BRITISH STANDARD

BS EN 1993-1-3:2006

Eurocode 3 — Design of steel structures —

Part 1-3: General rules — Supplementary rules for cold-formed members and sheeting

The European Standard EN 1993-1-3:2006 has the status of a British Standard

ICS 91.010.30

British Standards

1.5 Terminology and conventions for dimensions

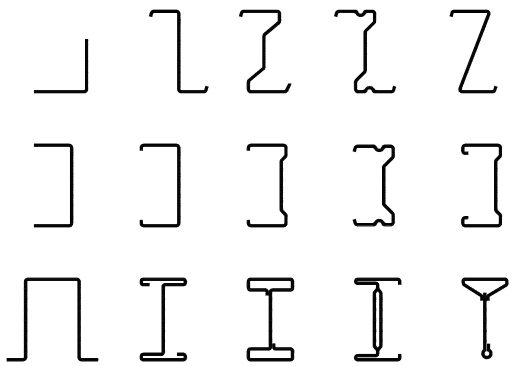
1.5.1 Form of sections

(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 uniform cross section or a tapering cross section along their length.

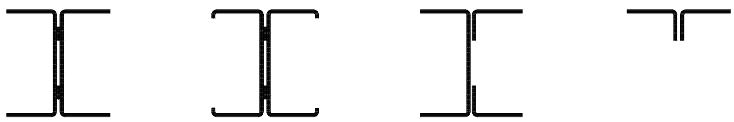
(2) The cross-sections of cold-formed members and profiled sheets essentially comprise a number of plane elements joined by curved elements.

(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



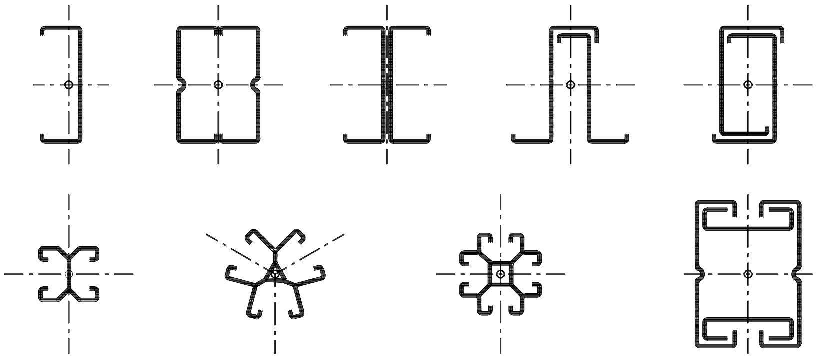
b) Open built-up sections

c) Closed built-up sections

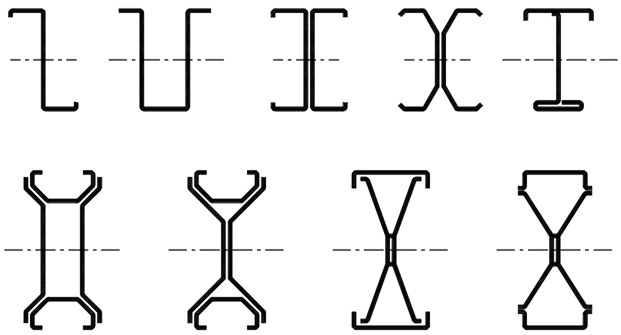
Figure 1.1: Typical forms of sections for cold-formed members

(4) Examples of cross-sections for cold-formed members and sheets are illustrated in figure 1.2.

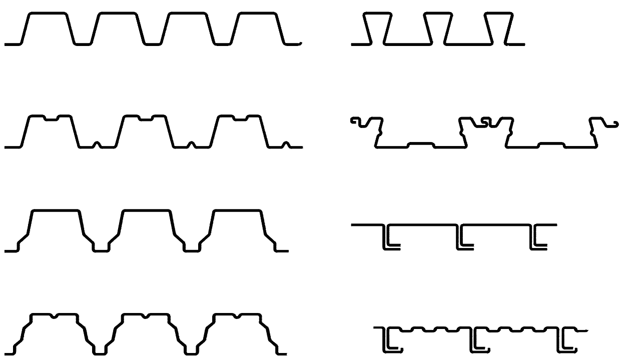
NOTE: All rules in this Part 1-3 of EN 1993 relate to the main axis properties, which are defined by the main axes y - y and z - z for symmetrical sections and u - u and v - v for unsymmetrical sections as e.g. angles and Zed-sections. In some cases the bending axis is imposed by connected structural elements whether the cross-section is symmetric or not.



