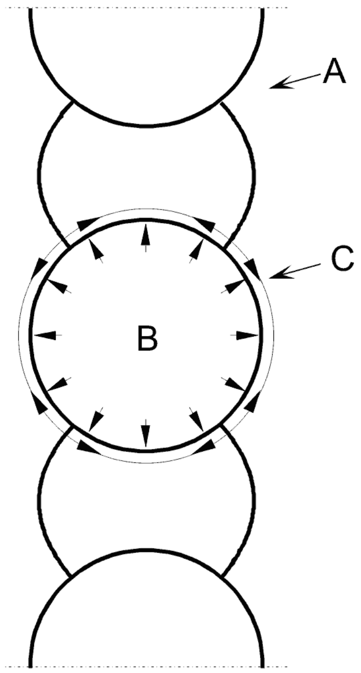
A Tie rod; B Bearing plate;
C Sheet pile; D Waling

Figure 1-1: Examples of connections between anchors and sheet pile walls



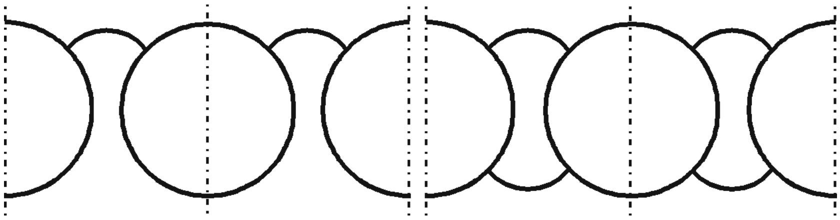
A Waling; B Strut

Figure 1-2: Example of bracing

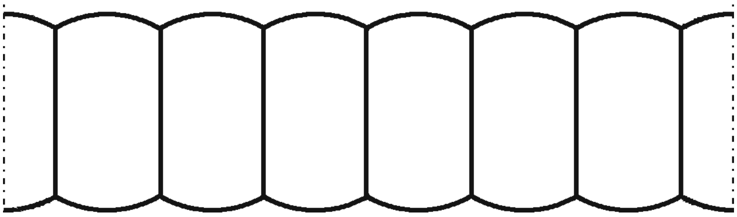


A T-junction; B Internal pressure;
C Circumferential tensile force

Figure 1-3: Cellular cofferdams

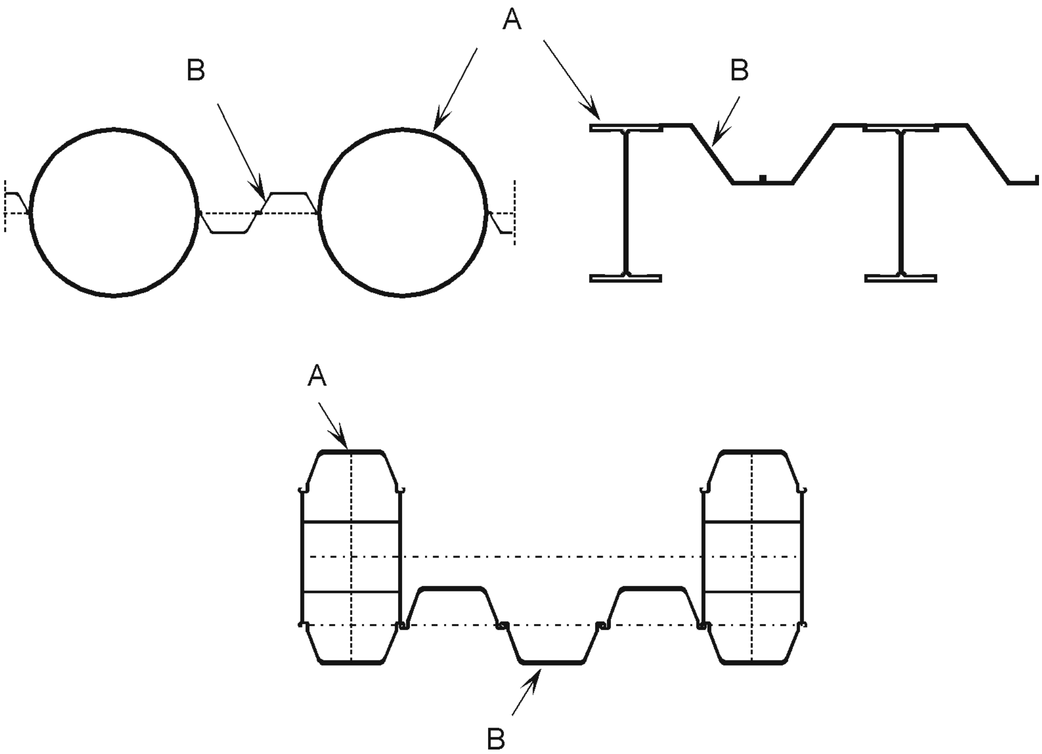


a) Structure formed with circular cells



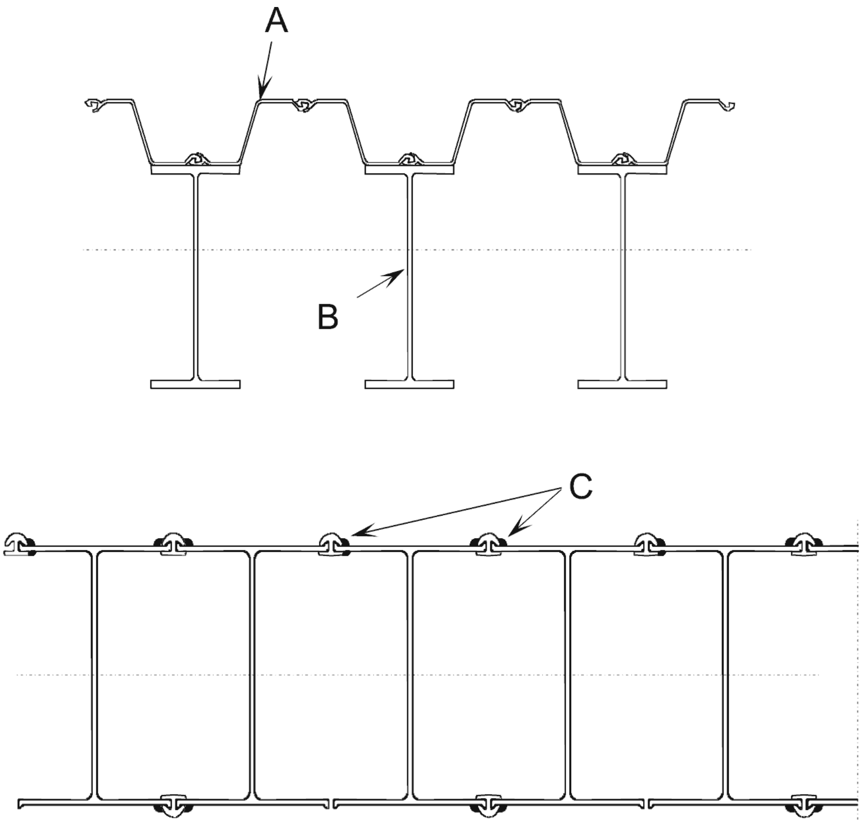
b) Structure formed with diaphragm cells

Figure 1-4: Examples of cellular structures



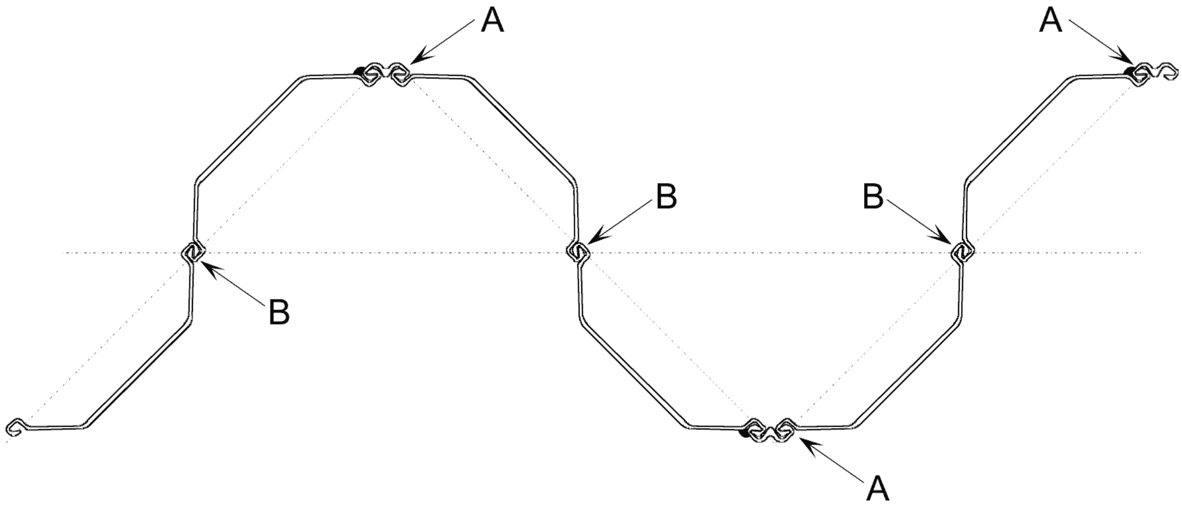
A Primary Elements; B Secondary Elements

Figure 1-5: Examples of combined walls



A Sheet pile welded to I-Section;
B I-section;
C Connector welded to I-Section

Figure 1-6: Examples of high modulus walls



A Connector welded to one double pile; B Crimped Interlock

Figure 1-7: Example of a jagged wall formed from U-profiles

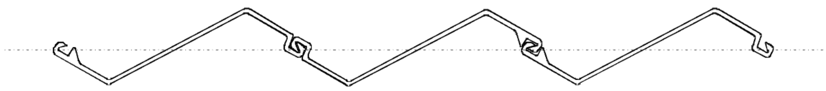
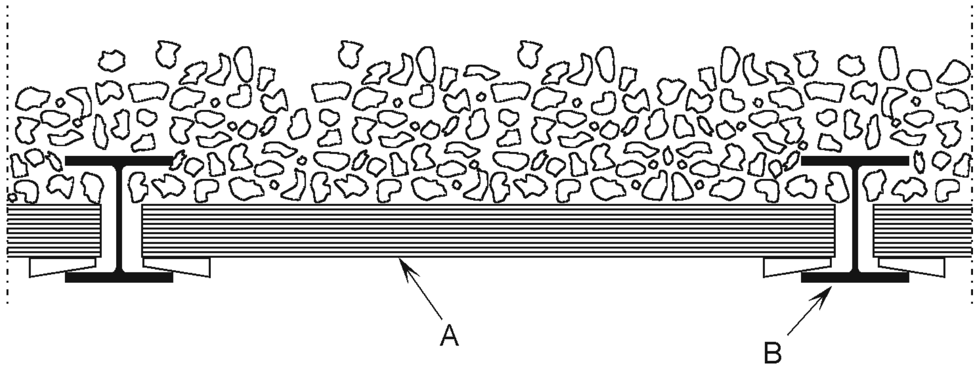
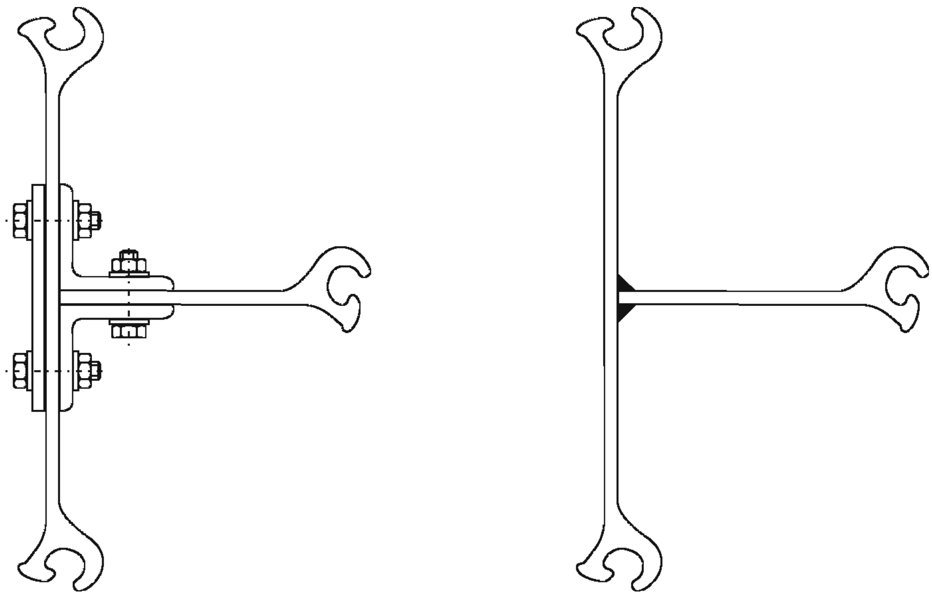


Figure 1-8: Example of a jagged wall formed from Z-profiles



A Lagging, boarding, planks; B Soldier, king or master pile

Figure 1-9: Example of a soldier pile wall



a) Bolted b) Welded

Figure 1-10: Examples of T-connections

1.9 Convention for sheet pile axes

(1) For sheet piling the following axis convention is used:

- generally
  - x - x is the longitudinal axis of a pile;
  - y - y is the principal axis nearest to the plane of the retaining wall;
  - z - z is the other principal axis;
- where necessary
  - u - u is the cross-sectional axis parallel to the retaining wall, if this does not coincide with y-y;
  - v - v is the other cross-sectional axis if this does not coincide with z-z.

NOTE: This differs from the axis convention used in EN 1993-1-1. Care therefore needs to be taken when cross-reference is made to Part 1.1.

2 Basis of design

2.1 General

(1)P For the design of bearing piles and sheet piling, including the design of walings, bracing and anchorages, the provisions of EN 1990 apply, except where different provisions are given in this document.

(2) In the following, specific provisions are given for the design of bearing piles and sheet piling to fulfil the safety and durability requirements for both serviceability and ultimate limit states.

(3)P The bearing resistance of the ground shall be determined according to EN 1997-1.

(4)P All design situations, including each stage of execution and use, shall be taken into account, see EN 1990.

(5)P The driveability of bearing piles and sheet piles shall be taken into account in the design of the structure, see 2.7.

(6)P The provisions given in this document apply equally to temporary and permanent structures, unless otherwise stated, see EN 1990.

(7) In the following distinction is made between bearing piles and retaining walls where relevant.

(8) For provisions regarding walings, bracing, connections and anchors, reference should be made to section 7.

2.2 Ultimate limit state criteria

(1)P The following ultimate limit state criteria shall be taken into account:

a) failure of the construction by failure in the soil (the soil resistance is exceeded);

b) structural failure;

c) combination of failure in the soil and structural failure.

NOTE: Failure of adjacent structures might be caused by deformations resulting from excavation. If adjacent structures are sensitive to such deformations, recommendations for dealing with the situation can be given in the project specification.

(2)P Verifications related to ultimate limit state criteria shall be carried out in accordance with EN 1997-1.

(3) Depending on the design situation the resistance to one or more of the following modes of structural failure should be verified:

- for bearing piles:
    - failure due to bending and/or axial force;
    - failure due to overall flexural buckling, taking account of the restraint provided by the ground and by the supported structure at the connections to it;
    - local failure at points of load application;
    - fatigue.
- for retaining walls:
    - failure due to bending and/or axial force;
    - failure due to overall flexural buckling, taking account of the restraint provided by the soil;
    - local buckling due to overall bending;
    - local failure at points of load application (e.g. web crippling);
    - fatigue.

2.3 Serviceability limit state criteria

(1) Unless otherwise specified, the following serviceability limit state criteria should be taken into account:

- for bearing piles:
    - limits to vertical settlements or horizontal displacements necessary to suit the supported structure;
    - vibration limits necessary to suit structures directly connected to, or adjacent to, the bearing piles.
- for retaining walls:
    - deformation limits necessary to suit the serviceability of the retaining wall itself;
    - limits to horizontal displacements, vertical settlements or vibrations, necessary to suit structures directly connected to, or adjacent to, the retaining wall itself.

(2) Values for the limits given in (1), in relation to the combination of actions to be taken into account according to EN 1990, should be defined in the project specification.

(3) Values for limits imposed by adjacent structures should be defined in the project documentation. Guidance for determining such limits is given in EN 1997-1.

NOTE: Serviceability criteria might be the governing criteria for the design.

2.4 Site investigation and soil parameters

(1)P Parameters for soil and/or backfill shall be determined from geotechnical investigation in accordance with EN 1997.

2.5 Analysis

2.5.1 General

(1) Global analysis should be carried out to determine the effects of actions (internal forces and moments, stresses, strains and displacements) over the whole or part of the structure. Additional local analyses of the structure should be carried out where necessary, e.g. load application points, connections etc.

(2) Analyses may be carried out using idealisations of the geometry, behaviour of the structure and behaviour of the soil. The idealisations should be selected with regard to the design situation.

(3) Except where the design is sensitive to the effects of variations, assessment of the effects of actions in piled foundations and in sheet pile walls may be carried out on the basis of nominal values of geometrical data.

(4)P Structural fire design shall be taken into account through the provisions of EN 1993-1-2 and EN 1991-1-2.

2.5.2 Assessment of actions

(1)P Where relevant, actions shall be taken from EN 1991, otherwise from the project documentation.

(2)P In the case of piled foundations, actions due to vertical or transverse ground movements (e.g. down-drag, etc.) shall be assessed in accordance with EN 1997-1.

(3)P The actions transmitted to the structure through the soil shall be assessed by using models selected in accordance with EN 1997-1, or defined in the project documentation.

(4)P Where necessary, the effects of actions resulting from variations in temperature with time, or from special loads not specified in EN 1991, shall be taken into account.

NOTE 1: It might be necessary to take into account temperature effects, for example on struts, if there are likely to be large variations in temperature. The design might prescribe measures to reduce the influence of temperature variations.

NOTE 2: Examples of special loads are:

- loads due to falling objects or swinging buckets;
- loads from excavators and cranes;
- imposed loads such as pumps, access ways, intermediate struts, staging for materials or stacking of bundles of steel reinforcement.

(5) Unless otherwise specified, for retaining walls subject to loads from a road or a railway track, simplified models for such loads (for example uniformly distributed loads) derived from those defined for bridges may be used, see EN 1991-2.

2.5.3 Structural analysis

2.5.3.1 General

(1)P The analysis of the structure shall be carried out using a suitable soil-structure model in accordance with EN 1997-1.

(2) Depending on the design situation, anchors may be modelled either as simple supports or as springs.

(3)P If connections have a major influence on the distribution of internal forces and moments, they shall be taken into account in the structural analysis.

2.5.3.2 Ultimate limit states

(1) The structural analysis of piled foundations for ultimate limit states may be based on the same type of model as used for serviceability limit states.

(2) Where accidental situations need to be taken into account, the assessment of effects of actions in the piles in a foundation may be carried out on the basis of a plastic model, both for the whole structure and for the soil-structure interaction.

NOTE: An example of an accidental situation is a ship collision against a bridge pier.

(3)P Assessment of the effects of actions in sheet pile retaining walls shall be carried out on the basis of the relevant failure mode for ultimate limit state verifications, using a soil structure interaction model as defined in 2.5.3.1 (1)P.

2.5.3.3 Serviceability limit states

(1)P For sheet pile retaining walls, and also for piled foundations, the global analysis shall be based on a linear elastic model of the structure, and a soil-structure model as defined in 2.5.3.1(1)P.

(2)P It shall be shown that no plastic deformations occur in the structure as a result of serviceability loading.

2.6 Design assisted by testing

2.6.1 General

(1) The general provisions for design assisted by testing given in EN 1990, EN 1993-1-1 and EN 1997-1 should be satisfied.

> **NOTE** Guidance on the determination of design resistance from tests is given in Annex D of EN 1990.

2.6.2 Bearing piles

(1) For guidance on the testing of bearing piles, reference should be made to EN 1997-1, EN 12699 and EN 14199.

2.6.3 Steel sheet piling

(1) The assumptions made in the design of sheet piling may be verified in stages by on-site testing during execution of the work (for instance in the case of an excavation procedure).

(2) Reference should be made to EN 1997-1 for calibration of a calculation model and modification of the design during execution.

2.6.4 Anchorages

(1) The general provisions for design assisted by testing given in EN 1997-1 , EN 1537 and EN 1993-1-11 should be followed for anchorages.

2.7 Driveability

(1)P In the design of all piles (bearing piles or sheet piles), the practical aspects of installing the piles to the required penetration depth shall be taken into account. Reference shall be made to EN 12063 and to EN 12699 and EN 14199.

(2)P The type, size and detailing of the piles shall be chosen, in combination with the effectiveness of the piling plant used for installation and extraction, and the driving procedure (driving parameters), to be suitable for the ground conditions through which the piles have to be driven.

(3)P If pile points, stiffeners or friction reducers are used as an aid to driving or to strengthen the piles during installation, their effects on the performance of the piles under service conditions shall be taken into account.

3 Material properties

3.1 General

(1)P This Part 5 of EN 1993 shall be used for the design of piles and retaining walls fabricated from steel conforming with the standards referred to in 3.2 to 3.9.

(2) This document may also be used for other structural steels, provided that adequate data exist to justify application of the relevant design and fabrication rules. Test procedures and test evaluation should conform with section 2 of EN 1993-1-1 and EN 1990 and the test requirements should align with those given in the relevant standards mentioned in 3.2 to 3.9.

(3)P Re-used and second quality piles shall as a minimum comply with the requirements concerning geometrical and material properties specified in the design and shall be free from damage and deleterious matters that would affect strength and durability.

3.2 Bearing piles

(1)P Reference shall be made to EN 1993-1-1 for steel properties.

(2)P For the properties of steel piles fabricated from steel sheet piles see 3.3 or 3.4.

3.3 Hot rolled steel sheet piles

(1)P Hot rolled steel sheet piles shall be in accordance with EN 10248.

(2)P Nominal values of the yield strength f_y and the ultimate tensile strength f_u for hot rolled steel sheet piles shall be obtained from Table 3-1, which are the minimum values given in EN 10248-1.

(3) Reference should be made to 3.2.2 of EN 1993-1-1 for ductility requirements.

NOTE: The steel grades listed in Table 3-1 are accepted as satisfying these requirements.

Table 3-1: Nominal values of yield strength f_y and ultimate tensile strength f_u for hot rolled steel sheet piles according to EN 10248-1

Steel name to EN 10027	f_y [N/mm^2]	f_u [N/mm^2]
S240 GP	240	340
S270 GP	270	410
S320 GP	320	440
S355 GP	355	480
S390 GP	390	490
S430 GP	430	510

3.4 Cold formed steel sheet piles

(1)P Cold formed steel sheet piles shall be in accordance with EN 10249.

(2)P Nominal values for the basic yield strength f_{yb} and the ultimate tensile strength f_u for cold formed steel sheet piles shall be obtained from Table 3-2 which is in accordance with EN 10249-1.

> **NOTE:** The basic yield strength f_{yb} is the nominal yield strength of the basic steel used for cold forming.