a) Compression members and tension members



b) Beams and other members subject to bending



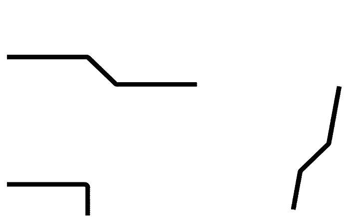
c) Profiled sheets and liner trays

Figure 1.2: Examples of cold-formed members and profiled sheets

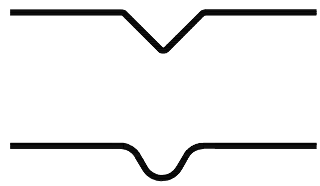
(5) Cross-sections of cold-formed members and sheets may either be unstiffened or incorporate longitudinal stiffeners in their webs or flanges, or in both.

1.5.2 Form of stiffeners

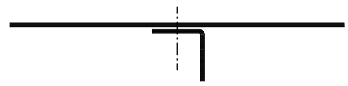
(1) Typical forms of stiffeners for cold-formed members and sheets are shown in figure 1.3.



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.

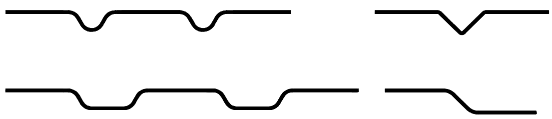
(3) Typical edge stiffeners are shown in figure 1.4.

a) Single edge fold stiffeners

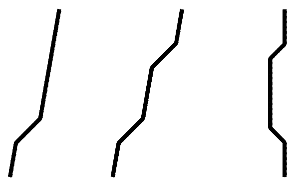
b) Double edge fold stiffeners

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 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.

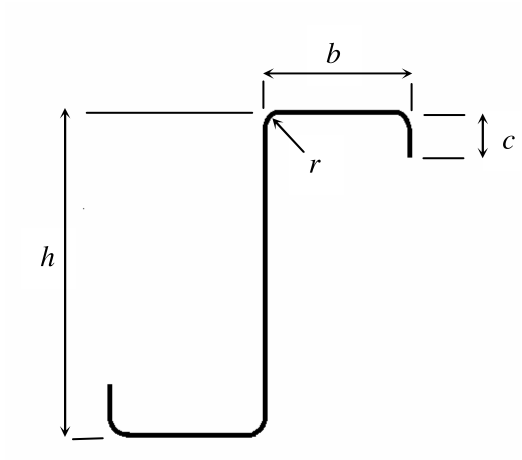


Figure 1.6: Dimensions of typical cross-section

(2) Unless stated otherwise, the other cross-sectional dimensions of cold-formed 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.

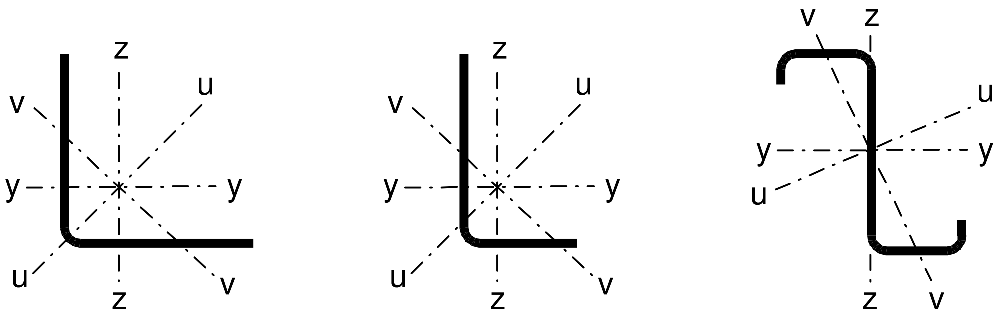


Figure 1.7: Axis convention

(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 members and sheeting 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)P Appropriate partial factors shall be adopted for ultimate limit states and serviceability limit states.