(3) Reference should be made to A.3.1 for ductility requirements.

Table 3-2: Nominal values of basic yield strength f_{yb} and ultimate tensile strength f_u for cold formed steel sheet piles according to EN 10249-1

Steel name to EN 10027	f_{yb} [N/mm^2]	f_u [N/mm^2]
S235 JRC	235	340
S275 JRC	275	410
S355 JOC	355	490

3.5 Sections used for waling and bracing

(1)P Reference shall be made to 3.1 and 3.2 of EN 1993-1-1 for properties of steels used for walings and bracing.

3.6 Connecting devices

(1)P Reference shall be made to EN 1993-1-8 for properties of bolts, nuts and washers and of welding consumables.

3.7 Steel members used for anchors

(1)P Reference shall be made to EN 1537 for anchors made from high strength steel with a specified minimum yield strength > $f_{y,spec}$.

> **NOTE:** The value $f_{y,spec}$ may be given in the National Annex. The value $f_{y,spec}$ = 500 N/mm² is recommended.

(2)P Reference shall be made to 3.2.1, 3.2.2 of EN 1993-1-1 and 3.9 of EN 1993-5 for the material properties of anchors made of non-high strength steel.

3.8 Steel members used for combined walls

(1)P Steel properties of special I-section piles used as the primary elements of combined walls shall be in accordance with EN 10248.

(2)P Tubes used as the primary elements in combined walls shall conform with EN 10210 or EN 10219.

(3)P Steel properties of built up box piles used as the primary elements of combined walls shall satisfy the requirements given in 3.2.

(4)P Steel properties of the secondary elements used for combined walls shall satisfy the requirements given in 3.3 or 3.4 respectively.

(5)P Hot rolled connecting devices for sheet piles shall be in accordance with EN 10248.

3.9 Fracture toughness

(1)P The material shall have sufficient toughness to avoid brittle fracture at the lowest service temperature expected to occur within the intended life of the structure.

> **NOTE:** The lowest service temperature to be taken into account may be given in the National Annex.

(2) For sheet piling with a flange thickness not more than 25mm, steels with values of T_{27J} according to Table 3-3 may be used, provided that the lowest service temperature is not lower than -30°C.

> **NOTE 1:** For other cases reference can be made to EN 1993-1-10.

> **NOTE 2:** The T_{27J} value is the test temperature at which an impact energy $K_V(T) > 27$ Joule is required to fracture a Charpy-V-notch specimen. For the test see EN 10045.

Table 3-3: Fracture toughness T_{27J} of steel sheet piles

Yield strength f_y [N/mm^2]		240	270	320	355	390	430
Values of T_{27J}	for lowest service temperature of -15°C	35°	35°	35°	15°	15°	15°
	for lowest service temperature of -30°C	20°	20°	20°	0°	0°	0°

Notes:

1) If there are holes (e.g. for anchors) in a flange stressed in tension, the reduction of the cross-sectional resistance should be taken into account by using a reduced yield strength or an effective cross-sectional area.

2) These values have been calculated for a lowest service temperature and a flange thickness of not more than 25mm without taking into account dynamic effects. For a flange thickness $25 < t_f \leq 30$ mm the values given in the table for T_{27J} should be reduced by 5° for lowest service temperature of –15° C and by 10° for lowest service temperature of –30° C.

3) Higher toughness requirements might be necessary if driving of the piles is foreseen in hard soils at temperatures below -10° C.

4 Durability

4.1 General

(1)P Dependant upon the aggressiveness of the media surrounding the steel member, measures against corrosion effects shall be taken into account if substantial losses of steel thickness are to be expected.

(2)P If corrosion is to be taken into account in the design by a reduction of thickness, reference shall be made to 4.4.

(3) Consideration should be given to the following measures to prolong the life of the structure:

- the use of additional steel thickness as a corrosion allowance;
- statical reserve;
- the use of protective coatings (usually paints, grouting or galvanizing);
- the use of cathodic protection, with or without protective coatings;
- providing a concrete, mortar or grout protection in the zone of high corrosion.

(4)P If the required design working life is longer than the duration of the protective effect of a coating, the loss of thickness occurring during the remaining design working life shall be taken into account in serviceability limit state and ultimate limit state verifications.

NOTE 1: A combination of different protective measures might be useful to obtain a high design working life. The whole protective system can be defined taking into account the design of the structure and of the protective coating as well as the feasibility of inspection.

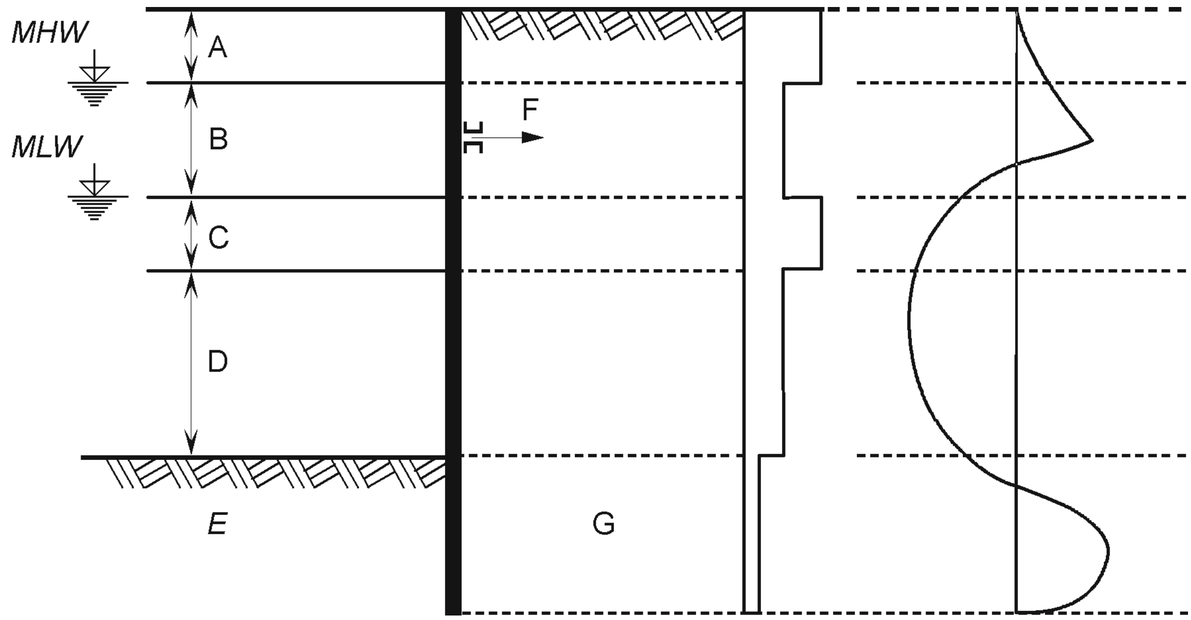
NOTE 2: Special care is necessary in areas where poorly isolated sources of direct current are likely to produce stray currents in the soil.

(5) The possibility that corrosion might not be uniform over the whole length of a pile may be taken into account, allowing an economic design to be achieved by selection of a moment distribution adapted to the corrosion distribution, see Figure 4-1.

(6)P The required design working life for sheet piling and bearing piles shall be given in the project specification.

(7) The loss of thickness due to corrosion may be neglected for a required design working life of less than 4 years, unless a different period is given in the project specification.

(8)P Corrosion protection systems shall be defined in the project specification.



a) Vertical zoning of sea water aggressivity

b) Corrosion rate distribution at side exposed to sea water

c) Typical bending moment distribution

A	Zone of high attack (splash zone);	B	Intertidal zone;
C	Zone of high attack (Low water zone);	D	Permanent immersion zone;
E	Buried zone (Water side);	F	Anchor;
G	Buried zone (Soil side)		
MHW	Mean high water;	MLW	Mean low water

NOTE: Corrosion rate distribution and zones of sea water aggressivity might vary considerably from the example shown in Figure 4-1, dependant upon the conditions prevailing at the location of the structure.

Figure 4-1: Example of corrosion rate distribution

4.2 Durability requirements for bearing piles

(1) Unless otherwise specified, the strength verification of individual piles for both serviceability and ultimate limit state should be carried out taking into account a uniform loss of steel thickness all around the perimeter of the cross-section.

(2) Unless otherwise specified, for serviceability and ultimate limit states the reduction of thickness due to corrosion of piles in contact with water or with soil (with or without groundwater) should be taken from section 4.4, dependant upon the required design working life of the structure.

(3) Unless otherwise specified in the project specification, corrosion inside hollow piles that have watertight closed ends, or are filled with concrete, may be neglected.

4.3 Durability requirements for sheet piling

(1) Unless otherwise specified, in the strength verification of sheet piles for both serviceability and ultimate limit states, the loss of thickness for parts of sheet pile walls in contact with water or with soil (with or without groundwater) should be taken from section 4.4, dependant upon the required design working life of the structure. Where sheet piles are in contact with soil or water on both sides, the corrosion rates apply to each side.

(2) If the aggressiveness of the soil or water is different on opposite sides of a sheet pile wall, two different corrosion rates may be applied.

4.4 Corrosion rates for design

(1) Corrosion rates given in this section should be considered as for design only.

> **NOTE:** Suitable values for corrosion rates may be given in the National Annex, taking into account local conditions. Values that may be used for guidance are given in Table 4-1 and Table 4-2.

(2) The loss of thickness due to atmospheric corrosion may be taken as 0,01 mm per year in normal atmospheres and as 0,02 mm per year in locations where marine conditions may affect the performance of the structure.

> **NOTE:** The following have a major influence on the corrosion rates in soils:
>
> - the type of soil;
> - the variation of the level of the groundwater table;
> - the presence of oxygen.
> - the presence of contaminants.

Table 4-1: Loss of thickness [mm] due to corrosion for piles and sheet piles in soils, with or without groundwater

Required design working life	5 years	25 years	50 years	75 years	100 years
Undisturbed natural soils (sand, silt, clay, schist, ...)	0,00	0,30	0,60	0,90	1,20
Polluted natural soils and industrial sites	0,15	0,75	1,50	2,25	3,00
Aggressive natural soils (swamp, marsh, peat, ...)	0,20	1,00	1,75	2,50	3,25
Non-compacted and non-aggressive fills (clay, schist, sand, silt, ...)	0,18	0,70	1,20	1,70	2,20
Non-compacted and aggressive fills (ashes, slag, ...)	0,50	2,00	3,25	4,50	5,75

Notes:

1) Corrosion rates in compacted fills are lower than those in non-compacted ones. In compacted fills the figures in the table should be divided by two.

2) The values given for 5 and 25 years are based on measurements, whereas the other values are extrapolated.

Table 4-2: Loss of thickness [mm] due to corrosion for piles and sheet piles in fresh water or in sea water

Required design working life	5 years	25 years	50 years	75 years	100 years
Common fresh water (river, ship canal, ...) in the zone of high attack (water line)	0,15	0,55	0,90	1,15	1,40
Very polluted fresh water (sewage, industrial effluent, ...) in the zone of high attack (water line)	0,30	1,30	2,30	3,30	4,30
Sea water in temperate climate in the zone of high attack (low water and splash zones)	0,55	1,90	3,75	5,60	7,50
Sea water in temperate climate in the zone of permanent immersion or in the intertidal zone	0,25	0,90	1,75	2,60	3,50

Notes:

1) The highest corrosion rate is usually found in the splash zone or at the low water level in tidal waters. However, in most cases, the highest bending stresses occur in the permanent immersion zone, see Figure 4-1.

2) The values given for 5 and 25 years are based on measurements, whereas the other values are extrapolated.

5 Ultimate limit states

5.1 Basis

5.1.1 General

(1)P Piles and their components shall be designed such that the basic design requirements for ultimate limit states given in section 2 are satisfied.

(2)P The following provisions shall be applied for the verification of the resistances of cross-sections and members with respect to ultimate limit states.

(3)P Reference shall be made to EN 1990 for the partial factors for actions and the method for combining actions to be applied.

(4) For the partial factors γ_{M0}, γ_{M1} and γ_{M2} to be applied to resistance see EN 1993-1-1.

NOTE: The partial factors γ_{M0}, γ_{M1} and γ_{M2} for piling may be chosen in the National Annex. The following values are recommended: $\gamma_{M0} = 1,00$; $\gamma_{M1} = 1,10$ and $\gamma_{M2} = 1,25$.

5.1.2 Design

(1)P Retaining walls and bearing piles shall be checked for:

- resistance of the cross-section and overall buckling of sheet piling (see 5.2) and of bearing piles (see 5.3);
- the resistance of walings, bracing, connections and anchors (see section 7);
- global failure of the structure as a result of soil failure (see section 2).

5.1.3 Fatigue

(1)P Where a structure or a part of it is sensitive to fatigue phenomena, appropriate criteria shall be defined in the project specification in accordance with EN 1993-1-9, provided a suitable corrosion protection is applied and maintained.

NOTE: In combination with severe corrosion the fatigue strengths may be reduced.

(2) The effects of impact or vibration during installation of bearing piles or sheet piles may be neglected in fatigue analysis.

5.2 Sheet piling

5.2.1 Classification of cross-sections

(1)P If elastic global analysis is used, it shall be verified that the maximum effects of actions do not exceed the corresponding resistances.

(2)P If plastic global analysis is used, it shall be verified that the maximum effects of actions do not exceed the plastic resistance of the steel pile. In addition, the rotation capacity shall be checked, see Table 5-1.

(3) The analysis method for the distribution of effects of actions should be consistent with the following classification of cross-sections:

- Class 1 cross-sections for which a plastic analysis involving moment redistribution may be carried out, provided that they have sufficient rotation capacity;

- Class 2 cross-sections for which elastic global analysis is necessary, but advantage can be taken of the plastic resistance of the cross-section;

- Class 3 cross-sections which should be designed using an elastic global analysis and an elastic distribution of stresses over the cross-section, allowing yielding at the extreme fibres;

- Class 4 cross-sections for which local buckling affects the cross-sectional resistance, see Annex A.

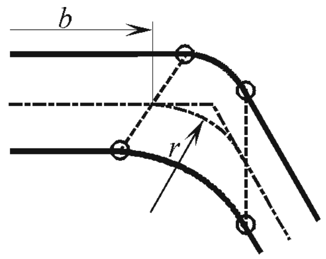
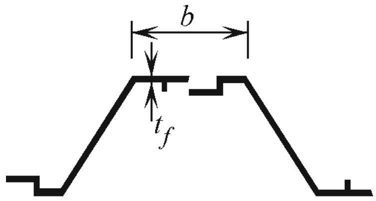
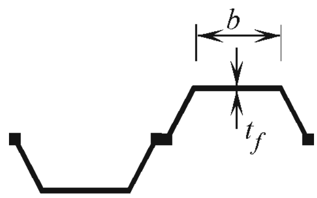
(4) The limiting proportions for class 1, 2 and 3 cross-sections may be obtained from Table 5-1 for steel sheet piles, taking into account a possible reduction of steel thickness due to corrosion.

NOTE: Further guidance on the classification of cross-sections is given in Annex C.

(5) An element which fails to satisfy the limits for class 1, 2 or 3 should be taken as class 4.

(6)P The effects of actions in other structural elements and connections shall not exceed the resistances of those elements and connections.

Table 5-1: Classification of cross-sections

Classification	Z-profile	U-profile
		
Class 1	- the same boundaries as for class 2 apply - a rotation check has to be carried out	
Class 2	$\frac{b/t_f}{\varepsilon} \leq 45$	$\frac{b/t_f}{\varepsilon} \leq 37$
Class 3	$\frac{b/t_f}{\varepsilon} \leq 66$	$\frac{b/t_f}{\varepsilon} \leq 49$

$\varepsilon = \sqrt{\frac{235}{f_y}}$	f_y [N/mm²]	240	270	320	355	390	430
	ε	0,99	0,93	0,86	0,81	0,78	0,74

Key:

b: width of the flat portion of the flange, measured between the corner radii, provided that the ratio r/t_f is not greater than 5,0; otherwise a more precise approach should be used;
t_f: thickness of the flange for flanges with constant thickness;
r: midline radius of the corners between the webs and the flanges;
f_y: yield strength.

Note: For class 1 cross-sections it should be verified that the plastic rotation provided by the cross-section is not less than the plastic rotation required in the actual design case. Guidance for this verification (rotation check) is given in Annex C.

5.2.2 Sheet piling in bending and shear

(1)P In the absence of shear and axial force, the design value of the bending moment M_{Ed} at each cross-section shall satisfy:

$$M_{Ed} \leq M_{c,Rd} \tag{5.1}$$

where:

M_{Ed} is the design bending moment, derived from a calculation according to the relevant case of EN 1997-1;

$M_{c,Rd}$ is the design moment resistance of the cross-section.

(2) The design moment resistance of the cross-section $M_{c,Rd}$ should be determined from the following:

- Class 1 or 2 cross-sections: $M_{c,Rd} = \beta_B \, W_{pl} \, f_y \, / \, \gamma_{M0}$ (5.2)

- Class 3 cross-sections: $M_{c,Rd} = \beta_B \, W_{el} \, f_y \, / \, \gamma_{M0}$ (5.3)

- Class 4 cross-sections: see Annex A.

where:

W_{el} is the elastic section modulus determined for a continuous wall;

W_{pl} is the plastic section modulus determined for a continuous wall;

γ_{M0} partial safety factor according to 5.1.1 (4);

β_B is a factor that takes account of a possible lack of shear force transmission in the interlocks and has the following values:

$\beta_B = 1{,}0$ for Z-piles and triple U-piles

$\beta_B \leq 1{,}0$ for single and double U-piles.

NOTE 1: The degree of shear force transmission in the interlocks of U-piles is strongly influenced by:

- the type of soil into which the piles have been driven;

- the type of element installed;

- the number of support levels and their way of fixation in the plane of the wall;

- the method of installation;

- the treatment of the interlocks to be threaded on site (lubricated or partly fixed by welding, a capping beam etc.);

- the cantilever height of the wall (e.g. if the wall is cantilevered to a substantial distance above the highest waling or below the lowest waling).

NOTE 2: The numerical values for β_B covering these parameters, based on local design experience, may be given in the National Annex.

(3)P The webs of sheet piles shall be verified for shear resistance.

(4) The design value of the shear force V_{Ed} at each cross-section should satisfy:

$$V_{Ed} \leq V_{pl,Rd} \tag{5.4}$$

where:

$V_{pl,Rd}$ is the design plastic shear resistance for each web given by $\dfrac{A_V\, f_y}{\sqrt{3}\,\gamma_{M0}}$; (5.5)

A_v is the projected shear area for each web, acting in the same direction as V_{Ed}.

(5) The projected shear area A_v may be taken as follows for each web of a U-profile or a Z-profile, see Figure 5-1:

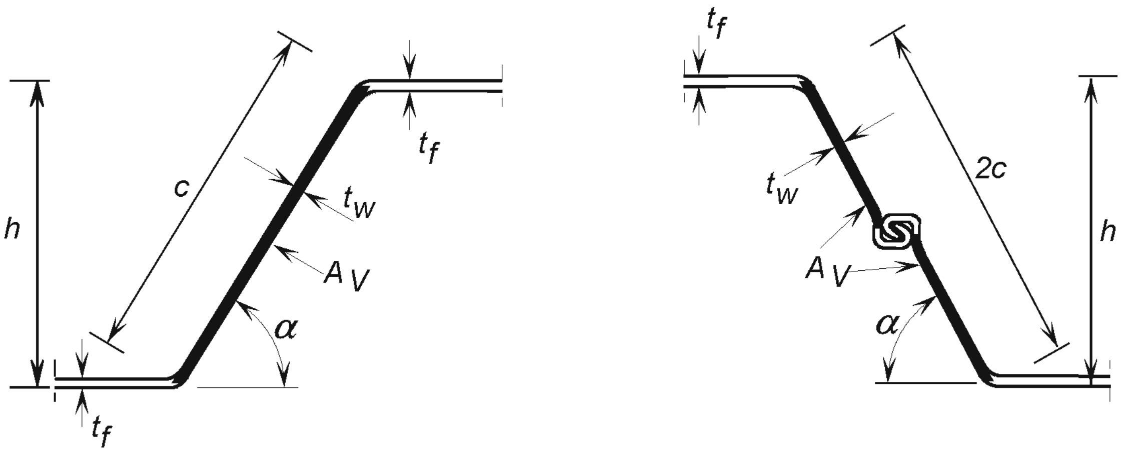
$$A_V = t_W\,(h - t_f) \tag{5.6}$$

where:

h is the overall height;

t_f is the flange thickness;

t_w is the web thickness. In the case of varying web thicknesses $t_{w,i}$ over the slant height c, excluding the interlocks, t_w in expression (5.6) should be taken as the minimum value of $t_{w,i}$.



$c = \dfrac{h - t_f}{\sin\alpha}$ $c = \dfrac{h - t_f}{2\sin\alpha}$

a) Z-pile b) U-pile

Figure 5-1: Definition of the shear area

(6) In addition the shear buckling resistance of the webs of sheet piles should be verified if

$c/t_w > 72\,\varepsilon$.

(7) The shear buckling resistance should be obtained from:

$$V_{b,Rd} = \frac{(h - t_f)\,t_w f_{bv}}{\gamma_{M0}} \tag{5.7}$$

where $f_{b,v}$ is the shear buckling strength according to Table 6-1 of EN 1993-1-3 for a web without stiffening at the support and for a relative web slenderness given by:

$$\bar{\lambda} = 0.346 \frac{c}{t_w} \sqrt{\frac{f_y}{E}} \tag{5.8}$$

(8) Provided that the design value of the shear force V_{Ed} does not exceed 50% of the design plastic shear resistance $V_{pl,Rd}$ no reduction need be made in the design moment resistance $M_{c,Rd}$.

(9) When V_{Ed} exceeds 50% of $V_{pl,Rd}$ the design moment resistance of the cross-section should be reduced to $M_{V,Rd}$, the reduced design plastic moment resistance allowing for the shear force, obtained as follows:

$$M_{V,Rd} = \left[\beta_B W_{pl} - \frac{\rho A_V^{\ 2}}{4 t_W \sin\alpha} \right] \frac{f_y}{\gamma_{M0}} \quad \text{but} \quad M_{V,Rd} \leq M_{c,Rd} \tag{5.9}$$

with:

$$\rho = (2\, V_{Ed} / V_{pl,Rd} - 1)^2 \tag{5.10}$$

where:

A_v is the shear area according to (5.6);

t_w is the web thickness;

α is the inclination of the web according to Figure 5-1;

β_B is the factor defined in 5.2.2(2)P.

NOTE: A_v and t_w are related to the width considered for W_{pl}.

(10)P If steel sheet piling made of U-piles has been connected by welding or by crimping in order to enhance the shear force transmission in these interlocks, the connections shall be verified assuming that the shear force can be transferred only in the connected interlocks.

NOTE: This assumption allows for a safe-sided design of the connections.

(11)P The verification of the butt welds for the transmission of the shear force shall be in accordance with 4.7 of EN 1993-1-8.

(12)P The layout of the butt welds shall be in accordance with 4.3 of EN 1993-1-8 taking into account corrosion if relevant.

(13) In the case of intermittent butt welds, a length of not less than l should be made continuous at each end of the pile in order to avoid possible overstressing during installation. Reference should be made to 1993-1-8 for the design of the welds.

NOTE: The value l may be given in the National Annex. A value of l = 500 mm is recommended.

(14)P It shall be verified that the crimped points of interlocks are able to transmit the resulting interlock shear forces.

(15) Provided that the spacing of the single or double crimped points does not exceed 0,7 m and the spacing of triple crimped points does not exceed 1,3 m, each crimped point may be assumed to transmit an equal shear force $V_{Ed} \leq R_k / \gamma_{M0}$ where R_k is the characteristic resistance of the crimped point determined by testing in accordance with section 2.6.

NOTE: For the determination of R_k by testing see EN 10248.

5.2.3 Sheet piling with bending, shear and axial force

(1) For combined bending and compression, member buckling need not be taken into account if:

$$\frac{N_{Ed}}{N_{cr}} \leq 0{,}04 \tag{5.11}$$

where:

N_{Ed} is the design value of the compression force;

N_{cr} is the elastic critical load of the sheet pile, calculated with an appropriate soil model, taking into account only compression forces in the sheet pile.

(2) Alternatively N_{cr} may be taken as:

$$N_{cr} = EI \beta_D \pi^2 / \ell^2 \tag{5.12}$$

in which ℓ is the buckling length, determined according to Figure 5-2 for a free or partially fixed earth support or according to Figure 5-3 for a fixed earth support and β_D is a reduction factor, see 6.4.

(3)P If the criterion given in (1) is not satisfied, the buckling resistance shall be verified.

NOTE: This verification can be carried out using the procedure given in (4) to (7).

(4) Provided that the boundary conditions are supplied by elements (anchor, earth support, capping beam etc.) that give positional restraint corresponding to the non-sway buckling mode, the following simplified buckling check may be used:

- for class 1, 2 and 3 sections:

$$\frac{N_{Ed}}{\chi N_{pl,Rd} (\gamma_{M0} / \gamma_{M1})} + 1{,}15 \frac{M_{Ed}}{M_{c,Rd} (\gamma_{M0} / \gamma_{M1})} \leq 1{,}0 \tag{5.13}$$

where:

$N_{pl,Rd}$ is the plastic design resistance of the cross-section ($A f_y / \gamma_{M0}$);

$M_{c,Rd}$ is the design moment resistance of the cross-section, see 5.2.2 (2);

γ_{M1} is the partial factor according to 5.1.1 (4);

γ_{M0} is the partial factor according to 5.1.1 (4);

χ is the buckling coefficient from 6.3.1.2 of EN 1993-1-1, using curve d and a non dimensional slenderness given by:

$$\bar{\lambda} = \sqrt{\frac{A\,f_y}{N_{cr}}}$$

with:

N_{cr} is the elastic critical load, which may be determined according to (5.12);

A is the cross-sectional area;

- for class 4-sections: see Annex A.

NOTE: Buckling curve *d* also covers driving imperfections up to 0,5% of ℓ which is considered to be good practice.

(5) For the simplified approach the buckling length ℓ may be determined as follows, assuming a non-sway buckling mode according to (7):

- for a free earth support, provided that sufficient restraint exists according to (6), ℓ may be taken as the distance between the toe and the horizontal support (waling, anchor), see Figure 5-2;

- for a fixed earth support ℓ may be taken as 70 % of the distance between the toe and the horizontal support (waling, anchor), see Figure 5-3.

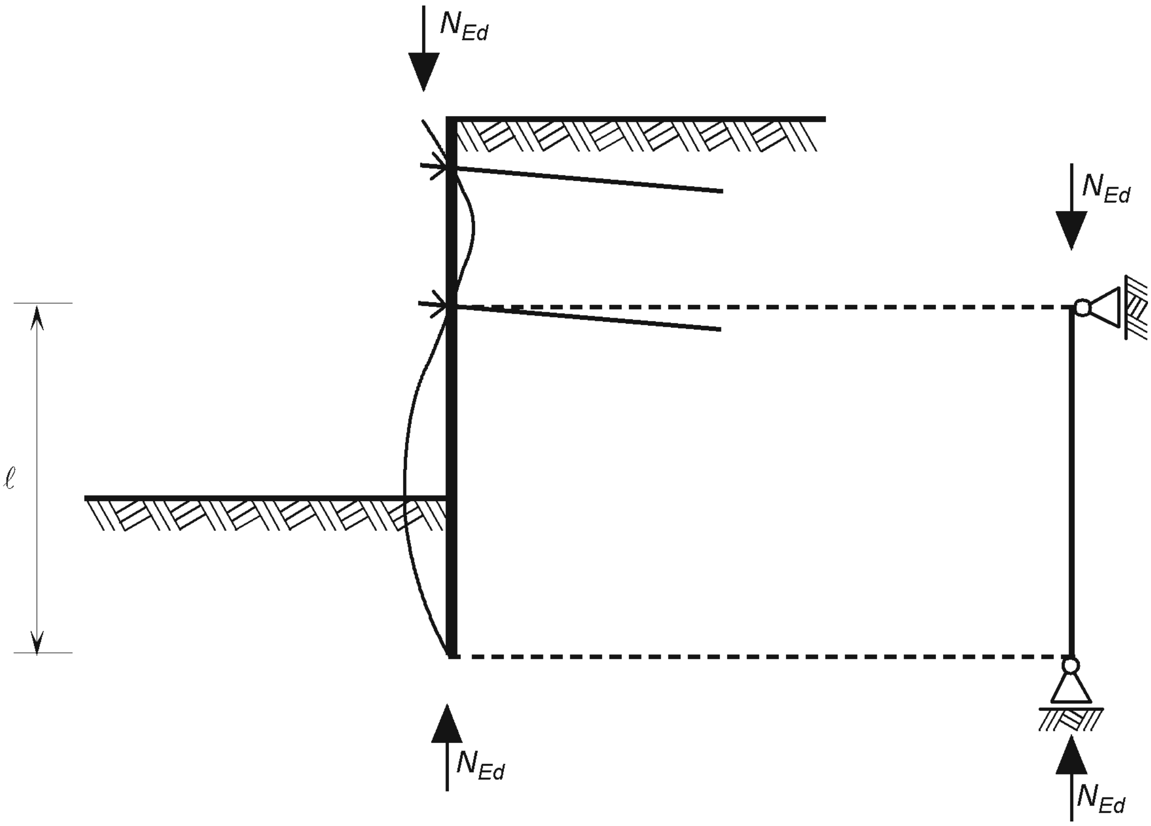
(6) It may be assumed that a free earth support provides sufficient restraint for the simplified approach if the toe of the sheet pile wall is fixed in bedrock or if the toe of the sheet pile wall is able to resist an additional horizontal force $F_{Q,Ed}$ by passive earth pressure or by friction according to Figure 5-4. $F_{Q,Ed}$ is given by:

$$F_{Q,Ed} = \pi N_{Ed}\left(\frac{d}{\ell} + 0{,}01\right) \tag{5.14}$$

where d is the maximum relative deflection of the sheet pile wall occurring between the supports according to a first order analysis. The force $F_{Q,Ed}$ can be resisted by providing an additional pile length Δh according to Figure 5-4 if the soil resistance is fully mobilised in the absence of friction.

(7) If the supplementary displacement of a horizontal support (anchor, waling) due to a support load of $N_{Ed}/100$ is less than $\ell/500$, the support may be assumed to provide enough restraint for the assumption of a non-sway buckling mode.

(8) If the system does not provide enough restraint, a detailed buckling investigation should be carried out, based on the methods given in EN 1993-1-1.



a) deflected shape due to buckling b) simplified system

Figure 5-2: Possible determination of buckling length ℓ, free earth support

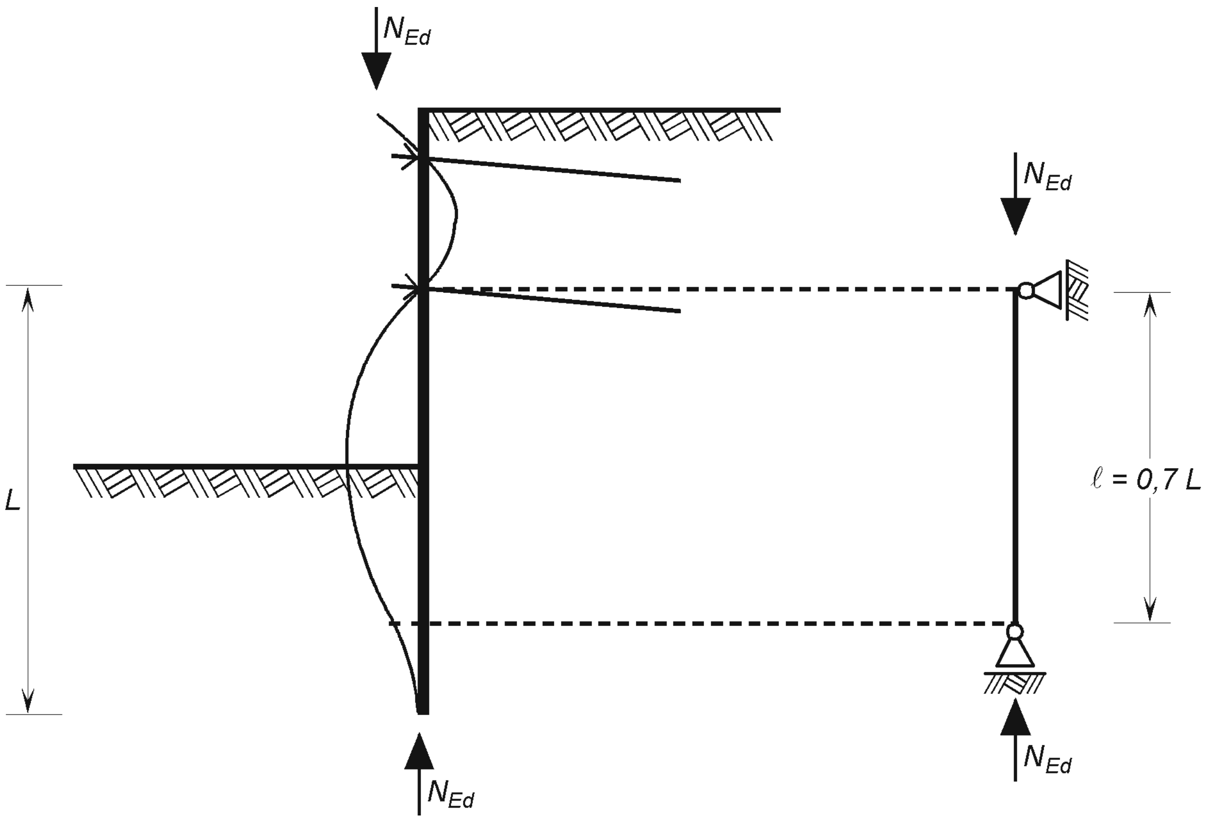
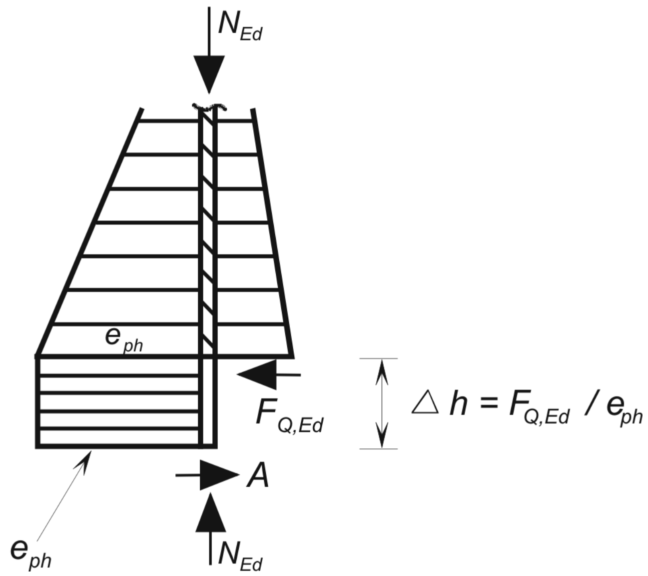


Figure 5-3: Possible determination of buckling length ℓ, fixed earth support



e_{ph} Horizontal passive earth pressure

A Friction force

Figure 5-4: Determination of supplementary horizontal force $F_{Q,Ed}$

(9) For members subject to axial force, the design value of the axial force N_{Ed} at each cross-section should satisfy:

$$N_{Ed} \leq N_{pl,Rd} \tag{5.15}$$

in which $N_{pl,Rd}$ is the plastic design resistance of the cross-section with:

$$N_{pl,Rd} = A f_y / \gamma_{M0} \tag{5.16}$$

(10) The effects of axial force on the plastic moment resistance of the cross-section of class 1, 2 and 3 sheet piles may be neglected if:

- for Z-profiles of class 1 and 2:

$$\frac{N_{Ed}}{N_{pl,Rd}} \leq 0{,}1 \tag{5.17}$$

- for U-profiles of class 1 and 2:

$$\frac{N_{Ed}}{N_{pl,Rd}} \leq 0{,}25 \tag{5.18}$$

- for class 3 profiles:

$$\frac{N_{Ed}}{N_{pl,Rd}} \leq 0{,}1 \tag{5.19}$$

(11) If the axial force exceeds the limiting values given in (10), the following criteria should be satisfied in the absence of shear force:

- Class 1 and 2 cross-sections:
  - for Z-profiles:

$$M_{N,Rd} = 1{,}11\ M_{c,Rd}\ (1 - N_{Ed} / N_{pl,Rd}) \quad \text{but} \quad M_{N,Rd} \leq M_{c,Rd} \tag{5.20}$$

  - for U-profiles:

$$M_{N,Rd} = 1{,}33\ M_{c,Rd}\ (1 - N_{Ed} / N_{pl,Rd}) \quad \text{but} \quad M_{N,Rd} \leq M_{c,Rd} \tag{5.21}$$

- Class 3 cross-sections:

$$M_{N,Rd} = M_{c,Rd}\ (1 - N_{Ed} / N_{pl,Rd}) \tag{5.22}$$

- Class 4 cross-sections: see Annex A.

where:

$M_{N,Rd}$ is the reduced design moment resistance allowing for the axial force.

(12) If the axial force exceeds the limiting value given in (10), account should be taken of the combined presence of bending, axial and shear force as follows:

a) Provided that the design value of the shear force V_{Ed} does not exceed 50% of the design plastic shear resistance $V_{pl,Rd}$ no reduction need be made in combinations of moment and axial force that satisfy the criteria in (11).

b) When V_{Ed} exceeds 50% of $V_{pl,Rd}$ the design resistance of the cross-section to combinations of moment and axial force shall be calculated using a reduced yield strength $f_{y,red} = (1 - \rho) f_y$ for the shear area, where $\rho = (2\, V_{Ed} / V_{pl,Rd} - 1)^2$.

5.2.4 Local effects of water pressure

(1)P In the case of differential water pressure exceeding 5 m head for Z-piles and 20 m head for U-piles the effects of water pressure on transverse local plate bending shall be taken into account to determine the overall bending resistance.

(2) As a simplification, this verification may be carried out for Z-piles using the following procedure:

- if the differential water pressure is more than 5 m head, the cross-sectional verification should be carried out at the locations of the maximum overall bending moments;
- the effect of differential water pressure should be taken into account by using a reduced yield strength

  $$f_{y,red} = \rho_P\, f_y$$

  with ρ_P taken from Table 5-2, for the determination of the cross-sectional resistance;

- to determine ρ_P from Table 5-2 the differential water pressure acting at the relevant locations of maximum moment should be taken into account.

Table 5-2: Reduction factors ρ_P for Z-piles due to differential water pressure

w	$(b/t_{min})\ \varepsilon = 20{,}0$	$(b/t_{min})\ \varepsilon = 30{,}0$	$(b/t_{min})\ \varepsilon = 40{,}0$	$(b/t_{min})\ \varepsilon = 50{,}0$
5,0	1,00	1,00	1,00	1,00
10,0	0,99	0,97	0,95	0,87
15,0	0,98	0,96	0,92	0,76
20,0	0,98	0,94	0,88	0,60

Key:

b is the width of the flange, but b should not be taken as less than $c/\sqrt{2}$, where c is the slant height of the web

t_{min} is the lesser of t_f or t_w

t_f is the flange thickness

t_w is the web thickness

w is the differential head in m

$\varepsilon = \sqrt{\frac{235}{f_y}}$; f_y is the yield strength in N/mm².

Notes:

1) ρ_P = 1,0 may be used if the interlocks of Z-piles are welded.

2) Intermediate values may be interpolated linearily.

5.2.5 Straight web steel sheet piles

(1)P The effects of actions for strength verification of straight web steel sheet piles used in cellular structures, shall be determined from a model that describes the behaviour of the piling and of the fill at ultimate limit states.

(2)P Reference shall be made to EN 1997-1 and to EN 1990 for partial factors to be applied to the fill and the actions.

(3)P The fill model shall be in accordance with EN 1997-1.

(4)P The piling model shall be in accordance with EN 1993-1-1.

NOTE: It can be beneficial to use models taking into account large displacements for the piling.

(5) A two-dimensional analysis in the governing horizontal plane may be used.

(6) The internal pressure resulting from or transmitted through the fill should be determined using a value not less than the at rest value of the earth pressure, see EN 1997-1.

(7) The tensile resistance $F_{ts,Rd}$ of plain straight web steel sheet piles, (other than junction piles) should be taken as the lesser of the interlock resistance and the resistance of the web, using:

$$F_{ts,Rd} = \beta_R R_{k,s} / \gamma_{M0} \quad \text{but} \quad F_{ts,Rd} \leq t_w f_y / \gamma_{M0} \tag{5.23}$$

where:

f_y is the yield strength;

$R_{k,s}$ is the characteristic interlock resistance;

t_w is the web thickness;

β_R is the reduction factor for interlock resistance.

NOTE: The value β_R may be given in the National Annex. The value β_R = 0,8 is recommended.

(8) The characteristic resistance of the interlock $R_{k,s}$ depends upon the cross-section of the interlock and the steel grade adopted. The characteristic interlock resistance $R_{k,s}$ should be determined by testing according to section 2.6 and EN 10248.

(9) Plain piles should be verified such that:

$$F_{t,Ed} \leq F_{ts,Rd} \tag{5.24}$$

where:

$F_{ts,Rd}$ is the design tensile resistance according to expression (5.23);

$F_{t,Ed}$ is the design value of the circumferential tensile force.

(10)P When piles of different sizes are used in the same segment of a wall, the lowest tensile resistance shall be used for the verification.

(11)P The deviation angle (180° minus the angle between two adjacent faces) shall be limited to the maximum value given by the manufacturer.

(12)P For welded junction piles, steel grades with appropriate properties shall be used.

(13)P The design of junction piles according to Figure 5-5 and Figure 5-6 shall take account of the stresses due to plate bending.

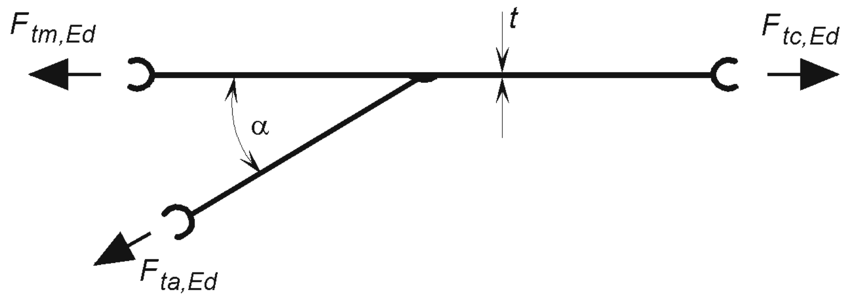


Figure 5-5: Welded junction pile

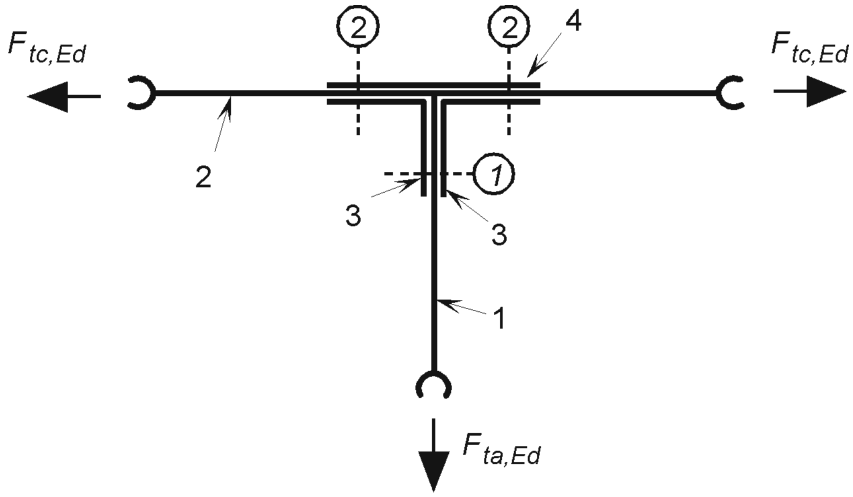


Figure 5-6: Bolted T-connection with backing plate

(14) Provided that welding is carried out according to the procedure given in EN 12063 the welded junction pile may be verified using:

$$F_{tc,Ed} \leq \beta_T \; F_{ts,Rd} \tag{5.25}$$

where:

$F_{ts,Rd}$ is the design tensile resistance of the pile according to expression (5.23);

$F_{tc,Ed}$ is the design tensile force in the main cell given by:

$$F_{tc,Ed} = p_{m,Ed} \; r_m \tag{5.26}$$

with:

$p_{m,Ed}$ is the design value of the internal pressure of the main cell in the governing horizontal plane due to water pressure and the at rest pressure of the fill;

r_m is the radius of the main cell, see Figure 5-7.

β_T is a reduction factor that takes into account the behaviour of the welded junction pile at ultimate limit states and should be calculated as follows:

$$\beta_T = 0{,}9\,(1{,}3 - 0{,}8\, r_a / r_m)\,(1 - 0{,}3 \tan \varphi_d) \tag{5.27}$$

in which r_a and r_m are the radii of the connecting arc and of the main cell according to Figure 5-7 and φ_d is the design value of the internal friction angle of the fill material.

NOTE 1: The factor β_T takes into account the rotation capacity (ductility) of the junction pile as well as the rotation demand (up to 20°) according to a model covering the behaviour of the cofferdam at ultimate limit states.

NOTE 2: Expression (5.27) although developed for cellular cofferdams with aligned connecting arcs, see Figure 5-7, yields acceptable results for alternative configurations. Where more appropriate values are required, these values can be determined either by comparable experience or by testing in combination with a suitable design model in accordance with (1)P.