(3)P 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 fastener 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.

(5) For verifications at serviceability limit states the partial factor $\gamma_{M,ser}$ 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 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 members and sheeting are designed to contribute to the overall strength and stability of a structure;

Structural Class II: Construction where cold-formed 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 structural classes may be considered.
NOTE 2: For requirements for execution of sheeting see EN 1090.

3 Materials

3.1 General

(1) All steels used for cold-formed members and profiled sheets should be suitable for cold-forming and welding, if needed. Steels used for members and sheets to be galvanized 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.

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 4	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

NOTE 1: For steel strip less than 3 mm thick conforming to EN 10025, if the width of the original strip is greater than or equal to 600 mm, the characteristic values may be given in the National Annex. Values equal to 0,9 times those given in Table 3.1a are recommended.

NOTE 2: For other steel materials and products see the National Annex. Examples for steel grades that may conform to the requirements of this standard are given in Table 3.1b.

Table 3.1b: 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
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 10326	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 10326	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 10326	S220GD+AZ S250GD+AZ S280GD+AZ S320GD+AZ S350GD+AZ	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 10327	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.

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 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 3.1a and b

c) by appropriate tests.

(2) Where the characteristic values are determined from tests, such tests 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 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 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. In other cases the basic yield strength f_{yb} should be used. Where the yield strength is specified using the symbol f_{yb} the basic yield strength f_{yb} 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:

$$f_{ya} = f_{yb} + \left(f_u - f_{yb}\right)\frac{knt^2}{A_g} \quad \text{but} \quad f_{ya} \leq \frac{\left(f_u + f_{yb}\right)}{2} \qquad \ldots (3.1)$$

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:

$$\frac{\sum_{i=1}^{m} A_{g,i} f_{y,i}}{\sum_{i=1}^{m} A_{g,i}} \leq f_{ya} \qquad \ldots (3.2)$$

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 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
- 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

$$t = t_{cor} \quad \text{if } tol \leq 5\% \qquad \text{… (3.3a)}$$

$$t = t_{cor} = \frac{100 - tol}{95} \quad \text{if } tol > 5\% \qquad \text{… (3.3b)}$$

with $t_{cor} = t_{nom} - t_{metallic\ coatings}$ … (3.3c)

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 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 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 or ETA.

3.3.3 Welding consumables

(1) Welding consumables 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, 9.3.1 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.

NOTE 4: For hot dip galvanized fasteners see EN ISO 10684.

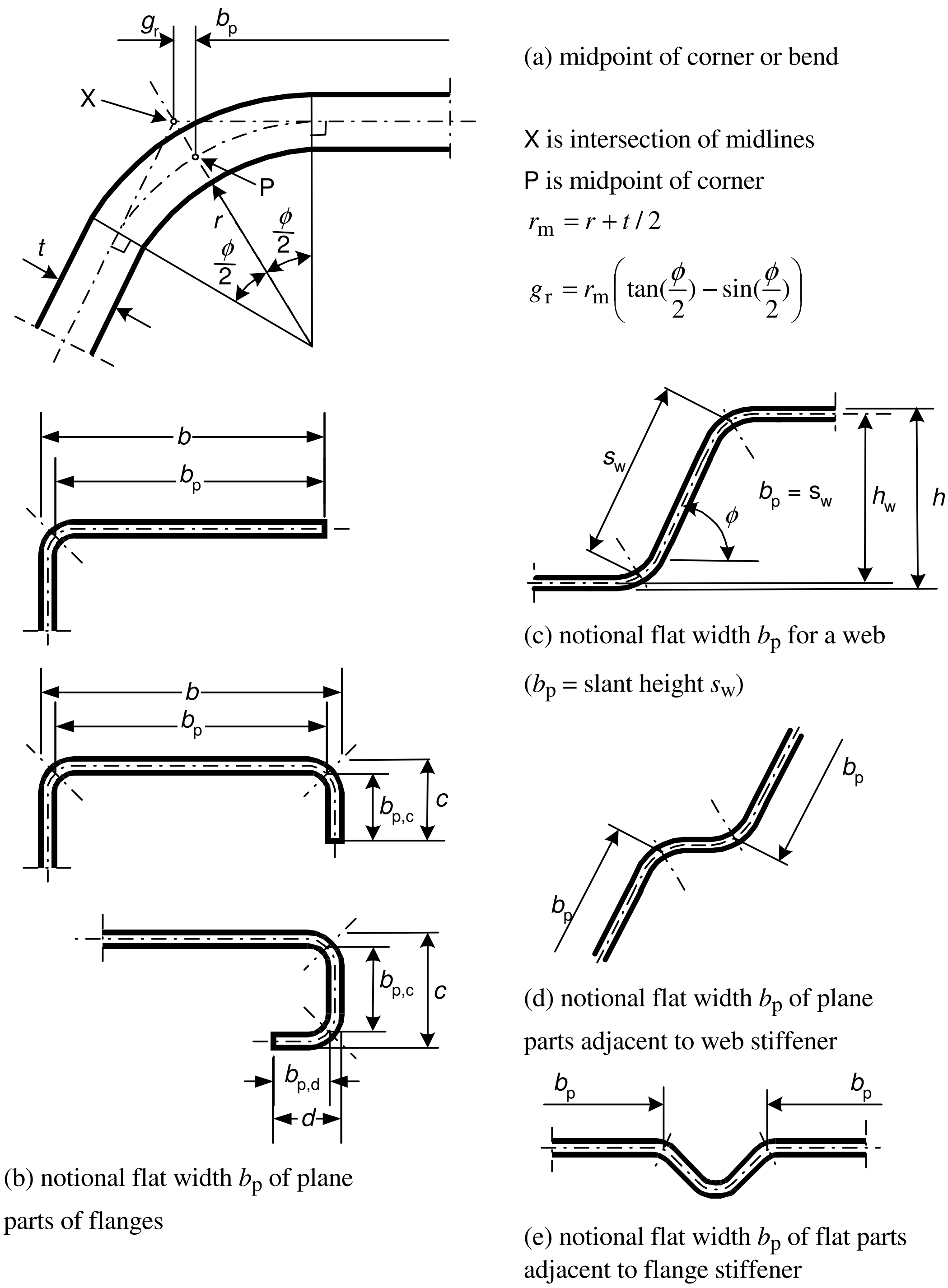
5 Structural analysis

5.1 Influence of rounded corners

(1) In cross-sections with rounded corners, the notional flat widths b_p of the plane elements should be measured from the midpoints of the adjacent corner elements as indicated in figure 5.1.

(2) In cross-sections with rounded corners, the calculation of section properties should be based upon the nominal 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 5.2, 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

X is intersection of midlines

P is midpoint of corner

$r_m = r + t/2$

$g_r = r_m \left(\tan(\frac{\phi}{2}) - \sin(\frac{\phi}{2}) \right)$

(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 5.1: 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 5.2, using the following approximations:

$$A_g \approx A_{g,sh}(1 - \delta) \quad \text{… (5.1a)}$$

$$I_g \approx I_{g,sh}(1 - 2\delta) \quad \text{… (5.1b)}$$

$$I_w \approx I_{w,sh}(1 - 4\delta) \quad \text{… (5.1c)}$$

with:

$$\delta = 0{,}43 \frac{\sum_{j=1}^{n} r_j \frac{\phi_j}{90°}}{\sum_{i=1}^{m} b_{p,i}} \qquad \ldots (5.1d)$$

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.