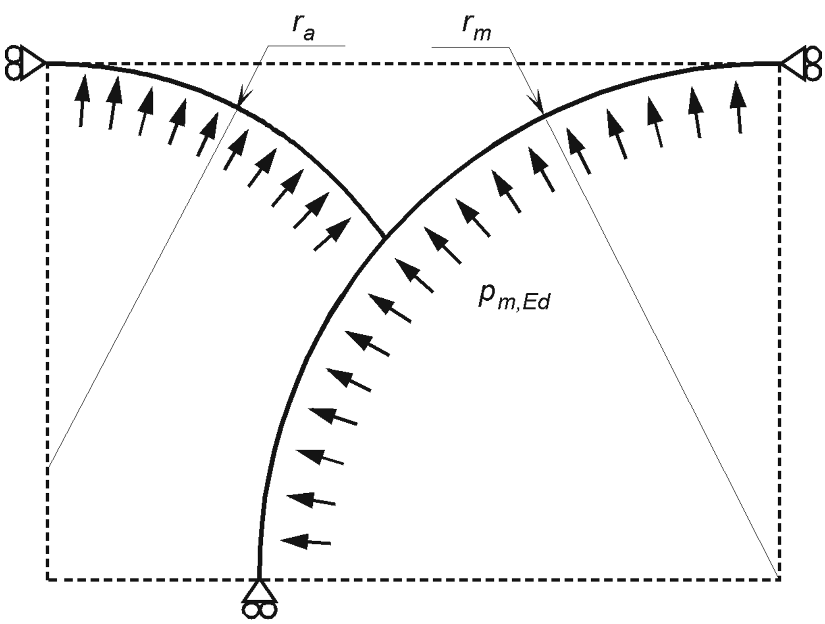


Figure 5-7: Geometry of circular cell and the aligned connecting arc

(15) For a 90° junction pile a bolted T-connection may be used.

(16) For junction piles built up as a bolted T-connection shown in Figure 5-6, the verification may be carried out using the following procedure.

(17) The interlock strength should be verified according to (9).

(18) The connections should be verified as follows, see Figure 5-6:

- verification of the shear and bearing resistance of the bolts (1) according to 3.6 of EN 1993-1-8, assuming the tensile force $F_{ta,Ed}$ is equally distributed;

- verification of the bolt spacing (1) according to 3.5 of EN 1993-1-8;

- verification of the net cross-section of the web 1 and of the adjacent legs of the angles 3 according to the provisions given in 6.2.5 of EN 1993-1-8;

- verification of the bolts (2) according to 3.11 of EN 1993-1-8 for their tensile resistance using a T-stub model according to 6.2.4 (mode 3) of EN 1993-1-8;

- verification of the backing plate 4 and of the adjacent legs of the angles 3 according to the provisions given in 6.2.4 (mode 1 and mode 2) of EN 1993-1-8. In order to permit the use of the design failure modes given in 6.2.4 of EN 1993-1-8, the web of the pile 2 (see Figure 5-6) should be taken as the flange of the equivalent T-stub for modes 1 and 2;

- verification of the web of the pile 2 for the tensile force $F_{tc,Ed}$ against yielding of the net cross-section.

(19)P Other types of junction piles may be verified accordingly.

5.3 Bearing piles

5.3.1 General

(1)P The effects of actions in piles shall be determined in accordance with EN 1997-1, taking account of both equilibrium and compatibility.

(2)P Ultimate limit state verifications shall be carried out for failure in the soil for both individual piles and pile groups according to EN 1997, and for failure of the piles and their connections to the structure according to EN 1993-5, EN1992 and EN1994.

5.3.2 Design methods and design considerations

(1)P For piles subjected to axial and transverse loading, the soil resistance shall be taken from EN 1997-1.

(2) The effects of actions in the pile due to transverse forces should be taken into account in combination with those due to axial forces and applied moments. They may be determined by superimposing the results of separate calculations in which the soil in contact with separate portions of the pile length is assumed to be resisting different actions. Alternatively the axial force, bending moments and transverse forces may be considered as resisted by soil over the same length of pile, provided that the soil is capable of resisting their combined effects.

(3)P The structural design of an individual pile shall be verified in accordance with section 5 of EN 1993-1-1.

(4) For axial forces acting at the head of the pile, the distribution of stress may be conservatively taken as constant over the length of the pile for the determination of the effects of actions, except in the case of negative skin friction.

(5) The transmission of torsional moments acting at the head of the pile should not be assumed unless special provisions allow the introduction of the torque into the soil. The distribution of the torque should be taken as constant over the pile length.

5.3.3 Steel piles

(1)P Cross sectional verification of steel bearing piles shall be in accordance with EN 1993-1-1.

(2) Reference to section 7.8 of EN 1997 may be made for indications of the soil conditions under which overall buckling of piles has to be taken into account.

(3) If the soil provides insufficient lateral restraint, the slenderness criterion for overall buckling may be assumed to be satisfied if $N_{Ed} / N_{cr} \leq 0{,}10$, where N_{cr} is the critical value of the axial force N_{Ed}.

(4)P If overall buckling verification is required reference shall be made to section 5 of EN 1993-1-1. The following effects should be taken into account:

- in addition to the imperfections given in 5.3 of EN 1993-1-1 due consideration should be given to supplementary initial imperfections (e.g. due to joints or installation) in accordance with EN 12699 and EN 14199;

- lateral support due to the surrounding soils may be taken into account using an appropriate model (e.g. p-y approach, subgrade reaction model) based on second order theory.

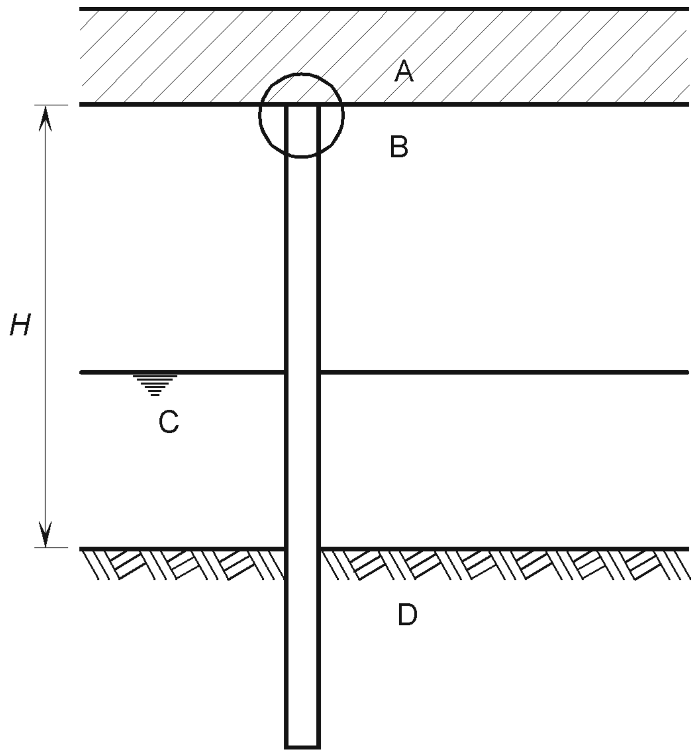
(5) The buckling length may be estimated using the following approach (see Figure 5-8):

$$l_{cr} = k\,H \tag{5.28}$$

The value k takes into account the connection between the pile head and the concrete deck or the steel structure.

(6) For a more precise determination of the buckling length, for instance for micro piles, reference should be made to 5.3.3 (4)P.

(7)P Execution shall be in accordance with EN 12699 and EN 14199.



A	concrete or steel structure	B	connection
C	water or soft soil	D	firm soil

$$l_{crit} = k\,H \qquad \text{with} \quad k = \begin{cases} 1{,}0 & \text{connection B fixed and pinned} \\ 0{,}7 & \text{connection B fixed and restrained} \\ 2{,}0 & \text{connection B non fixed, but restrained} \end{cases}$$

Figure 5-8: Simplified estimation of buckling length for bearing piles

5.3.4 Steel piles filled with concrete

(1)P Steel piles filled with concrete shall be designed in accordance with EN 1994.

(2) Cross sectional verifications of steel piles filled with concrete shall be in accordance with EN1994-1.

(3) Reference shall be made to 5.3.3 and section 6.7 of EN 1994-1-1 for overall buckling verification.

(4)P Concreting of a steel pile shall be carried out in accordance with EN 1536, EN 12699 and EN 14199.

5.4 High modulus walls

(1)P The design of high modulus walls shall be carried out according to the provision for sheet pile walls, taking into account the specific geometry of the sections used, see Figure 1-6, allowing for local effects due to earth and water pressures and the introduction of anchor and waling forces.

(2) The determination of cross-section resistance may be conservatively based on an elastic analysis of the cross-section, provided that:

- buckling of plate elements is checked using EN 1993-1-5;
- the shear lag effect is taken into account for wide elements.

5.5 Combined walls

5.5.1 General

(1) In the following provisions for the ultimate limit state are given for the following types of combined walls, see Figure 1-5:

- mixed tube and sheet pile walls;
- mixed special I-section and sheet pile walls;
- mixed built-up section and sheet pile walls.

(2) The design of the primary and secondary elements should be based on their functionality:

- the primary elements act as retaining elements against the earth and water pressures and may act as bearing piles for vertical loads;
- the secondary elements only fill the gap between the primary elements and transmit the loads resulting from earth and water pressures to the primary elements.

(3)P No transmission of longitudinal shear forces may be taken into account in the free interlocks between primary and secondary elements.

(4)P It shall be stated in the project specification whether driving imperfections need to be considered in the design of the combined wall. The design values of any driving imperfections shall be given as percentages of the length of the primary elements, assuming a linear distribution.

5.5.2 Secondary elements

(1)P Sheet piles used as secondary elements for combined walls shall be in accordance with EN 10248.

(2)P For the design of secondary elements, it shall be verified that they are able to transmit the internal forces resulting from earth and water pressures into the primary elements via the connecting devices.

NOTE: It can be advantageous to take into account arching effects leading to a supplementary loading on the primary elements and a reduction of the earth pressures acting on the secondary elements.

(3) The verification according to (2)P may be carried out using a simplified two dimensional frame model for the secondary elements. If required in the project specification, driving imperfections should be taken into account in this simplified analysis via the imposed displacement δ using the boundary conditions given in Figure 5-9, which shows a double U-pile as an example of a secondary element.

NOTE: The driving imperfection perpendicular to the plane of the retaining wall is assumed to be absorbed by rotation at the interlocks ("interlock swing").

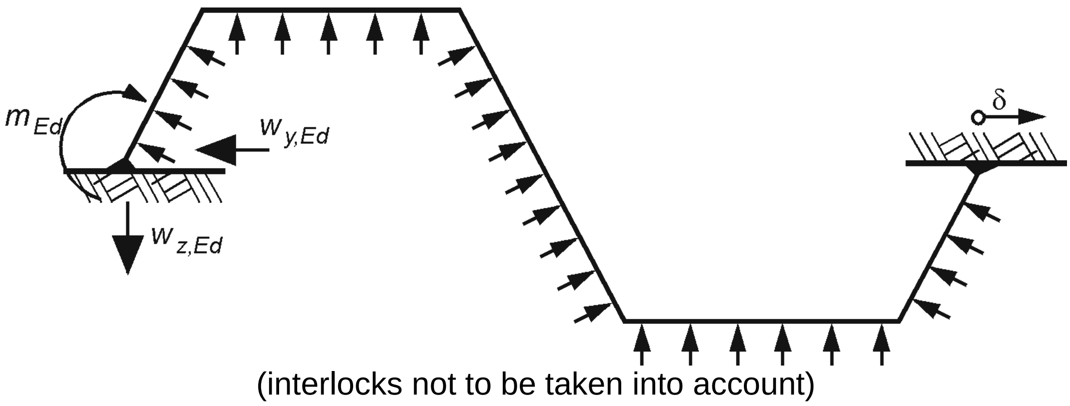


Figure 5-9: Simplified model for secondary elements

(4) For the verification of the cross-section in the simplified frame model, a plastic analysis combined with large displacements may be used. If members of the frame model are stressed in compression, particular attention should be paid to the possibility of instability, such as "snap-through".

(5) Alternatively the verification according to (2)P may be based on the results of testing in accordance with section 2.6.

NOTE: For test evaluation see Annex D of EN 1990.

(6)P The test set-up shall be able to simulate the behaviour of the intermediate elements.

(7) For sheet piles used as secondary elements, further verification may be omitted if all the following conditions are met:

- wall thickness of the sheet piles: ≥10mm;
- pressure difference acting on the sheet piles: ≤40kN/m^2, corresponding to 4 m differential water head;

- maximum clearance between the primary elements is 1,8m for U-piles and 1,5m for Z-piles.

(8) The secondary elements may be shorter than the primary elements, because they are not required beyond the point of zero pressure. Dependant upon the type of soil, an extra length of 1,5 to 2,5m should be added to secondary elements beyond this point in accordance with EN 1997-1.

NOTE: For shortened secondary elements, care should be taken to avoid underflow in the case of high differential water pressure, or where there is a danger of scour.

(9) Passive earth pressures may be assumed to act on a continuous retaining wall due to spatial earth pressure distribution even if the secondary elements have been shortened. Reference should be made to EN 1997-1.

5.5.3 Connecting devices

(1)P The connections between the primary and secondary elements shall be designed to allow the transmission of the design forces from the secondary elements into the primary elements.

(2) This verification may be based on the results of testing in accordance with section 2.6.

(3)P If the verification is carried out by calculation it shall be verified that the connections are able to transfer the support reactions determined according to 5.5.2(3).

(4) Plasticity should be taken into account for the verification of the connecting devices in plate bending.

5.5.4 Primary elements

(1)P The overall effects of actions due to earth and water pressures shall be determined taking into account the loading on both primary and secondary elements and possible supplementary loading due to arching effects in the ground, see 5.5.2(2)P.

(2)P Account shall be taken of the reduction of the overall resistance of the primary elements due to the forces introduced by the secondary elements via the connecting devices. This requirement may be deemed to be satisfied, if the earth pressure is supposed to act on the primary elements directly, due to the arching effect and if the differential water pressure acting on the secondary elements is $\leq h$ m head.

NOTE: The value h may be given in the National Annex. A value of $h = 5$ m is recommended.

(3) For strength verification of primary elements, unless a more advanced method is used, the design forces from secondary elements introduced via connections, should be taken into account using support reactions determined according to 5.5.2 (3).

(4) The overall resistance may be determined either by testing in accordance with section 2.6 or by calculation as given below.

(5)P The verification of I-section or tubular piles shall be in accordance with section 5 of EN 1993-1-1.

(6) The effects on the resistance of I-section piles due to the introduction of forces from secondary elements via connections should be taken into account in accordance with EN 1993-1-1.

> **NOTE:** The procedure given in Annex D.1 may be used to determine the reduced overall resistance of I-section piles used as primary elements in combined walls due to the application of the design forces from the secondary elements.

(7) The effects on the resistance of tubular piles due to the introduction of forces from secondary elements via connections should be taken into account in accordance with EN 1993-1-1 and EN 1993-1-6.

> **NOTE:** The procedure given in Annex D.2 may be used to determine the reduced overall resistance for tubular piles used as primary elements in combined walls due to the application of the design loads from the secondary elements.

(8)P For the application of concentrated loads via walings, anchors etc. the tubular pile shall either be verified accordingly or be provided with stiffeners or be filled with concrete or with high grade compacted, non-cohesive material in order to avoid local buckling.

(9) In the case of a tubular pile that is filled according to (8)P, the full cross-sectional resistance in accordance with EN 1992, EN 1993 and EN 1994 may be used in the filled part of the tube.

(10)P Built-up sections used as primary elements shall be verified according to 5.4, provided that due consideration is given to the effect of load application resulting from the secondary elements.

(11) If the simplified approach of 5.4(2) is used, the local effects due to the application of the support reactions determined according to 5.5.2(3) should be taken into account.

6 Serviceability limit states

6.1 Basis

(1)P The significance of settlements and vibrations, and their limiting values in each case, shall be given in the project specification.

(2)P The limiting values shall be confirmed by a serviceability limit state verification.

(3) Even if no limiting values are given, it should be verified that plastic deformations do not occur, using a model in accordance with 2.5.3.3 (1)P.

(4)P The design of sheet piles or bearing piles shall be checked at serviceability limit states using appropriate design situations as specified in EN 1997-1, taking into account a possible reduction of steel thickness due to corrosion.

6.2 Displacements of retaining walls

(1)P EN 1997-1 shall be taken into account when assessing the displacements of retaining walls.

(2) Displacements due to the movement of supports (such as walings, bracing, anchorages) should be taken into account.

(3) If necessary, initial imperfections due to driving should be taken into account in addition to the deformations due to loading based on the driving tolerances indicated in EN 12063.

NOTE: This might be necessary if a particular clearance is required in a cofferdam.

(4) When assessing the displacements of a sheet pile wall account should be taken of the fact that the quality of the workmanship and supervision during execution has an important influence on the magnitude of those displacements.

6.3 Displacements of bearing piles

(1)P EN 1997-1 shall be taken into account when determining the displacements of bearing piles and micro piles.

6.4 Structural aspects of steel sheet piling

(1)P When calculating the displacements of the retaining structure, the possible supplementary displacements due to local deformation at the location of anchors, walings and bracing shall be taken into account where their effect is significant.

NOTE: These effects might be relevant if high local transverse forces are introduced into unstiffened jagged walls, see Figure 1-7, through an H-beam used as waling.

(2)P The effective flexural stiffness shall be taken into account.

(3) The effective flexural stiffness of sheet piling made of U-piles may be determined as follows, taking into account the degree of shear force transmission in interlocks that are located close to the centroidal axis of the wall:

$$(EI)_{eff} = \beta_D \ (EI) \tag{6.1}$$

where:

I is the second moment of area of the continuous wall;

β_D is a factor with a value ≤ 1,0, accounting for the possible reduction due to insufficient shear force transmission in the interlocks.

NOTE 1: β_D depends on many local influences as given in note 1 to 5.2.2(2). The numerical value for β_D may be given in the National Annex.

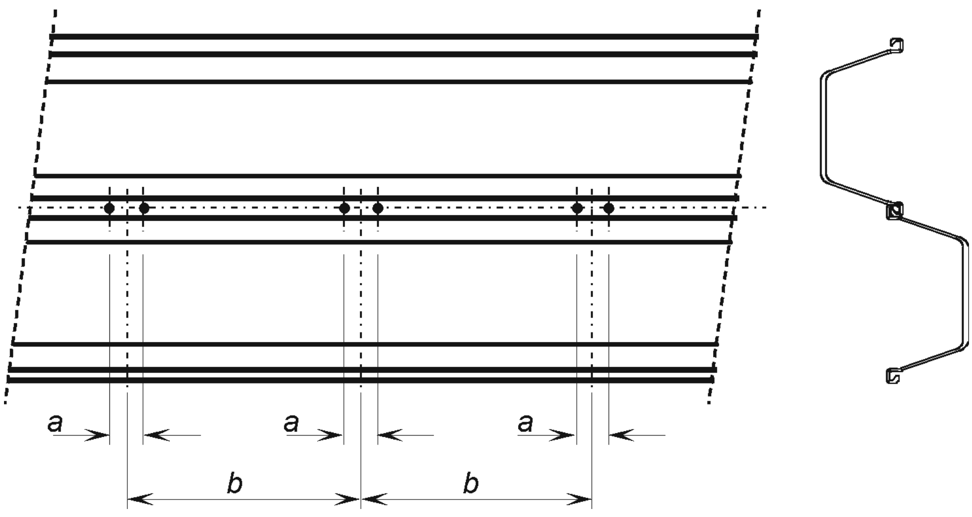
NOTE 2: The transmission of shear forces in the interlocks of U-piles may be enhanced by connecting the interlocks by continuous or intermittent welding or by crimping.

(4)P Crimped points shall be able to transmit the required interlock shear force. The representative shear force R_{ser} transmitted by a crimped point at serviceability limit state is: R_{ser} = 75 kN.
It shall be verified by testing, in accordance with EN 10248, that the stiffness of the crimped point is not less than 15 kN/mm.

NOTE 1: This stiffness requirement corresponds to a shear force of 75 kN at a displacement of 5 mm.

NOTE 2: Crimped points may be single, double or triple crimped points.

(5) Provided that the spacing of the single or double crimped points does not exceed 0,7 m (see Figure 6-1) and the spacing of triple crimped point does not exceed 1,30 m, each crimped point may be assumed to transmit an equal shear force $V_{ser} \leq R_{ser}$.



a ≤ 100 mm b ≤ 700 mm

Figure 6-1: Spacing of double crimped points

7 Anchors, walings, bracing and connections

7.1 General

(1)P The effects of actions in anchors, walings, bracing and connections shall be determined from the structural analysis taking into account the interaction between the soil and the structure.

(2)P Where necessary, effects of actions such as those due to temperature changes or to specific loads shall be taken into account, see 2.5.2(4).

(3) Appropriate simplified methods of analysis may be used in which the actions applied to the various elements of the structure take account of the behaviour of individual members.

(4) For partial factor γ_{Mb} and $\gamma_{Mt,ser}$ to be applied to connections see EN 1993-1-8.

NOTE: The partial factors γ_{Mb} and $\gamma_{Mt,ser}$ may be defined in the National Annex. The values γ_{Mb} = 1,25 and $\gamma_{Mt,ser}$ = 1,10 are recommended.

7.2 Anchorages

7.2.1 General

(1)P The verification of the cross-sections and the connections between the steel parts of dead-man anchors, including tie rods, anchor heads or couplers, shall be carried out according to the following.

NOTE: Design provisions for the steel parts of prestressed anchors are given in EN 1537.

(2)P The testing procedure and the use of test results for determining the design resistance of dead-man anchors and grouted anchors in respect of pull-out failure of the anchor (soil-structure behaviour), shall be in accordance with the principles laid down in EN 1997-1 and EN 1537.

7.2.2 Basic design provisions

(1)P For anchor design, consideration shall be given to both serviceability and ultimate limit states.

(2)P The anchor length shall be such as to prevent failure of the soil or bond failure before yielding of the minimum required cross-section of the anchor. The anchorage length shall be calculated in accordance with EN 1997-1.

(3) For dead-man anchors steel with a specified yield strength not greater than 800 N/mm^2 should be used.

(4) The axial stiffness of the anchor should be taken into account in the design of the retaining wall. It may be assessed by preliminary testing or from comparable experience.

NOTE: It might be useful to “bracket” the effect of the anchor stiffness on the design of the retaining wall by using a maximum/minimum approach for the stiffness.

7.2.3 Ultimate limit state verification

(1)P The tensile resistance $F_{t,Rd}$ of anchors shall be taken as the lesser of $F_{tt,Rd}$ and $F_{tg,Rd}$.

(2) Unless otherwise specified, the tensile resistance $F_{tt,Rd}$ of threads of anchors should be taken as:

$$F_{tt,Rd} = k_t \frac{f_{ua} A_S}{\gamma_{Mb}} \tag{7.1}$$

Where:

A_s is the tensile stress area at the threads;

f_{ua} is the tensile strength of the steel anchor;

γ_{Mb} is the partial factor according to 7.1 (4).