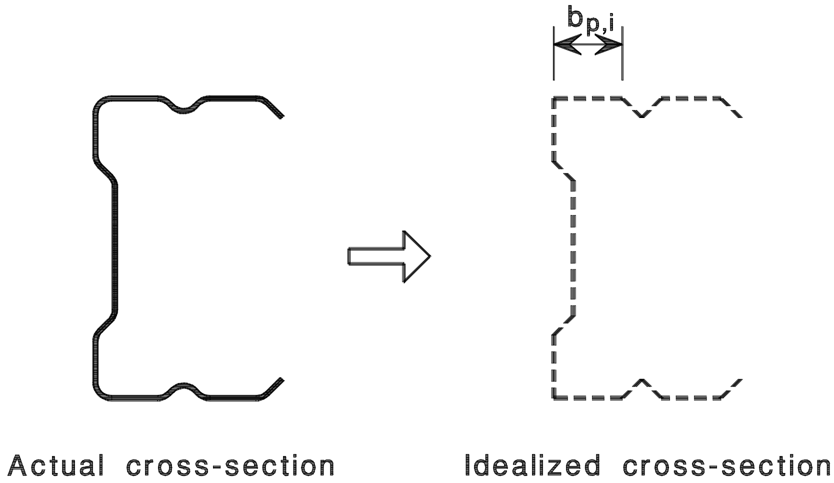


Figure 5.2: Approximate allowance for rounded corners

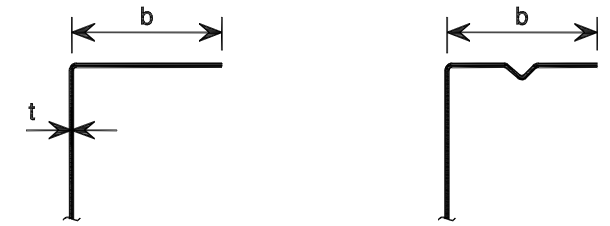
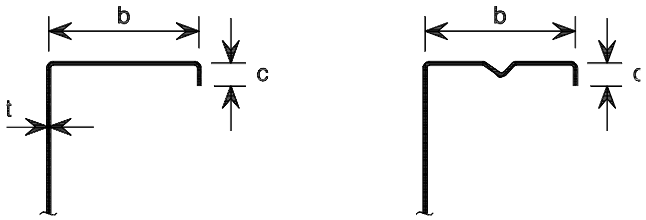
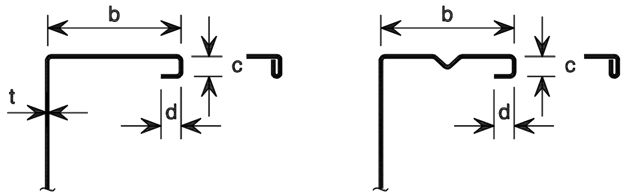
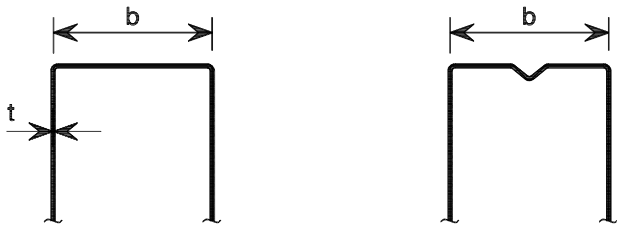
(6) Where the internal radius $r > 0{,}04\, t\, E / f_y$ then the resistance of the cross-section should be determined by tests.

5.2 Geometrical proportions

(1) The provisions for design by calculation given in this Part 1-3 of EN 1993 should not be applied to cross-sections outside the range of width-to-thickness ratios *b/t*, *h/t*, *c/t* and *d/t* given in Table 5.1.

NOTE: These limits *b/t*, *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
	$b/t \leq 50$
	$b/t \leq 60$ $c/t \leq 50$
	$b/t \leq 90$ $c/t \leq 60$ $d/t \leq 50$
	$b/t \leq 500$
	$45° \leq \phi \leq 90°$ $h/t \leq 500 \sin\phi$

(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 $c/b <$ 0,2 or $d/b <$ 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.

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 1993-1-8, 3.6.3.

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)

$$N_{c,Rd} = A_{eff} f_{yb} / \gamma_{M0} \qquad \text{...(6.2)}$$

- if the effective area A_{eff} is equal to the gross area A_g (section with no reduction due to local or distortional buckling)

$$N_{c,Rd} = A_g \left(f_{yb} + (f_{ya} - f_{yb}) 4(1 - \bar{\lambda}_e / \bar{\lambda}_{e0}) \right) / \gamma_{M0} \text{ but not more than } A_g f_{ya} / \gamma_{M0} \qquad \text{...(6.3)}$$

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};

f_{ya} is the average yield strength, see 3.2.2;

f_{yb} is the basic yield strength.;

$\bar{\lambda}_{e\,max}$ is the relative slenderness of the element which corresponds to the largest value of $\bar{\lambda}_e / \bar{\lambda}_{e0}$;

For plane elements $\bar{\lambda}_e = \bar{\lambda}_p$ and $\bar{\lambda}_{e0} = 0{,}673$, see 5.5.2;

For stiffened elements $\bar{\lambda}_e = \bar{\lambda}_d$ and $\bar{\lambda}_{e0} = 0{,}65$, see 5.5.3.

(2) The internal axial 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 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 check, then that shift should be neglected only if the shift has been calculated at yield strength and not with the actual compressive stresses.

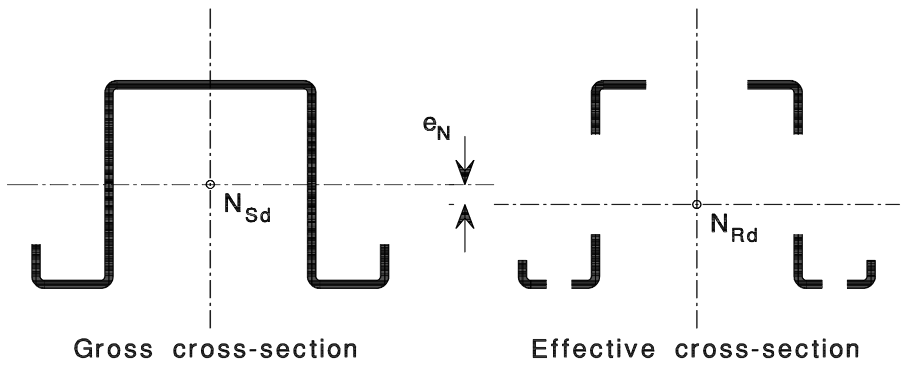


Figure 6.1: Effective cross-section under compression

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}

$$M_{c,Rd} = W_{eff} f_{yb} / \gamma_{M0} \qquad \text{...(6.4)}$$

- if the effective section modulus W_{eff} is equal to the gross elastic section modulus W_{el}

$$M_{c,Rd} = f_{yb}\left(W_{el} + (W_{pl} - W_{el})4(1 - \bar{\lambda}_{e\,max} / \bar{\lambda}_{e0})\right) / \gamma_{M0} \text{ but not more than } W_{pl} f_{yb} / \gamma_{M0} \qquad \text{...(6.5)}$$

where

$\bar{\lambda}_{e\,max}$ is the slenderness of the element which correspond to the largest value of $\bar{\lambda}_e / \bar{\lambda}_{e0}$;

For double supported plane elements $\bar{\lambda}_e = \bar{\lambda}_p$ and $\bar{\lambda}_{e0} = 0{,}5 + \sqrt{0{,}25 - 0{,}055(3+\psi)}$ where ψ is the stress ratio, see 5.5.2;

For outstand elements $\bar{\lambda}_e = \bar{\lambda}_p$ and $\bar{\lambda}_{e0} = 0{,}673$, see 5.5.2;

For stiffened elements $\bar{\lambda}_e = \bar{\lambda}_d$ and $\bar{\lambda}_{e0} = 0{,}65$, see 5.5.3.

The resulting bending moment resistance as a function of a decisive element is illustrated in the figure 6.2.