NOTE 1: k_t may be given in the National Annex, k_t = 0,9 is a recommended value.

NOTE 2: Conservatively, the net area of the threaded portion can be used instead of the tensile stress area.

(3) The tensile resistance $F_{tg,Rd}$ of the shaft of an anchor should be taken as

$$F_{tg,Rd} = A_g f_y / \gamma_{M0} \tag{7.2}$$

where:

A_g is the gross cross-sectional area of the anchor rod.

(4) The design provisions given in (2) and (3) do not cover the occurrence of bending in the thread. Detailing of the connection providing enough rotation tolerance and, if relevant, the installation procedure for the tie rods can avoid the occurrence of bending in the threads.

(5) If the anchors are provided with a dead-man end, or with other load distributing members at their end, no account should be taken of the contribution of bond along the anchor shaft. The whole of the force should be transferred through the load distributing device.

(6)P The design tensile resistance of the washer plate assembly $B_{t,Rd}$ shall be taken as the lesser of the design tension resistance $F_{tg,Rd}$ given in (3) and the design punching shear resistance of the anchor head and the nut $B_{p,Rd}$, from Table 3-4 of EN 1993-1-8.

(7)P The design of steel load-distributing members shall be in accordance with EN 1993-1-1.

(8) In the case of an inclined anchor, it should be demonstrated that the component of the anchor force acting in the direction of the longitudinal axis of the sheet pile can be safely transferred from the anchor to the walings or the flange of the sheet pile and into the ground, see EN 1997-1.

7.2.4 Serviceability limit state verification

(1)P For serviceability limit state verifications, the cross-section of the anchor shall be designed to prevent deformations due to yielding of the tie rod under the characteristic load combination.

(2) The principle (1)P may be deemed to be satisfied provided that

$$F_{t,ser} \leq \frac{f_y\, A_S}{\gamma_{Mt,ser}} \tag{7.3}$$

where:

A_s	is	the tensile stress area of the threaded portion or the gross cross-sectional area of the shaft, whichever is smaller;
$F_{t,ser}$	is	the axial force of the anchor under characteristic loading;
$\gamma_{Mt,ser}$	is	the partial factor according to 7.1 (4).

7.2.5 Durability requirements

(1)P Reference shall be made to EN 1537 for the durability requirements of anchors made from high strength steel as defined in 3.7 (1)P.

(2)P Reference shall be made to 4.1 for anchors made from other steel grades.

NOTE: The occurrence of bending of the anchor rod at the connection with the sheet pile wall might have a detrimental effect on the durability of the retaining structure. Due consideration needs to be given to this, especially for retaining walls whose stability is reliant solely on anchors.

7.3 Walings and bracing

(1)P The structural properties of walings and bracing used in structural analysis shall be in accordance with the design details.

(2)P For the verification of ultimate limit states, the effects of actions on the walings and bracing shall be determined for all relevant design situations.

NOTE: If a strut fails there is unlikely to be any warning such as gradual movement, or any time to take remedial measures. Failure of an anchor might lead to progressive failure. As the consequences of these members failing can be very serious, a conservative approach to the design of such members and their connections might be appropriate.

(3)P The cross-sectional resistance of the members shall be in accordance with EN 1993-1-1.

7.4 Connections

7.4.1 General

(1)P The resistance of connections shall be verified according to EN 1993-1-8.

7.4.2 Bearing piles

(1) Unless otherwise specified, the connection between the bearing pile and the pile cap may be taken into account in different (conservative) ways for the design of the steel pile and for the design of the pile cap.

> **NOTE:** The degree of fixity at the connection between a pile and the pile cap or foundation will dictate the local shear forces and moments that have to be designed for.

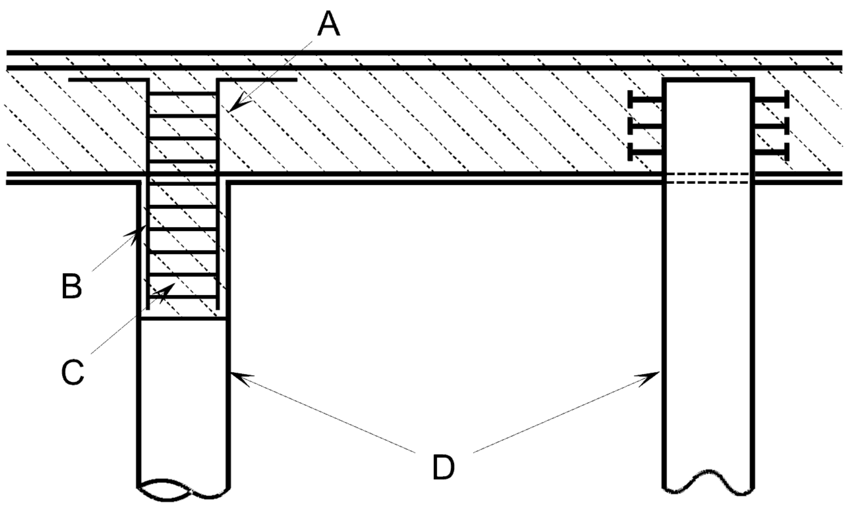
(2) The structural properties of connections (pinned or fixed connections) between the heads of the piles and the pile cap, which depend on their rigidity and design detailing, should be chosen in accordance with the selected method of load transfer, examples of which are provided in Figure 7-1 and Figure 7-2, see also EN 1994.

> **NOTE:** Direct connection of a steel structure to a bearing pile is also possible as illustrated in Figure 7-3.

(3)P Durability aspects shall be taken into account in the design of connections between pile and pile cap.

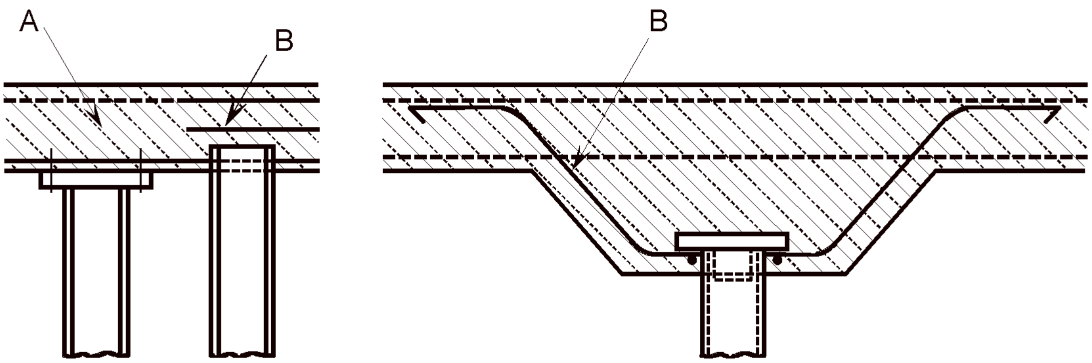
(4) Joints between two pile elements should be designed in accordance with EN 1993-1-8.

> **NOTE:** The National Annex may give information on the design procedure for pile couplers.



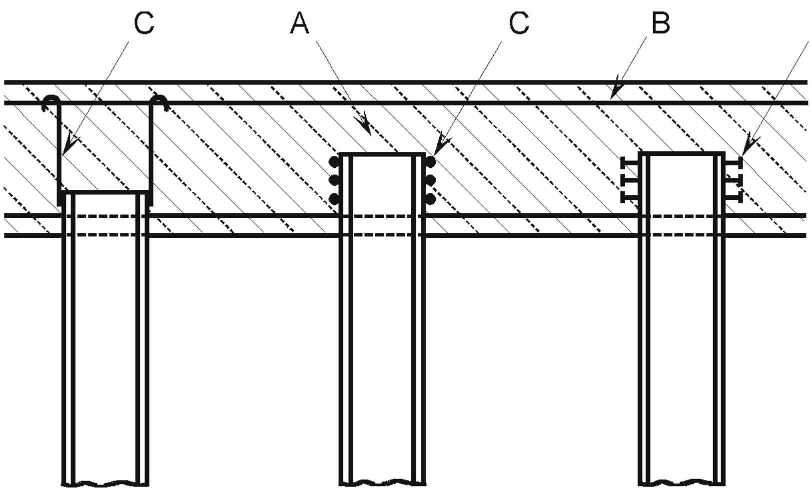
A Concrete slab / pile cap; B Reinforcement;
C Reinforced concrete infill; D Steel pile

Figure 7-1: Tubular and box type piles, examples of connections with the pile cap



A Pile cap;
B reinforcement designed to take into account the method of load transfer to the concrete slab

a) compressive loading



A Pile cap;
B reinforcement designed to take into account the method of load transfer to the concrete slab
C Rebar welded to piles
D Shear studs or welded on angle

b) compressive and tensile loading

Figure 7-2: Examples of bearing pile connections with a concrete pile cap

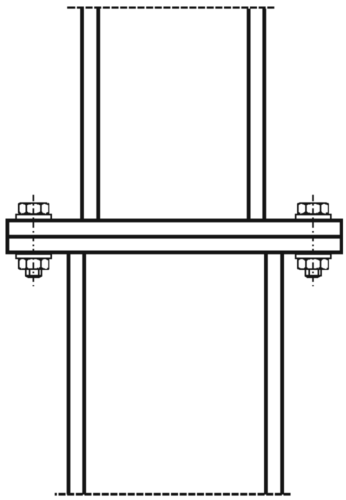


Figure 7-3: Example of a bearing pile connection to a column of a steel structure above the foundation

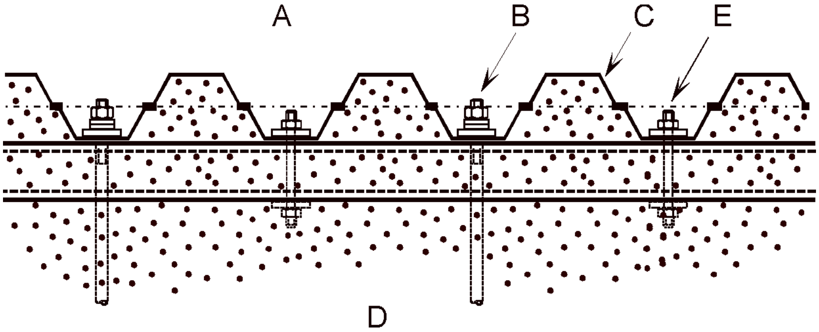
7.4.3 Anchoring

(1)P The resistance of the sheet pile to the introduction of the anchor force into its flange via a washer plate with a waling behind the wall (see Figure 7-4), or without using a waling (see Figure 7-5a), shall be verified.

NOTE: A possible procedure for this verification is given in (3).

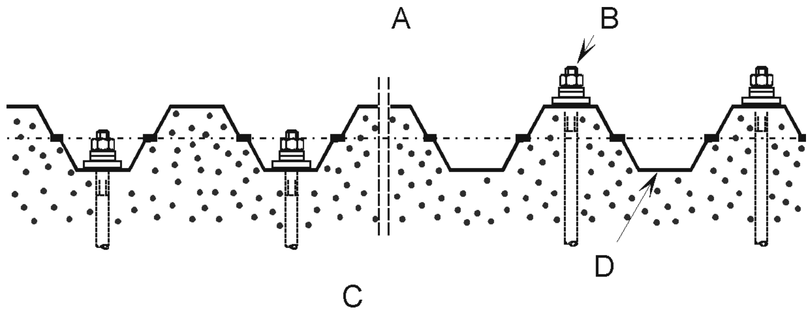
(2)P The resistance of the sheet pile to the introduction of the anchor force or strut force into the webs via a waling (see Figure 7-6) or via a washer plate (see Figure 7-5b) shall be verified.

NOTE: Possible procedures for these verifications are given in (4) and (5).



A Excavation; B Anchor;
C Sheet pile wall; D Soil;
E Tie bolt;

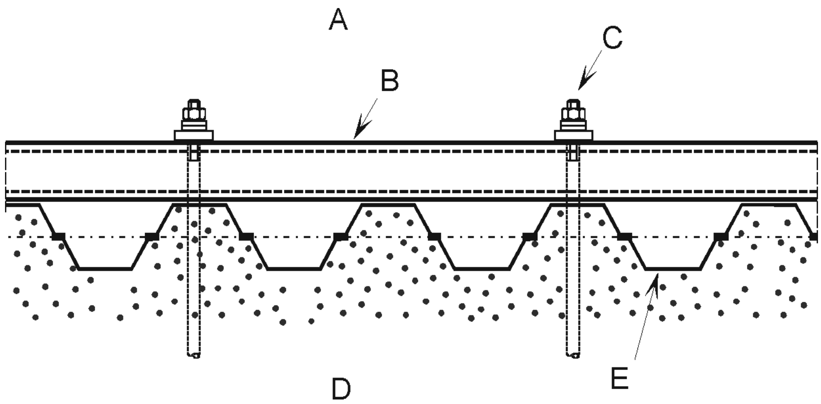
Figure 7-4: Example of anchoring with a waling behind the sheet pile wall



A Excavation; B Anchor;
C Soil; D Sheet pile wall

a) anchor located in an in-pan of the sheet pile wall b) anchor located on an out-pan of the sheet pile wall

Figure 7-5: Examples of anchoring without a waling



A Excavation; B Waling;
C Anchor; D Soil;
E Sheet pile wall

Figure 7-6: Example of a waling in front of the sheet pile wall

(3) The resistance of the sheet pile to that part of the anchor force to be introduced into the flange via a washer plate with a waling behind the wall (see Figure 7-4) or without using a waling (see Figure 7-5a) may be verified in accordance with the following:

a) Shear resistance of flange:

$$F_{Ed} \leq R_{Vf,Rd} \tag{7.4}$$

where:

F_{Ed} is the design value of the local transverse force applied through the flange;

$R_{Vf,Rd}$ is the design value of the shear resistance of the flange under the washer plate, given as

$$R_{Vf,Rd} = 2{,}0\ (b_a + h_a)\ t_f\ \frac{f_y}{\sqrt{3}\,\gamma_{M0}} \tag{7.5}$$

with:

b_a is the width of the washer plate;

f_y is the yield strength of the sheet piling;

h_a is the length of the washer plate, but $\leq$ 1,5 b_a;

t_f is the flange thickness;

b) tensile resistance of webs:

$$F_{Ed} \leq R_{tw,Rd} \tag{7.6}$$

where:

$R_{tw,Rd}$ is the design value of the tensile resistance of 2 webs, given as

$$R_{tw,Rd} = 2{,}0\ h_a\ t_w\ f_y\ /\ \gamma_{M0} \tag{7.7}$$

with:

t_w is the web thickness;

c) width of washer plate:

$$b_a \geq 0{,}8\ b \tag{7.8}$$

where:

b_a is the width of the washer plate;

b is the width of the flange, see figure in Table 5-1;

NOTE: A smaller value for b may be taken provided flange bending is checked.

d) thickness of washer plate:

the washer plate should be verified for bending and should have a minimum thickness of $2t_f$.

(4) The verification of the resistance of the sheet pile to that part of the anchor force or strut force to be introduced into the webs via a waling (see Figure 7-6) may be carried out as follows:

$F_{Ed} \leq 0,5\ R_{c,Rd}$: no further verification necessary

$F_{Ed} > 0,5\ R_{c,Rd}$: $$\frac{F_{Ed}}{R_{c,Rd}} + 0,5\frac{M_{Ed}}{M_{c,Rd}} \leq 1,0 \qquad (7.9)$$

where:

F_{Ed} is the design value of the local transverse force per web applied through the waling;

$R_{c,Rd}$ is the design resistance to the local transverse force. $R_{c,Rd}$ should be taken as the minimum of $R_{e,Rd}$ and $R_{p,Rd}$ for each web, given by:

$$R_{e,Rd} = \frac{\varepsilon}{4e}\left(s_S + 4,0\ s_{ec}\right)\sin\alpha\left(t_w^2 + t_f^2\right)f_y / \gamma_{M0} \qquad (7.10)$$

$$R_{p,Rd} = \chi\, R_{po} / \gamma_{M0} \qquad (7.11)$$

with:

$$\chi = 0,06 + \frac{0,47}{\lambda} \leq 1,0 \qquad (7.12)$$

$$\lambda = \sqrt{\frac{R_{p0}}{R_{cr}}} \qquad (7.13)$$

$$R_{cr} = 5,42\, E\frac{t_w^3}{c}\sin\alpha \qquad (7.14)$$

$$R_{p0} = \sqrt{2}\,\varepsilon\, f_y\, t_w \sin\alpha\left(s_S + t_f\sqrt{\frac{2b\sin\alpha}{t_w}}\right) \qquad (7.15)$$

b is the width of the flange, see figure in Table 5-1;

c is the slant height of the web as shown in Figure 5-1;

e is the eccentricity of the force introduced into the web, given by

$$r_0 \tan\left(\frac{\alpha}{2}\right) - \frac{t_w}{2\sin\alpha}\text{, but not less than 5 mm;} \tag{7.16}$$

f_y is the yield strength of the sheet pile;

r_0 is the outside radius of the corner between flange and web;

$$s_{ec} = 2{,}0\pi\, r_0\left(\frac{\alpha}{180}\right) \text{ with } \alpha \text{ in degrees;} \tag{7.17}$$

s_s is the length of stiff bearing, determined from 6.3 of EN 1993-1-5. If the waling consists of two parts, e.g. two channel-sections, s_s is the sum of both parts plus the minimum of the distance between the two parts or the length s_{ec};

t_f is the flange thickness;

t_w is the web thickness;

α is the inclination of the web, see Figure 5-1;

ε = $\sqrt{\frac{235}{f_y}}$ with f_y in N/mm²;

M_{Ed} is the design value of the bending moment at the location of the anchor force or strut force;

$M_{c,Rd}$ is the design bending resistance of the sheet pile from 5.2.2(2).

(5) If a washer plate is used for the introduction of the anchor force into the webs according to Figure 7-5b the expressions given in (4) may be applied, provided that the width of the washer plate is greater than the width of the flange to prevent an additional eccentricity e as given in (4).

8 Execution

8.1 General

(1)P The piling works shall be carried out in accordance with the project specification.

(2)P If there are differences between what is constructed on site and the project specification, the consequences shall be investigated and modifications shall be introduced if necessary.

(3) The execution requirements should conform with EN 1997-1.

(4) Any specific requirements should be given in the project specification.

8.2 Steel sheet piling

(1)P Sheet piling shall be executed in accordance with EN 12063.

(2)P The tolerances for position and verticality of sheet piles shall be as specified in Table 2 of EN 12063.

(3) In order for the piling to develop its nominal resistance and stiffness properties, the wall alignment should be in accordance with 8.5 of EN 12063.

8.3 Bearing piles

(1)P The installation of bearing piles shall conform with 4 of EN 1997-1.

(2)P The installation of bearing piles shall also be in accordance with EN 12699 and EN 14199.

(3)P The tolerances for position and verticality of bearing piles shall be as specified in EN 12699 and EN 14199.

8.4 Anchorages

(1)P The execution of anchorages shall be in accordance with EN 1997-1 and EN 1537 if applicable.

A [normative] - Thin walled steel sheet piling

A.1 General

A.1.1 Scope

(1) This annex should be used for the determination of the resistance and stiffness of steel sheet piling and for some special aspects of cold-formed steel sheet piling with class 4 cross-sections. For the determination of actions and effects of actions, reference should be made to section 2.

(2) Reference should be made to 5.2 for the classification of cross-sections.

(3) Although the design methods in this annex are presented in terms of cold-formed sheet piling, they may also be applied to class 4 hot rolled profiles.

(4) Design assisted by calculation included in this document, assumes that the cross-sections are limited to those made up of elements without intermediate stiffeners. This restriction need not be applied to the design assisted by testing, see A.7. For profiles made up of elements with intermediate stiffeners and designed by calculation reference should be made to EN 1993-1-3.

(5) In the case of thin walled steel sheet piling, design by calculation might not always lead to economic solutions and it is often useful to use tests for the determination of resistance.

NOTE: Guidance for testing are given in Annex B.

(6) Restrictions regarding geometrical properties or materials only apply to design by calculation.

A.1.2 Form of cold formed steel sheet piles

(1) Cold formed steel sheet piles are products made from hot rolled flat products according to EN 10249. They consist of straight and rounded walls. Over their entire length, within the permitted tolerances, they have a constant cross-section and a thickness not less than 2 mm.

(2) These sheet piles are obtained solely by cold forming (rolling or pressing).

(3) The edges of the cross-section of a sheet pile might consist of interlocks.

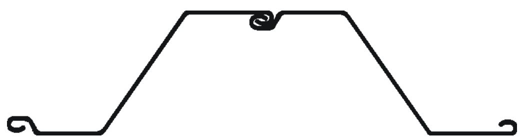
(4) Some examples of cold formed piling sections covered in this annex are given in Table A-1.

A.1.3 Terminology

(1) The terminology for cross-section dimensions given in 1.5.3 of EN 1993-1-3 applies.

(2) For cold formed steel sheet piles the axis convention given in 1.9 applies.

Table A-1: Examples of cold formed piling sections

	Example of cross-section
Ω - profile	
Z - profile	
Trench sheet profile	

A.2 Basis of design

A.2.1 Ultimate limit states

(1)P The general provisions given in 2.2 and 5.1 shall also be applied to cold formed profiles, except where different provisions are given in this annex.

A.2.2 Serviceability limit states

(1)P The general provisions given in 2.3, 6.1 and 6.2 shall also be applied to cold formed profiles, except where different provisions are given in this annex.

(2) Reference should be made to section 7 of EN 1993-1-3 for serviceability limit state verifications.

A.3 Properties of materials and cross-sections

A.3.1 Material properties

(1)P For the properties of the materials covered in this annex reference shall be made to section 3.

(2) The provisions given in this annex apply to class 4 steel sheet piles according to EN 10248 and EN 10249.

(3) These design methods may also be applied to other structural steels with similar strength and toughness properties, provided that all of the following conditions are satisfied:

- the steel satisfies the requirements for chemical analysis, mechanical tests and other control procedures to the extent and in the manner prescribed in EN 10248 or EN 10249;
- a minimum ductility is required that should be expressed in terms of limits of
  - f_u / f_y
  - the elongation at failure on a gauge length of 5,65 $\sqrt{A_0}$ (where A_0 is the original cross-section area)
  - the ultimate strain ε_u, where ε_u corresponds to the ultimate strength f_u.

  NOTE: These limiting values may be given in the National Annex. The following values are recommended:

  - $f_u / f_y \geq 1,1$;
  - elongation at failure ≥ 15 %;
  - $\varepsilon_u \geq 15\ \varepsilon_y$;
  - where ε_y corresponds to the yield strength f_y;
- the steel is supplied either:
  - to another recognized standard for structural steel sheet;
  - with mechanical properties and chemical composition at least equivalent to one of the steel grades that are listed in Table 3-1 or Table 3-2 respectively.

(4) The nominal values of the basic yield strength f_{yb} given in Table 3-1 and Table 3-2 should be adopted as characteristic values in design calculations. For other steels the characteristic values should be based on the results of tensile tests carried out in accordance with EN 10002-1.

(5) It may be assumed that the properties of steel in compression are the same as those in tension.

(6) For the steels covered by this annex, the other material properties to be used in design should be taken as follows:

- modulus of elasticity: E = 210 000 N/mm^2;
- shear modulus: G = E / [2(1 + ν)] N/mm^2;
- Poisson's ratio: ν = 0,3;

- coefficient of linear thermal elongation: α = $12 \times 10\text{-}6$ 1/K;

- unit mass: ρ = 7850 kg/m^3.

(7) The effect of an increased yield strength due to cold forming may be taken into account on the basis of tests in accordance with A.7.

(8) Where the yield strength is specified using the symbol f_y either in this annex or in EN 1993-1-3, either the basic yield strength f_{yb} from Table 3-2 or the yield strength from Table 3-1 should be used.

NOTE: This differs from the convention used in EN 1993-1-3.

(9) The provisions for design by calculation given in this annex may be used only for steel within the range of nominal thickness t as follows:

2,0 mm $\leq t \leq$ 15,0 mm.

(10) For thicker or thinner class 4 steel sheet pile cross-sections, the load bearing capacity should be determined by design assisted by testing in accordance with A.7.

A.3.2 Section properties

(1) Section properties should be calculated, taking due account of the sensitivity of the properties of the overall cross-section to any approximations used, see 5.1 of EN 1993-1-3, and their influence on the predicted resistance of the member.

(2) The effects of local buckling should be taken into account by using effective cross-sections as specified in A.4.

(3) The properties of the gross cross-section should be determined using the specified nominal dimensions. In calculating gross cross-sectional properties, small holes need not be deducted but allowance should be made for large openings.

(4) The net area of a pile cross-section, or an element of a cross-section, should be taken as its gross area minus appropriate deductions for all holes and openings.

(5) The influence of rounded corners on the profile properties should be taken into account according to 5.1.4 of EN 1993-1-3.

NOTE: An example of an idealized sheet pile cross-section with sharp corners is given in Figure A-1.

(6) For design by calculation, the width-to-thickness ratios should not exceed the values given in Table A-2.

(7) The use of width-to-thickness ratios exceeding these values is not precluded, but the resistance of the pile at ultimate limit states and its behaviour at serviceability limit states should be verified by testing in accordance with A.7.

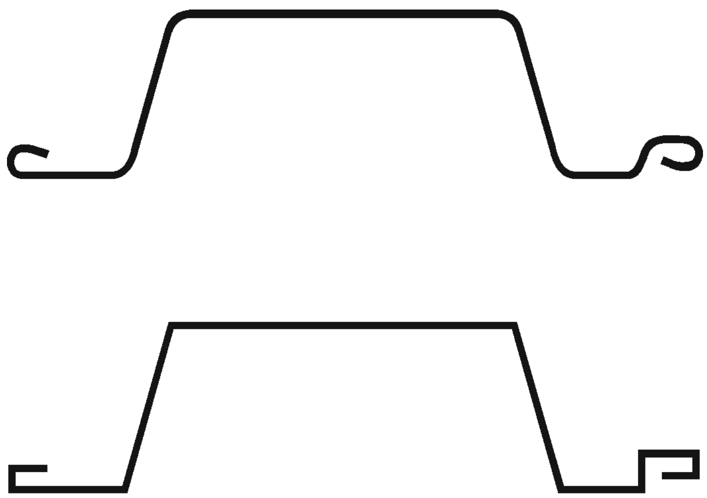
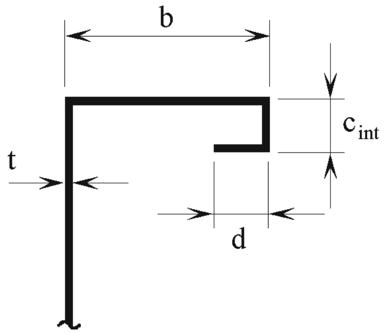
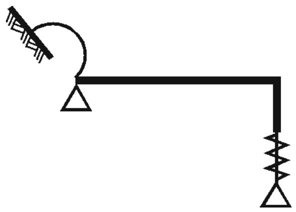
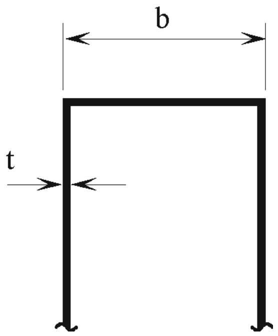
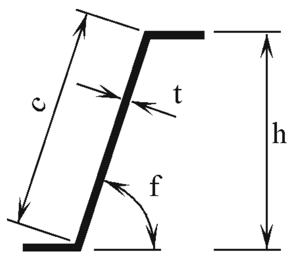
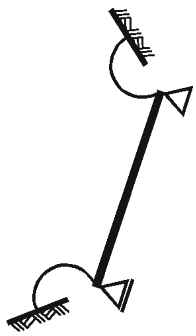


Figure A-1: Example of an idealized cross-section

Table A-2: Maximum width-to-thickness ratios; modelling of statical behaviour

Part of Cross section	Modelling of statical behaviour	
		b/t ≤ 90
		b/t ≤ 200
		45° ≤ ϕ ≤ 90° c/t ≤ 200

A.4 Local buckling

(1) The effects of local buckling should be taken into account in determining the resistance and stiffness of class 4 steel sheet pile cross-sections according to section 5.5 of EN 1993-1-3, except where different provisions are given in this annex.

(2) Unstiffened plane elements of sheet pile cross-sections are covered in 5.5.2 of EN 1993-1-3.

(3) Plane elements with interlocks acting as edge stiffeners should be taken into account according to 5.5.3.2 of EN 1993-1-3.

> **NOTE:** Figure A-2 gives an example of the idealization of the geometry of the interlock acting as an edge stiffener.

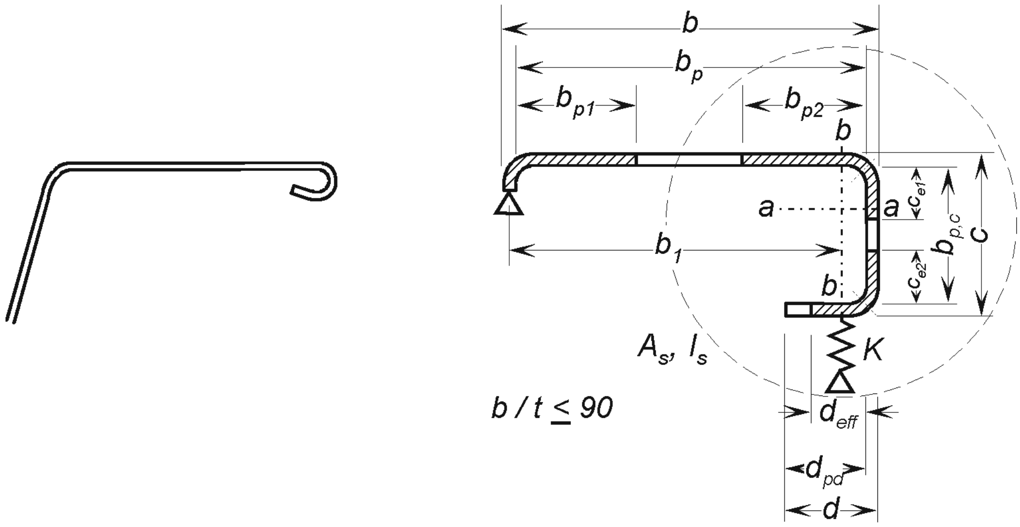


Figure A-2: Interlock to be treated as an edge stiffener

(4) For plane compression elements with interlocks acting as edge stiffeners, the design should be based on the principle given in 5.5.3.1 (1) of EN 1993-1-3.

(5) The spring stiffness of the interlock acting as an edge stiffener should be determined according to expression (5.10) of EN 1993-1-3.

(6) Expression (5.10) of EN 1993-1-3 may be applied to sheet piling as follows for the Z-profile as shown in Figure A-3 and Figure A-4, by using the plate bending stiffness ($E\,t^3$) / 12 / (1 - ν^2). The stiffness of the rotational spring representing the web, see Figure A-4, may be determined from:

$$EI_w\,\theta = \tfrac{1}{2} \times 1 \times 1 \times s_w \qquad \text{(A.1)}$$

$$C_\theta = \frac{1}{\theta} = \frac{2EI_w}{c} \qquad \text{(A.2)}$$

$$I_w = \frac{t^3}{12(1-\nu^2)}\ . \qquad \text{(A.3)}$$

The actual bending moment acting in the rotational spring due to the unit load is $u \times b_p$ and the corresponding rotation is given by:

$$\theta = \frac{u\,b_p}{C_\theta} = \frac{u\,b_p\,c}{2EI_w} \qquad \text{(A.4)}$$

So expression (5.10) of EN 1993-1-3 becomes:

$$\delta = \frac{2u\,b_p^2(1-\nu^2)}{E\,t^3}(3c + 2\,b_p) \qquad \text{(A.5)}$$

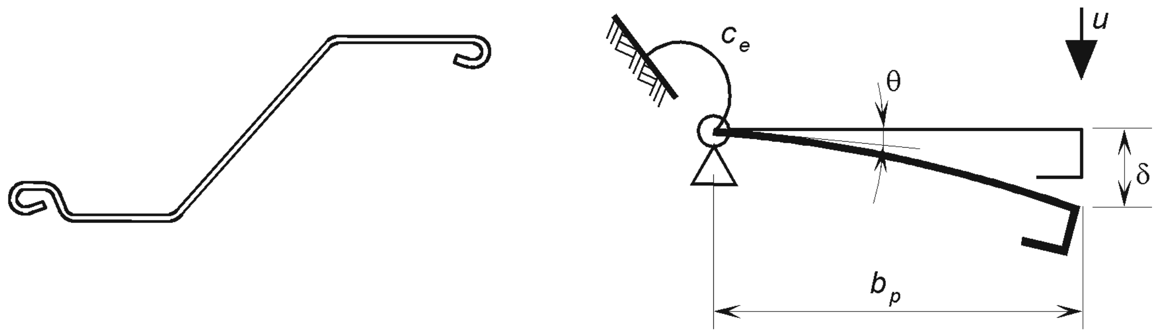


Figure A-3: Determination of spring stiffness of the flange

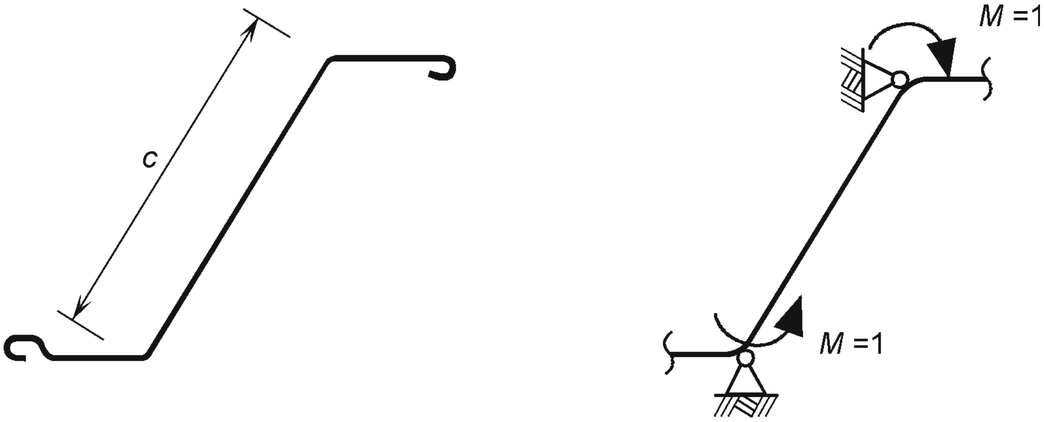


Figure A-4: Determination of the spring stiffness of the web

A.5 Resistance of cross-sections

A.5.1 General

(1)P The design values of the internal forces and moments at each cross-section shall not exceed the design values of the corresponding resistances.

(2)P The design resistance of a cross-section shall be determined either by calculation, using the methods given in this section, or by design assisted by testing, in accordance with A.7.

(3)P The provisions of A.5 shall not be applied except for monoaxial bending with $M_z = 0$.

(4) It may be assumed that one of the principal axes of the sheet piling is parallel to the system axis of the retaining wall.

(5) For design by calculation, the resistance of the cross-section should be verified for:

- bending moment, taking into account the effects of local transverse bending;
- local transverse forces;
- combined bending moment and shear force;
- combined bending moment and axial force;
- combined bending moment and local transverse forces.

(6) Design assisted by testing may be used instead of design by calculation for any of these resistances.

NOTE: Design assisted by testing is particularly likely to be beneficial for cross-sections with relatively high b_p / t ratios, for instance in relation to inelastic behaviour or web crippling.

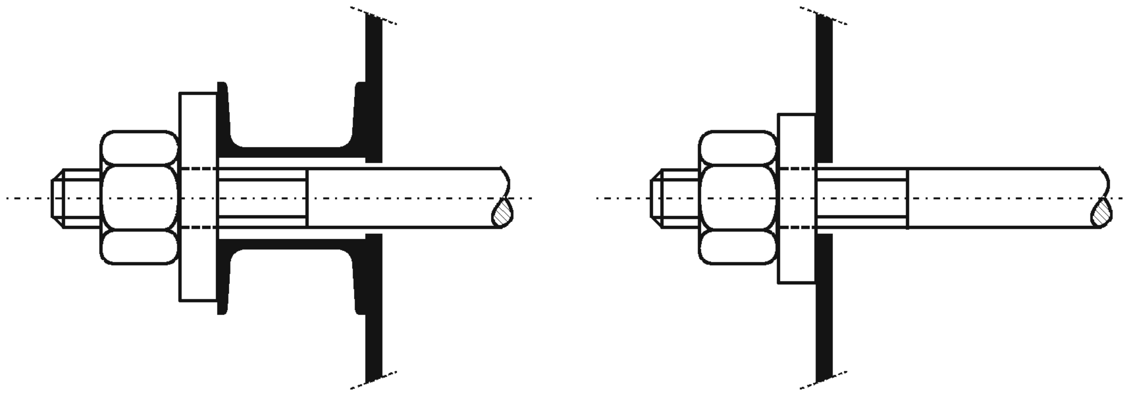
(7) For design by calculation, the effects of local buckling should be taken into account by using effective cross-sectional properties determined as specified in A.4.

(8) The provisions given in this section do not account for possible global instability of the sheet piles, so for sheet piling where instability due to compression forces might occur, reference should be made to section 6.2 of EN 1993-1-3.

(9) The criterion given in 5.2.3(1)P should be applied. Higher axial forces leading to overall instability should be avoided when using class 4 cross-sections.

(10) Walings in front of or behind the sheet pile wall should be used to introduce forces from anchors or struts (see Figure A-5a), thereby allowing for redistribution of the forces. If a washer plate is used to introduce the force from a tie rod directly into the sheet pile as shown in Figure A-5b, tests in accordance with section 2.6 should be carried out if the thickness of the sheet pile profile is ≤ 6 mm.

(11) When using iterative calculation procedures, several iterations should be carried out if necessary to avoid a lack of accuracy.



a) with a waling b) with a washer plate

Figure A-5: Introduction of anchor forces

A.5.2 Bending moment

(1) The moment resistance of the class 4 sheet pile cross-section should be determined according to 6.1.4 of EN 1993-1-3, except where different provisions are given in this annex.

(2) The effects of shear lag may be neglected in steel sheet piling.

(3) No plastic redistribution of bending moments should be made in retaining walls consisting of class 4 cross-sections.

(4) If the moment resistance of the profile is different for positive and negative bending moments, this should be taken into account in the design.

A.5.3 Shear force

(1) The shear resistance of the web should be determined according to 6.1.5 of EN 1993-1-3, except where different provisions are given in this annex.

(2) The shear buckling strength f_{bv} should be determined using Table 6-1 of EN 1993-1-3 for webs without stiffening at the support.

A.5.4 Local transverse forces

A.5.4.1 General

(1) If the waling is located in front of the wall on the excavation side as shown in Figure 7-6, the verification should be carried out according to A.5.4.2.

(2) If the waling is located behind the wall as shown in Figure 7-4, the verification should be carried out according to A.5.4.3.

A.5.4.2 Webs subject to transverse compressive forces

(1) To avoid crushing, crippling or buckling in a web subject to a support reaction via a waling, the applied transverse force F_{Ed} should satisfy:

$$F_{Ed} \leq R_{w,Rd}$$

where:

$R_{w,Rd}$ is the local transverse resistance of the web.

(2) For an unstiffened web, the local transverse resistance $R_{w,Rd}$ should be obtained from 6.1.7.3 of EN 1993-1-3 except where different provisions are given in this annex.

NOTE: Z-profiles are covered by this paragraph, considering a double pile made up of two Z-profiles.

(3) For a waling acting as support:

- the value of the effective bearing length l_a to be used in expression (6.18) of EN 1993-1-3 should be determined according to 6.1.7.3 (4) of EN 1993-1-3;

- the value of the coefficent α to be used in expression (6.18) of EN 1993-1-3 should be obtained from the following:

  for category 1: $\alpha = 0,075$

  for category 2: $\alpha = 0,15$.

NOTE: Category 1 applies if the distance between the waling and the edge of the pile is $\leq 1,5\ h_w$, where h_w is the depth of the profile, otherwise category 2 applies, see Figure 6-9 of EN 1993-1-3.

A.5.4.3 Webs subject to transverse tensile forces

(1) For webs subject to transverse tensile forces, checks should be carried out in accordance with 7.4.3 (3).

A.5.5 Combined shear force and bending moment

(1) For combined shear force and bending moment, the verification should be carried out using expression (6.27) of EN 1993-1-3.

A.5.6 Combined bending moment and local transverse forces

(1) For combined bending moment and local transverse forces, the verification should be carried out according to 6.1.11 of EN 1993-1-3.

A.5.7 Combined bending moment and axial force

(1) The combination of bending moment with axial tension should be verified according to 6.1.8 of EN 1993-1-3, without taking bending about the z-z axis into account.

(2) The verification for combined bending moment and axial compression should be carried out according to 6.1.9 of EN 1993-1-3 without taking bending about the z-z axis into account.

A.5.8 Local transverse bending

(1)P In the case of a differential water pressure exceeding 1 m head, the effects of water pressure on transverse local plate bending shall be taken into account when determining the overall bending resistance.

(2) As a simplification, this verification may be carried out using the following procedure:

- the cross-sectional verification need only be carried out at the locations of the maximum moments where the differential water pressure is more than 1 m head;

- the effect of differential water pressure should be taken into account by using a reduced plate thickness $t_{red} = \rho_P\, t$ with ρ_P according to Table A-3;

- for the determination of ρ_P according to Table A-3 the differential water pressure acting at the relevant locations of the maximum moments should be taken into account.

Table A-3: Reduction factors ρ_P for plate thickness due to differential water pressure

w	$(b/t_{min})\ \varepsilon = 40,0$	$(b/t_{min})\ \varepsilon = 60,0$	$(b/t_{min})\ \varepsilon = 80,0$	$(b/t_{min})\ \varepsilon = 100,0$
1,0	0,99	0,98	0,96	0,94
2,5	0,98	0,94	0,88	0,78
5,0	0,95	0,86	0,67	0,00
7,5	0,92	0,75	0,00	0,00
10,0	0,88	0,58	0,00	0,00

Key:

b is the width of the flange, but b should not be taken as less than $c/\sqrt{2}$, where c is the slant height of the web;

t_{min} is the minimum thickness of flange or web;

w is the head of differential water pressure in m;

$\varepsilon = \sqrt{\frac{235}{f_y}}$, with f_y in N/mm²

Note: These values apply to Z-piles and are conservative for Ω- and U-piles. An increase of ρ_P is possible (for instance if interlocks are welded), but an additional investigation is then necessary.

A.6 Design by calculation

(1) The following procedure may be adopted for the design of a retaining wall made up of class 4 sheet piles.

(2) The effects of actions in the piles at ultimate limit states may be determined using an elastic beam model and an appropriate model for the soil in accordance with EN 1997-1.

(3) If required, the structural input data for the beam model should be chosen as a best estimate.

(4) For axial compression it should be verified whether buckling may be neglected.

(5) For design by calculation to be applicable, it should be verified that the corresponding criteria given in this annex are fulfilled by the steel sheet piles that are expected to be used.

(6) Based on the resistances of the cross-sections provided by the manufacturer of the steel sheet piles, the chosen pile cross-section should be verified according to A.5, making due allowance for corrosion effects, if necessary.

> **NOTE:** The cross-section resistance data that might be provided by the manufacturer are: $M_{c,Rk}$, N_{Rk}, $V_{b,Rk}$, $R_{w,Rk}$, taking into account the steel grade and the reduced thickness due to corrosion.

(7) If required, the effective stiffness of the cross-section at ultimate limit states should be used with the beam model in an iterative procedure.

> **NOTE:** The stiffness data for the cross-section at ultimate limit states might be provided by the manufacturer in section property tables.

(8) If a verification at serviceability limit states is required, an elastic beam model combined with an appropriate model for the soil in accordance with 1997-1 may be used.

(9) Reference should be made to section 7.1 of EN 1993-1-3 for the determination of the cross-section stiffness data to be used for serviceability states verifications.

A.7 Design assisted by testing

A.7.1 Basis

(1) The following procedure should be used to apply the principles for design assisted by testing given in section 5 of EN 1990, to the specific requirements of cold formed steel sheet piling.

(2) Although the following provisions have been developed for cold formed profiles, they may also be applied to hot rolled steel sheet piles.

(3) Testing may be undertaken under any of the following circumstances:

a) if the properties of the steel are unknown;

b) if there is a need to take account of the actual properties of the cold formed profile;

c) if adequate analytical procedures are not available for designing a sheet pile profile by calculation alone;

d) if realistic data for design cannot otherwise be obtained;

e) if the performance of an existing structure needs to be checked;

f) if it is desirable to build a number of similar structures or components on the basis of a prototype;

g) if confirmation of consistency of production is required;

h) if it is necessary to prove the validity and adequacy of an analytical procedure;

i) if it is desirable to produce resistance tables based on tests, or on a combination of testing and analysis;

j) if it is desirable to take into account practical factors that might alter the performance of a structure, but are not addressed by the relevant analysis method in design by calculation.

(4) Testing as a basis for tables of load carrying capacity should be carried out in accordance with A.7.3.

NOTE: Information is given in Annex B on procedures for thin walled steel sheet piles.

(5) Tensile testing of steel should be carried out in accordance with EN 10002-1. Testing of other steel properties should be carried out in accordance with the relevant European Standards.

A.7.2 Conditions

(1) The provisions given in A.3.1 of EN 1993-1-3 should be applied, except where different provisions are given in this annex.

(2) During load application, up to attainment of the service load, the load may be removed and then reapplied. For this purpose the service load may be estimated as 30 % of the ultimate load. Above the service load, the loading should be held constant at each increment until any time-dependent deformations due to plastic behaviour have become negligible.