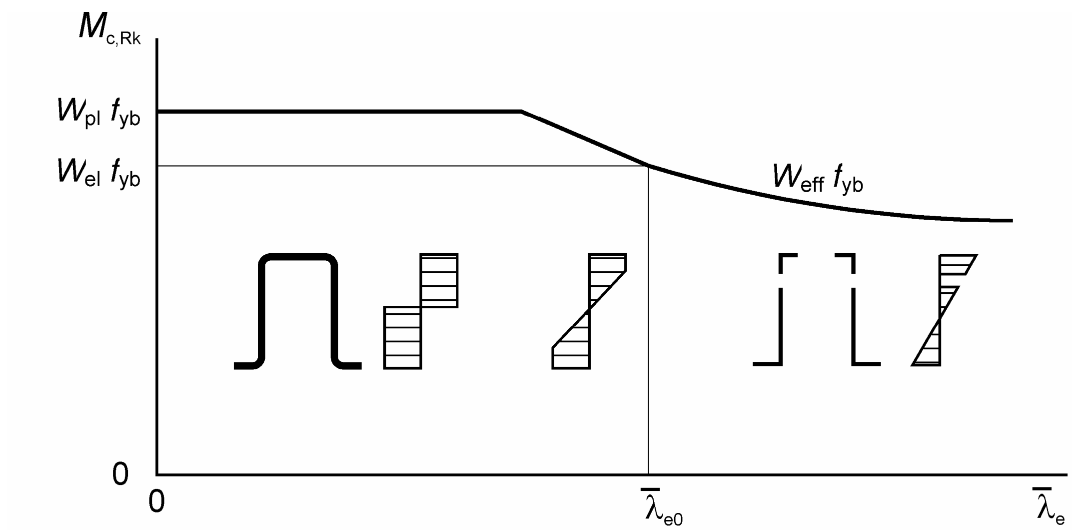


Figure 6.2: Bending moment resistance as a function of slenderness

(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:

$$M_{c,Rd} = W_{el} f_{ya} / \gamma_{M0} \qquad \text{…(6.6)}$$

(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 bending moment about the major main axis;

$M_{z,Ed}$ is the 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.

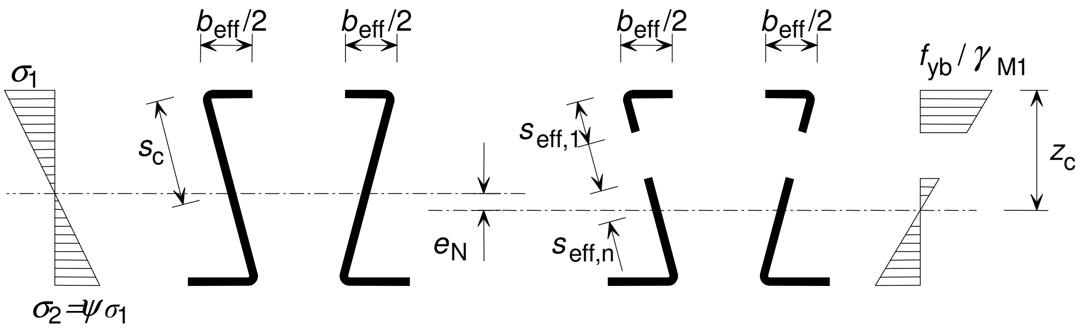


Figure 6.3: Effective cross-section for resistance to bending moments

(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 $\psi = -1$.

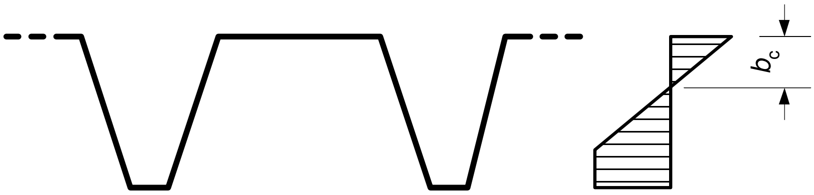


Figure 6.4: Measure b_c for determination of effective width

(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 should be taken into account according to EN 1993-1-5.

6.1.5 Shear force

(1) The design shear resistance $V_{b,Rd}$ should be determined from:

$$V_{b,Rd} = \frac{\dfrac{h_w}{\sin\phi} t f_{bv}}{\gamma_{M0}} \qquad \text{... (6.8)}$$

where:

f_{bv} is the shear strength considering buckling according to Table 6.1;

h_w is the web height between the midlines of the flanges, see figure 5.1(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)]
$\bar{\lambda}_w \leq 0,83$	$0,58 f_{yb}$	$0,58 f_{yb}$
$0,83 < \bar{\lambda}_w < 1,40$	$0,48 f_{yb} / \bar{\lambda}_w$	$0,48 f_{yb} / \bar{\lambda}_w$
$\bar{\lambda}_w \geq 1,40$	$0,67 f_{yb} / \bar{\lambda}_w^2$	$0,48 f_{yb} / \bar{\lambda}_w$