A.7.3 Cross-sectional data based on testing

(1) The cross-sectional resistances and the effective stiffness of a cold formed steel sheet pile may be determined according to A.4.2 of EN1993-1-3.

B [informative] - Testing of thin walled steel sheet piles

B.1 General

(1) Loading may be applied through air bags, or by cross beams arranged to simulate distributed loading. To prevent distortion of the profile at the points of load application or support, transverse ties and/or stiffeners (such as timber blocks or steel plates) may be applied.

(2) For tests on Z-piles at least one double sheet pile should be used.

(3) For Ω-piles at least one sheet pile should be used.

(4) The accuracy of measurement should be consistent with the magnitude of the measurements and should be within +/- 1% of the value to be determined.

(5) The cross-sectional measurements of the test specimen should cover the following geometrical properties:

- overall dimensions (width, depth and length) to an accuracy of +/- 1,0 mm;
- width of flat profile parts to an accuracy of +/- 1,0 mm;
- radii of bends to an accuracy of +/- 1,0 mm;
- inclination of flat walls (angle between two surfaces) to an accuracy of +/- 2°;
- the thickness of the material to an accuracy of +/- 0,1 mm.

(6) It should be ensured that the load direction remains constant during the test.

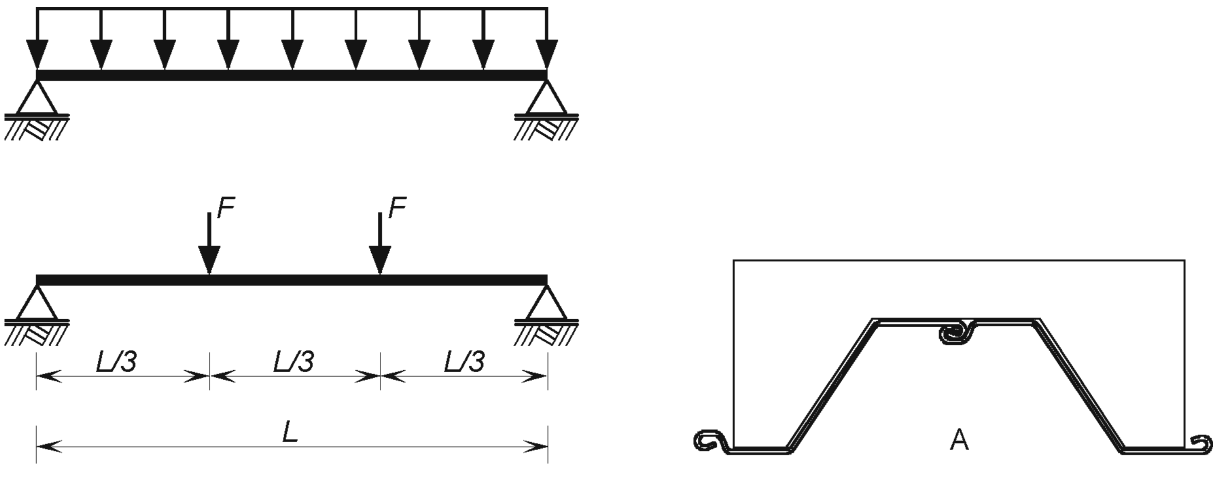
B.2 Single span beam test

(1) The test setup shown in Figure B-1 should be used to obtain the moment resistance (when the shear force is negligible) and the effective bending stiffness.

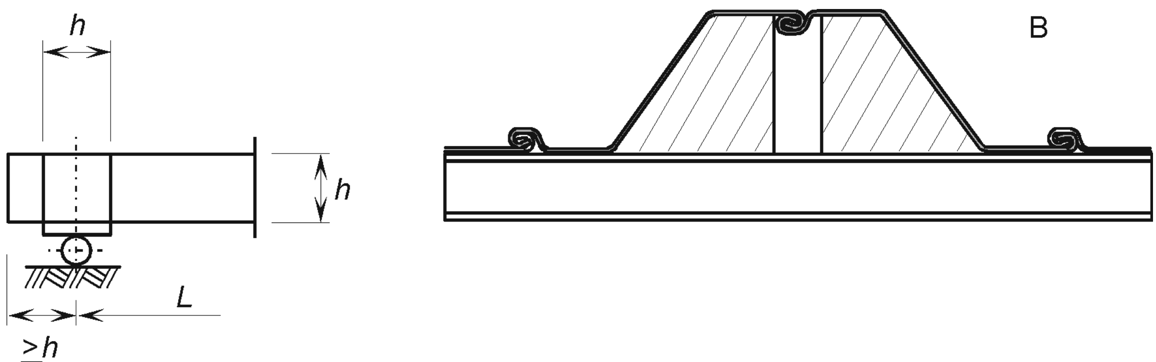
(2) In this test at least two load points as shown in Figure B-1 should be used.

(3) The span should be chosen in such a way that the test results represent the moment resistance of the sheet piling. The deflections should be measured in the middle of the span on both sides of the sheet (excluding the deformations of the supports).

(4) The maximum load applied to the specimen coincident with or prior to failure should be recorded as representing the ultimate bending moment resistance. The bending stiffness should be obtained from the load deflection curve.



a) Loading



b) Preventing distortion of the section

A at the loading point
B at the support

NOTE: The direction of loading might also need to be reversed for unsymmetrical sections.

Figure B-1: Test set-up for moment resistance determination

B.3 Intermediate support test

(1) The test setup shown in Figure B-2 may be used to obtain the resistance to combined bending moment and shear force at the intermediate support of sheet piling, as well as the interaction between moment and support reaction for a given support (waling) width.

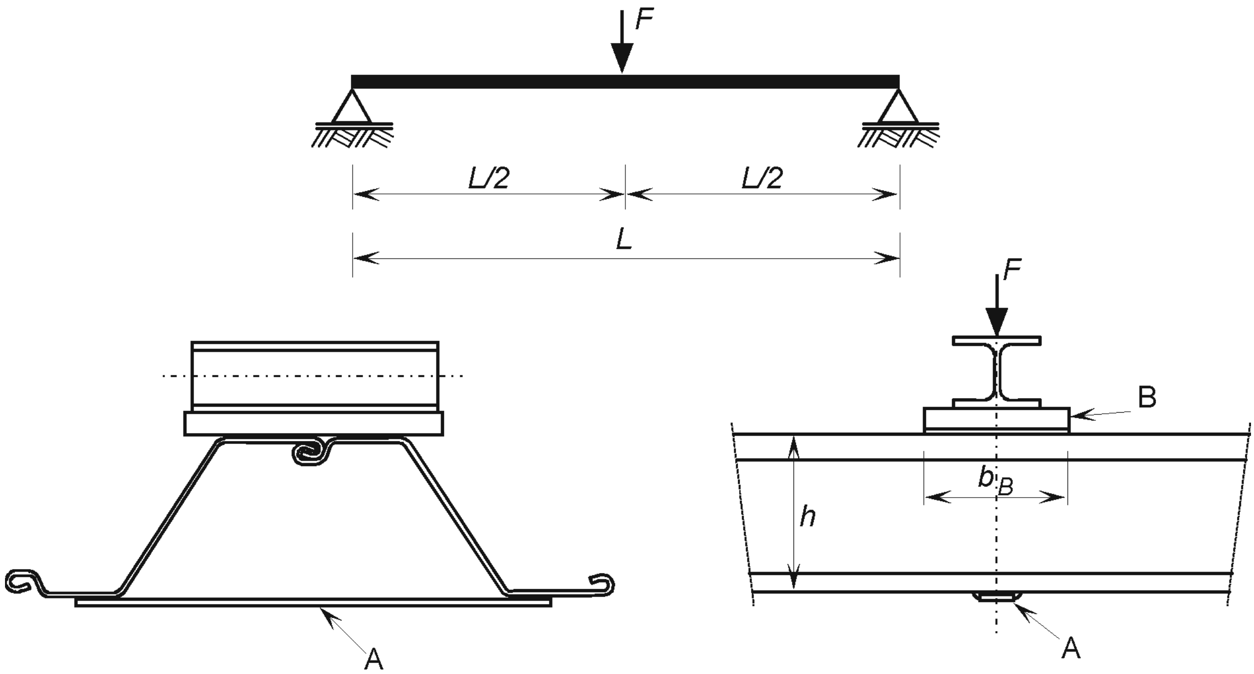
(2) In order to obtain a comprehensive record of the declining (unstable) part of the load deflection curve, the test should be continued for a suitable period after reaching the maximum load.

(3) The test span L should be selected so that it represents the portion of the pile between the points of contraflexure each side of the support.

(4) The width of the loading bar b_B should represent the waling width used in practice.

(5) The deformations of the specimen should be measured on both sides of the specimen (excluding the deformations of the supports).

(6) The maximum load applied to the specimen coincident with or prior to failure should be recorded as the ultimate crippling load. This represents the support bending moment and the support reaction for a given support width. To obtain information about the interaction between the moment and the support reaction, tests should be carried out with various spans.



A Tie B Plate

Figure B-2: Load introduction for the determination of bending resistance and shear resistance at intermediate support (waling)

B.4 Double span beam test

(1) As an alternative to B.3 double span beam tests may be carried out to determine the ultimate resistance of cold formed sheet piling. The loading should preferably be applied uniformly distributed (e.g. air bag).

(2) This loading may be replaced by any number of point loads that adequately reflect the behaviour under uniformly distributed loading (see Figure B-3).

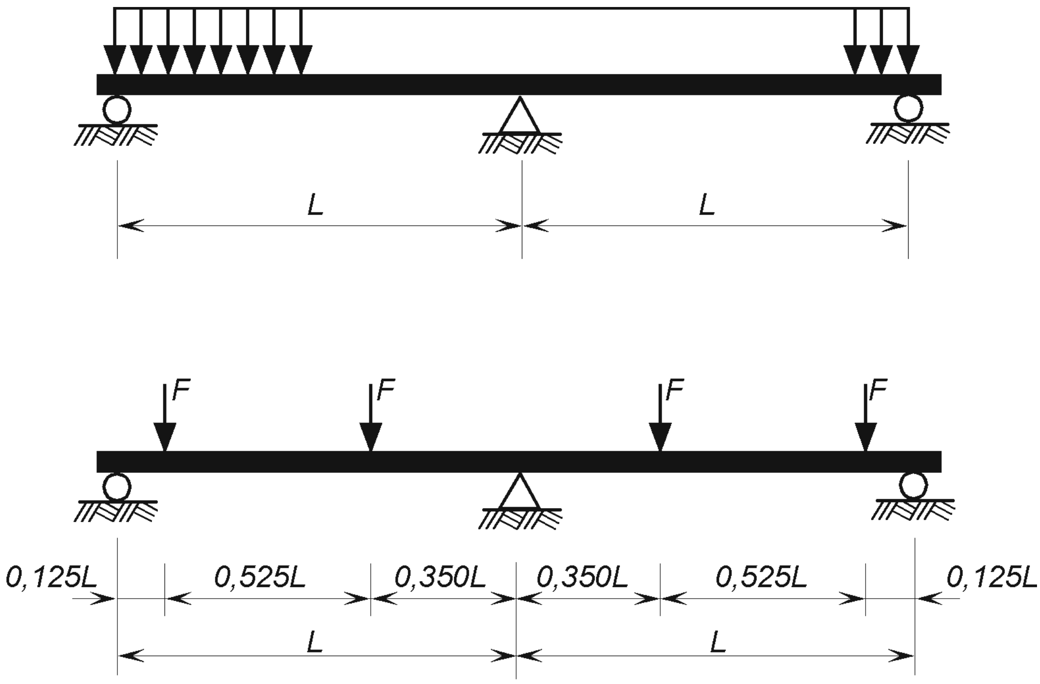


Figure B-3: Test set-up for double span tests

B.5 Evaluation of test results

B.5.1 General

(1) A specimen under test should be regarded as having failed if the applied test loads reach their maximum values, or if gross deformations exceeding agreed limits occur, see A.6.1 of EN 1993-1-3.

B.5.2 Adjustment of test results

(1) For the adjustment of test results reference should be made to A.6.2 of EN 1993-1-3.

B.5.3 Characteristic values

(1) The characteristic value R_D may be determined from test results according to A.6.3 of EN 1993-1-3.

B.5.4 Design values

(1) The design value of a resistance R_d should be derived from the corresponding characteristic value R_k determined by testing, using:

$$R_d = R_k / \gamma_M / \eta_{sys} \qquad \text{(B.1)}$$

where:

γ_M is the partial factor for resistance according to 5.1.1 (4);

η_{sys} is a factor for differences in behaviour under test and service conditions.

NOTE 1: The value to be ascribed to the symbol η_{sys} may be given in the National Annex. For the well defined standard testing procedures given in B.2, B.3 and B.4, η_{sys} = 1,0 is recommended.

NOTE 2: The value of γ_M can be determined using statistical methods for a family of at least four tests. Reference should be made to Annex D of EN 1990.

C [informative] - Guidance for the design of steel sheet piling

C.1 Design of sheet pile cross section at ultimate limit state

C.1.1 General

(1) The design values of the effects of actions should not exceed the design resistance of the cross-section.

(2) The design resistance should be determined taking into account a carefully chosen structural design model in accordance with 2.5.

(3) If required the reduction of cross section properties due to a loss of thickness induced by corrosion should be taken into account in accordance with 4.

(4) For U-piles possible lack of shear force transmission in the interlocks should be taken into account according to 5.2.2 (2).

(5) If the sheet piling is subject to transverse bending due to differential water pressure, the effects of the water pressure should be taken into account using 5.2.4.

(6) The resistance of the cross-section to the introduction of an anchor force into the flange of the sheet pile via a washer plate, or of an anchor or strut force into the webs of the sheet pile via a waling, should be determined according to 7.4.3.

(7) If the cross-sectional properties chosen for the determination of internal forces and moments do not satisfy the criteria given in (1) to (4), a new profile (or another steel grade) should be chosen and the calculation procedure repeated.

(8) Plastic resistance may be used for class 1 and class 2 cross-sections.

(9) If no moment redistribution, and therefore no plastic rotation, is taken into account for class 1 or 2 profiles, determination of the effects of actions for the verification of the cross-section may be carried out using an elastic beam model.

(10) If moment redistribution, and therefore plastic rotation, is taken into account in a design, the following design considerations should be fulfilled:

- only class 1 or class 2 cross-sections should be used in combination with a rotation check as given below;

- the verification of the cross-sections should be carried out using a beam model that allows for plastic rotation (e.g. plastic zone or plastic hinge beam model).

C.1.2 Verification of class 1 and class 2 cross-sections

(1) The classification of a cross-section may be carried out by using b/t_f ratios according to one of the following procedures:

- classification according to Table 5-1: b/t_f ratios determined for the full plastic moment resistance;

- classification according to Table C-1 in which the b/t_f ratios are given for 85 % to 100 % of the full plastic moment resistance, in steps of 5 %.

(2) If classification with a reduced level of the full plastic moment resistance with a reduction factor ρ_C = 0,85 to 0,95 is used to determine a class 1 or class 2 cross-section, then the design resistance of the cross-section should be determined with a reduced yield strength $f_{y,red} = \rho_C f_y$.

Table C-1: Classification of cross-sections in bending on a reduced $M_{pl,Rd}$ level

Type of pile	$M_{pl.Rd}$	100 %	95 %	90 %	85 %
	Reduction factor ρ_C	1,0	0,95	0,90	0,85
U-piles	Class 1 or 2	$\frac{b/t_f}{\varepsilon} \leq 37$	$\frac{b/t_f}{\varepsilon} \leq 40$	$\frac{b/t_f}{\varepsilon} \leq 46$	$\frac{b/t_f}{\varepsilon} \leq 49$
Z-piles	Class 1 or 2	$\frac{b/t_f}{\varepsilon} \leq 45$	$\frac{b/t_f}{\varepsilon} \leq 50$	$\frac{b/t_f}{\varepsilon} \leq 60$	$\frac{b/t_f}{\varepsilon} \leq 66$

(3) A plastic design with moment redistribution using class 1 or class 2 cross-sections may be carried out, provided that it can be shown that:

$$\phi_{Cd} \geq \phi_{Ed} \tag{C.1}$$

where:

ϕ_{Cd} is the design plastic rotation angle provided by the cross-section, see Figure C-1 and Figure C-2;

ϕ_{Ed} is the maximum design rotation angle demand for the actual design case.

(4) Plastic rotation angles ϕ_{Cd} are given in Figure C-1 for different $M_{pl,Rd}$ levels, dependent on $b / t_f / \varepsilon$ ratios of the cross-section. These diagrams are based on results from bending tests with steel sheet piles, see Figure C-2.

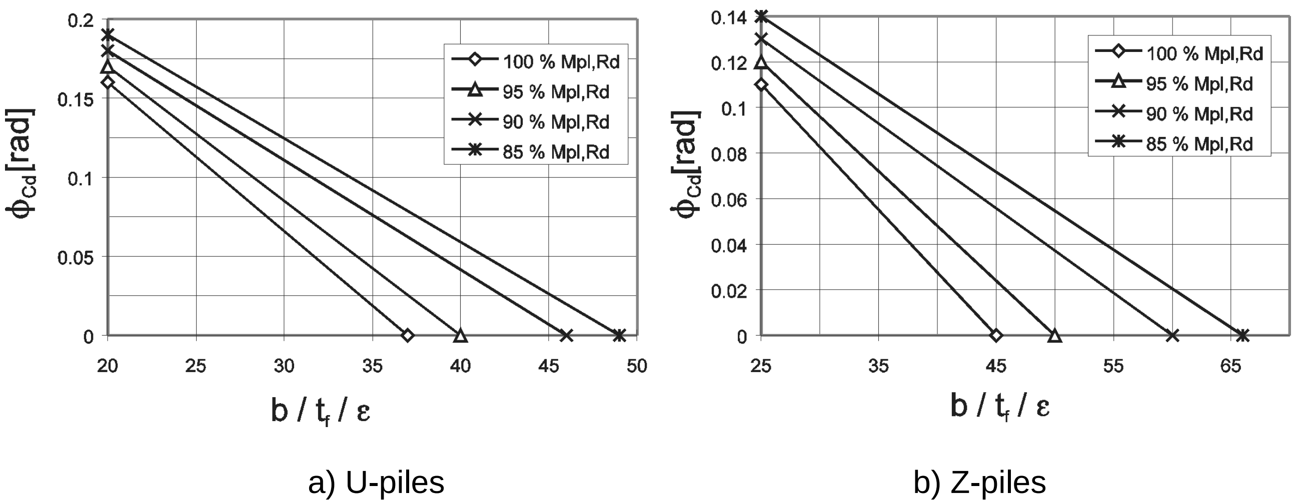


Figure C-1: Plastic rotation angle ϕ_{Cd} provided by the cross-section at different levels of $M_{pl,Rd}$

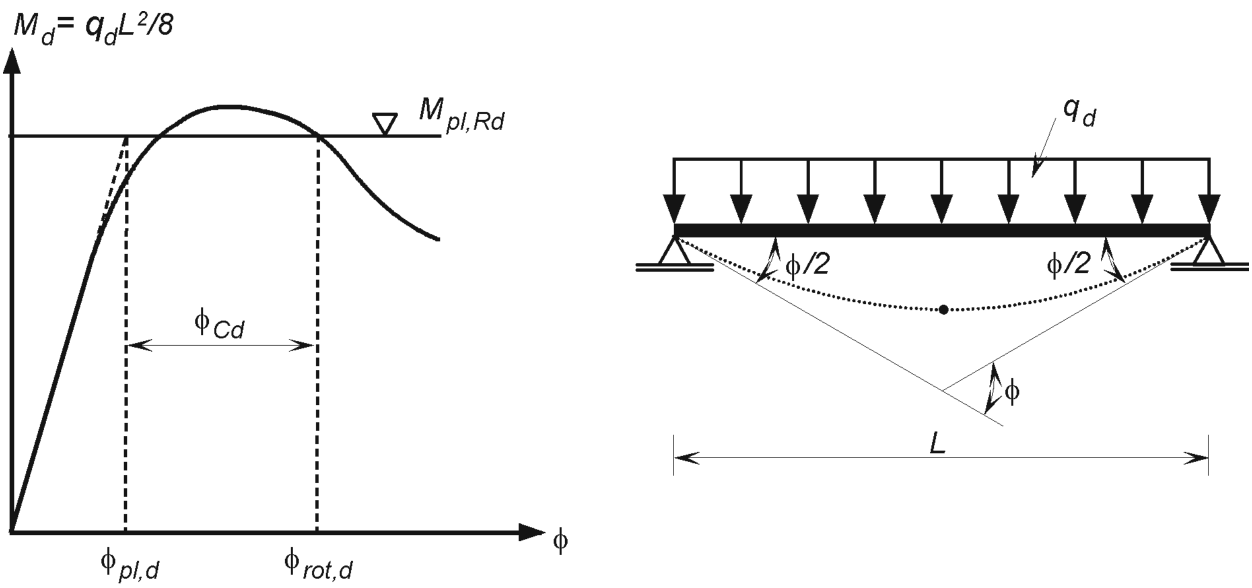


Figure C-2: Definition of the plastic rotation angle ϕ_{Cd}

(5) The design rotation angle ϕ_{Ed} for the actual design case may be determined using one of the following procedures:

a) for plastic hinge models:

ϕ_{Ed} is the maximum rotation angle in any plastic hinge;

b) alternatively for plastic hinge models and for plastic zone models:

$$\phi_{Ed} = \phi_{rot,Ed} - \phi_{pl,Ed} \tag{C.2}$$

where:

$\phi_{rot,Ed}$ is the design angle at ultimate limit state, measured at the points of zero moment (see Figure C-3);

$\phi_{pl,Ed}$ is the design elastic rotation angle, determined for the plastic moment resistance M_{pl}.

NOTE: As a simplified procedure $\phi_{pl,Ed}$ may be determined as follows:

$$\phi_{pl,Ed} = \frac{2}{3}\frac{M_{pl,Rd}\,L}{\beta_D\,EI} \qquad \text{(C.3)}$$

where:

L is the distance between the points of zero moment at ultimate limit state, see Figure C-3;

EI is the elastic bending stiffness of the sheet pile;

β_D is a factor defined in 4.4(3).

c) for plastic hinge or plastic zone models, using rotations determined from calculated displacements of the wall as shown in Figure C-4:

$$\phi_{Ed} = \phi_{rot,Ed} - \phi_{pl,Ed}, \qquad \text{(C.4)}$$

with:

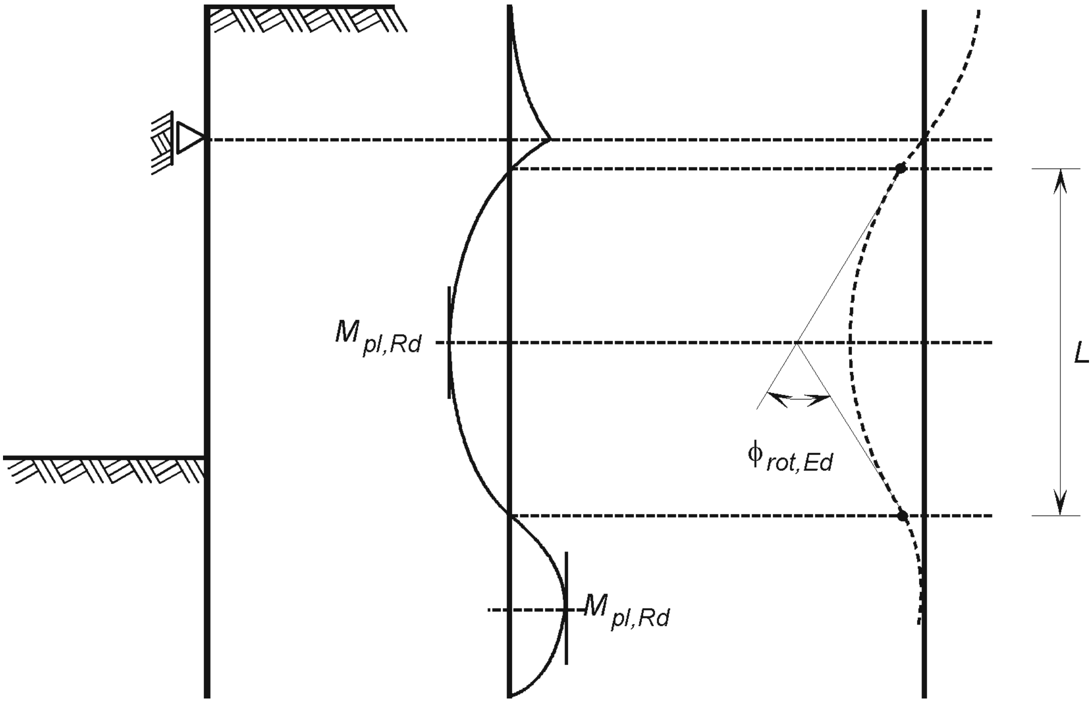
$$\phi_{rot,Ed} = \frac{w_2 - w_1}{L_1} + \frac{w_2 - w_3}{L_2} \qquad \text{(C.5)}$$

$$\phi_{pl,Ed} = \frac{5}{12}\frac{M_{pl,Rd}\,L}{\beta_D\,EI} \qquad \text{(C.6)}$$

NOTE: If the calculation program used for the design allows unloading of the sheet pile after the calculation process in order to obtain the plastic deformation, ϕ_{Ed} can be determined in this way and determination of the remaining plastic deformation is then straight forward.

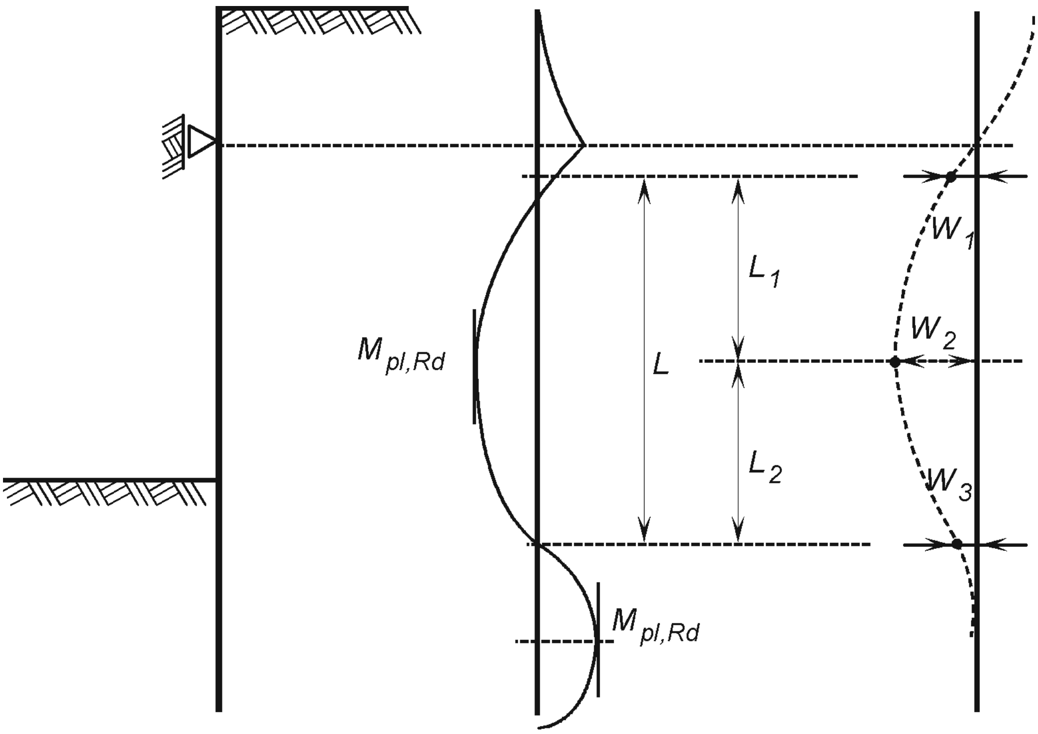
C.2 Serviceability limit state

(1) In the case of U-piles, possible lack of shear force transmission in the interlocks should be taken into account according to 6.4.



a) System b) Moment distribution c) Deflection (scaled up)

Figure C-3: Example of the determination of the total rotation angle $\phi_{rot,Ed}$



a) System b) Moment distribution c) Deflection (scaled up)

Figure C-4: Notation for the determination of the total rotation angle $\phi_{rot,Ed}$ from displacements

D [informative] - Primary elements of combined walls

D.1 I-sections used as primary elements

D.1.1 General

(1) I-sections used as primary elements in combined walls, see Figure 1-5, which appear to be class 1, class 2 or class 3 sections according to Table 5-2 of EN1993-1-1, may be verified according to the procedure given in D.1.2.

> **NOTE:** Class 4 cross-sections should be verified according to EN1993-1-3 and EN1993-1-7.

(2) If criterion (5.1) in EN1993-1-1 is not fulfilled, the global internal forces and moments should be determined using a beam model with second order theory. Reference should be made to 5.2.3 for the determination of the buckling length.

(3) If required, the local plate bending stresses due to the design forces introduced by the secondary elements via connections should be taken into account in accordance with 5.5.4, see Figure D-1.

D.1.2 Verification method

(1) If no more advanced method is used, the following simplified procedure allows for the verification of I-sections taking into account the interaction between overall bending, normal forces and local plate bending in the flanges due to design forces from the secondary elements.

> **NOTE:** Using a more advanced calculation method that takes into account both material and geometrical non-linearities might lead to a more economical design. This approach is also recommended to deal with higher water pressures exceeding 10m head.

(2) Up to a water pressure (or equivalent earth pressure in very soft soils) of 10m head the interaction between overall action effects and local plate bending may be taken into account as follows:

> The cross-sectional verification of the primary elements shall be carried out according to sections 6.2.9.2 and 6.2.10 of EN 1993-1-1, taking into account a reduced yield strength: $f_{y,red} = 0{,}9\,f_y$.
>
> Local plate bending of the flanges is verified according to (3).

(3) Local plate bending in the flanges should be verified for a cross-section through the flange located at the beginning of the fillet taking into account the design forces introduced via the connectors, see Figure D-1, using:

$$\frac{M_{Ed}}{M_{Rd}} + \left(\frac{N_{Ed}}{N_{Rd}}\right)^2 \leq 1 \qquad \text{(D.1)}$$

where M_{Ed} and N_{Ed} are the design action effects for plate bending, given by

$$M_{Ed} = m_{Ed} + w_{z,Ed}\, d \text{ and } N_{Ed} = w_{y,Ed} \tag{D.2}$$

M_{Rd} and N_{Rd} are the design values of the resistances for plate bending, given by:

$$M_{Rd} = 0{,}2875\, t^2 f_y / \gamma_{M0} \text{ and } N_{Rd} = t\, f_y / \gamma_{M0}$$

where t is the flange thickness at the beginning of the fillet.

NOTE 1: M_{Ed}, N_{Ed}, M_{Rd} and N_{Rd} are to be taken per unit length.

NOTE 2: The shear force interaction may be neglected.

(4) Reference should be made to EN1993-1-5 for the shear buckling verification of the webs.

(5) Reference should be made to section 6.3.3 of EN1993-1-1 for the overall buckling verification.

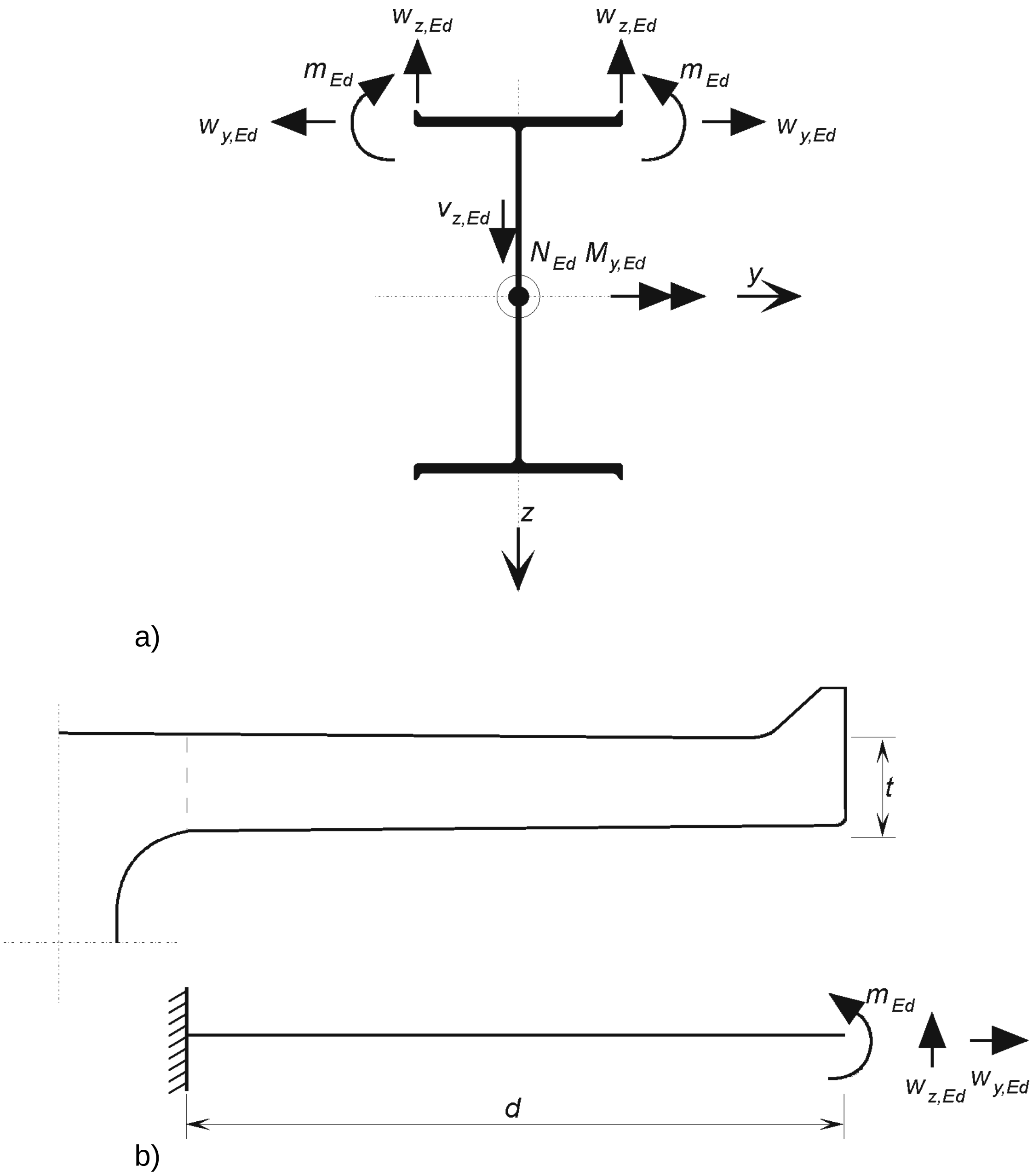


Figure D-1: I-section with overall and plate bending

D.2 Tubular piles used as primary elements

D.2.1 General

(1) Tubular piles used as primary elements in combined walls, which appear to be class 4 sections according to Table 5-2 of EN 1993-1-1, may be verified according to the following procedure.

(2) If the criterion (5.1) in EN 1993-1-1 is not fulfilled, the global internal forces and moments should be determined using a beam model with second order theory.

NOTE: To calculate F_{cr} the effect of the ovalisation on the second moment of area should be taken into account. See 5.2.3 for the determination of the buckling length.

(3) If required by section 5.5.4, the local shell bending stresses and displacements due to the design forces introduced by the secondary elements via the connectors may be estimated from Table D-1.

NOTE 1: The vertical support reactions from Figure 5-9 may be disregarded for the determination of local shell bending stresses.

NOTE 2: For simplification the horizontal forces $w_{y,Ed}$ may be assumed to act only in tension.

(4) The effect of the ovalisation of the tube due to local shell bending on the second moment of area about the wall axis, see Figure D-2, may be estimated using the reduction factor:

$$\beta_{o,I} = 1 - 1{,}5\,(e / r) \quad \text{(D.3)}$$

NOTE: The effect of the ovalisation on the section modulus may be neglected.

(5) The ovalisation e due to local shell bending, see Figure D-2 and Table D-1, may be estimated from:

$$e = 0{,}0684\, w_{y,Ed}\, \frac{r^3}{EI} \quad \text{but} \quad e \leq 0{,}1\, r \quad \text{(D.4)}$$

where:

EI is the stiffness for shell bending of the tube, given by:

$EI = E\, t^3 / 12;$

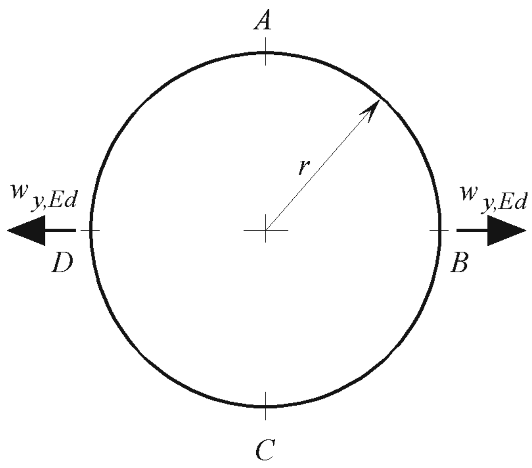
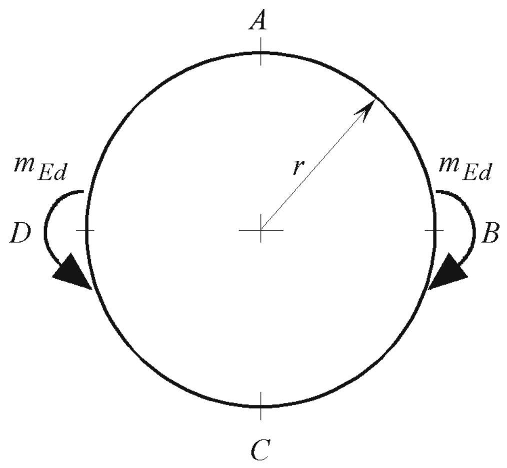
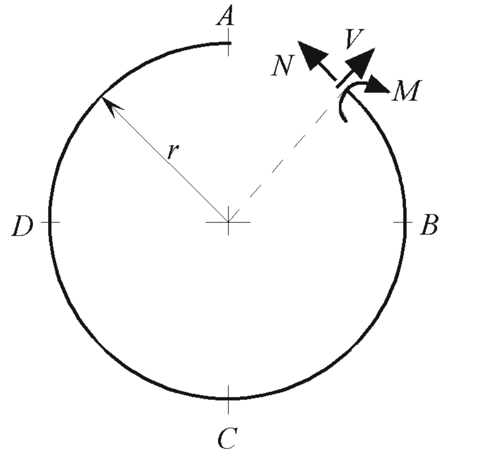
r is the mid-line radius of the tube;

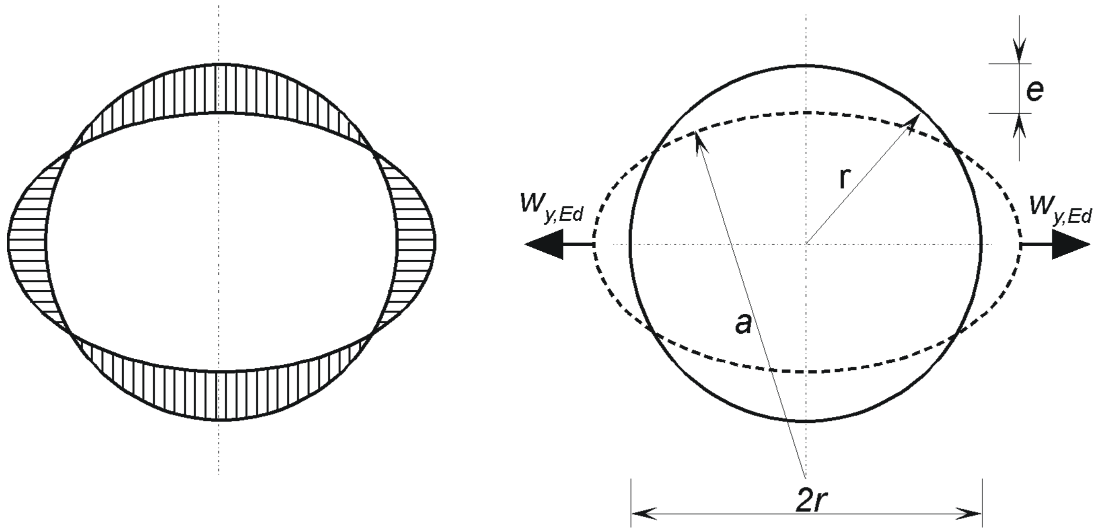
$w_{y,Ed}$ is the support reaction per unit length, determined from 5.5.2(3), see Figure 5-9.

(6) The radius of curvature a at the ovalisation, see Figure D-2, may be obtained from:

$$a = \frac{r}{1 - \frac{3e}{r}} \tag{D.5}$$

Table D-1: Local shell bending due to design forces from secondary elements

	
M_A = 0,182 $w_{y,Ed}\, r$ N_A = 0,5 $w_{y,Ed}$ V_A = 0 M_B = - 0,318 $w_{y,Ed}\, r$ N_B = 0 $V_B = \pm$ 0,5 $w_{y,Ed}$ ΔD_{BD} = 0,1488 $w_{y,Ed}\, r^3 / EI$ ΔD_{AC} = - 0,1368 $w_{y,Ed}\, r^3 / EI$	M_A = 0,137 m_{Ed} N_A = 0,637 m_{Ed} / r V_A = 0 $M_B = \pm$ 0,5 m_{Ed} N_B = 0 V_B = -0,637 m_{Ed} / r ΔD_{BD} = 0 ΔD_{AC} = 0
Where: - M, N and V are the internal forces and moments in shell bending according to the definition given in the figure. - $w_{y,Ed}$ and m_{Ed} are the design forces introduced by the secondary elements via the connecting devices. - ΔD_{BD} and ΔD_{AC} are the changes in diameter resulting from the applied forces (ovalisation). - r is the midline radius of the tube - EI is the shell bending stiffness of the tube	Definition of internal forces and moments in shell bending: 



a: radius of curvature at ovalisation
e: ovalisation due to local shell bending
r: midline radius of the tube
t: wall thickness of the tube
$w_{y,Ed}$: force introduced by the secondary elements

Figure D-2: Tube pile: geometrical data and local shell bending

D.2.2 Verification method

(1) The following procedure may be used for the verification of the tubular piles taking into account shell buckling, the interaction between overall bending, normal forces, local shell bending and overall buckling.

> **NOTE:** Alternatively the verification may be carried out according to 8.6 or 8.7 of EN1993-1-6 using a model suited for this type of analysis and which gives due consideration to the stiffening effect of the soil. This approach generally yields more economic results than the procedure given below.

(2) The buckling verification should be carried out for a cylindrical shell with a radius equal to the radius of curvature *a* at the ovalisation.

(3) Reference should be made to section 8.5 of EN1993-1-6 for the buckling verification.

(4) Shear buckling may be neglected at points of load introduction, provided that these points are stiffened by a concrete fill or appropriately designed stiffeners.

(5) If the tube is filled over a certain height with dense sand or stiff clay the circumferential compression stresses due to earth and water pressure may be neglected for the buckling verification in this part of the tube.

> **NOTE:** Information concerning the required density or stiffness may be given in the National Annex based on local experience.

(6) The critical buckling stress should be determined:

- for meridional (axial) stresses according to D.1.2.1 of EN1993-1-6 with C_x = 1,0 even for long cylinders;
- according to D.1.4.1 of EN1993-1-6 for shear stresses;
- according to D.1.3.1 of EN1993-1-6 using the boundary conditions of case 3 in Table D-3 or D-4 for circumferential compression stresses.

(7) The buckling parameters should be determined according to sections D.1.2.2, D.1.4.2 and D.1.3.2 of EN1993-1-6 respectively, taking into account quality class B for new tubes.

(8) The design values of stresses should be calculated using membrane theory in accordance with Annex A of EN1993-1-6.

(9) Reference should be made to section 8.5.3 of EN1993-1-6 for verification of the buckling strength.

NOTE 1: If the circumferential compressive stresses have to be taken into account for the buckling verification, non-uniform pressure distributions should be replaced by uniform distributions based on the maximum value.

NOTE 2: Shear may be neglected in the interaction check according to (3) of section 8.5.3 of EN 1993-1-6.

(10) The general cross-sectional verification should be carried out according to section 6.2.1 of EN1993-1-1 using the procedure given in section 6.2 of EN1993-1-6. For this verification the stresses due to both overall bending and local shell bending according to Table D-1 should be taken into account. The effect of ovalisation may be neglected and the full elastic cross-sectional properties may be used for this verification. The critical points where the yield criterion should be applied, should be determined taking into account the governing cross-sections and the governing points in those cross-sections (points A, B, C and D in Table D-1).

(11) For the overall buckling verification reference should be made to section 6.3.3 of EN1993-1-1 using full elastic cross-sectional properties, taking into account the effect of ovalisation in accordance with (4) of D.2.1.

(12) This verification may be deemed to be satisfied by verifying the interaction criterion:

$$\frac{N_{Ed}}{\frac{\chi N_{Rk}}{\gamma_{M1}}} + 1{,}5\,\frac{M_{Ed}}{\frac{M_{Rk}}{\gamma_{M1}}} \leq 1{,}0 \qquad \text{(D.6)}$$

where: N_{Ed} and M_{Ed} are the design values of the compressive force and the bending moment in the governing cross-section;

N_{Rk} and M_{Rk} are the characteristic resistances, determined in accordance with (11);

χ is the reduction factor due to overall flexural buckling taken from 6.3.1.2 of EN1993-1-1, based on a buckling length in accordance with 5.2.3.

NOTE: The slenderness should be determined according to 6.3.1.3 of EN1993-1-1, taking into account (2) of D.2.1.