[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 $\overline{\lambda}_{\mathrm{w}}$ should be obtained from the following:

- for webs without longitudinal stiffeners:

$$\overline{\lambda}_{\mathrm{w}} = 0{,}346 \frac{s_{\mathrm{w}}}{t} \sqrt{\frac{f_{\mathrm{yb}}}{E}} \qquad \text{... (6.10a)}$$

- for webs with longitudinal stiffeners, see figure 6.5:

$$\overline{\lambda}_{\mathrm{w}} = 0{,}346 \frac{s_{\mathrm{d}}}{t} \sqrt{\frac{5{,}34}{k_{\tau}} \frac{f_{\mathrm{yb}}}{E}} \quad \text{but} \quad \overline{\lambda}_{w} \geq 0{,}346 \frac{s_{\mathrm{p}}}{t} \sqrt{\frac{f_{\mathrm{yb}}}{E}} \qquad \text{... (6.10b)}$$

with:

$$k_{\tau} = 5{,}34 + \frac{2{,}10}{t} \left(\frac{\Sigma I_{\mathrm{s}}}{\mathrm{s}_{\mathrm{d}}} \right)^{1/3}$$

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 5.1(c).

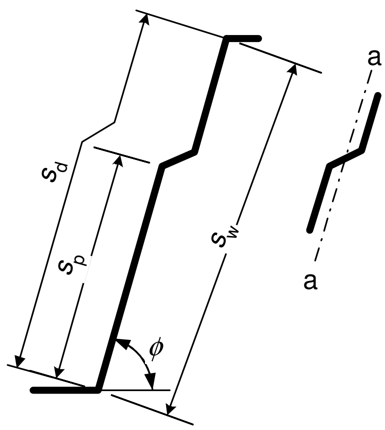


Figure 6.5: Longitudinally stiffened web

6.1.6 Torsional moment

(1) Where loads are applied eccentric to the shear centre of the cross-section, the effects of torsion 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 $M_{\mathrm{y,Ed}}$ and $M_{\mathrm{z,Ed}}$ 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 3.2.2):

$$\sigma_{\text{tot,Ed}} \leq f_{\text{ya}} / \gamma_{\text{M0}} \quad \text{... (6.11a)}$$

$$\tau_{\text{tot,Ed}} \leq \frac{f_{\text{ya}} / \sqrt{3}}{\gamma_{\text{M0}}} \quad \text{... (6.11b)}$$

$$\sqrt{\sigma_{\text{tot,Ed}}^{2} + 3\tau_{\text{tot,Ed}}^{2}} \leq 1{,}1 \frac{f_{\text{ya}}}{\gamma_{\text{M0}}} \quad \text{... (6.11c)}$$

where:

$\sigma_{\text{tot,Ed}}$ is the design total direct stress, calculated on the relevant effective cross-section;

$\tau_{\text{tot,Ed}}$ is the design total shear stress, calculated on the gross cross-section.

(5) The total direct stress $\sigma_{\text{tot,Ed}}$ and the total shear stress $\tau_{\text{tot,Ed}}$ should by obtained from:

$$\sigma_{\text{tot,Ed}} = \sigma_{\text{N,Ed}} + \sigma_{\text{My,Ed}} + \sigma_{\text{Mz,Ed}} + \sigma_{\text{w,Ed}} \quad \text{... (6.12a)}$$

$$\tau_{\text{tot,Ed}} = \tau_{\text{Vy,Ed}} + \tau_{\text{Vz,Ed}} + \tau_{\text{t,Ed}} + \tau_{\text{w,Ed}} \quad \text{... (6.12b)}$$

where:

$\sigma_{\text{My,Ed}}$ is the design direct stress due to the bending moment $M_{\text{y,Ed}}$ (using effective cross-section);

$\sigma_{\text{Mz,Ed}}$ is the design direct stress due to the bending moment $M_{\text{z,Ed}}$ (using effective cross-section);

$\sigma_{\text{N,Ed}}$ is the design direct stress due to the axial force N_{Ed} (using effective cross-section);

$\sigma_{\text{w,Ed}}$ is the design direct stress due to warping (using gross cross-section);

$\tau_{\text{Vy,Ed}}$ is the design shear stress due to the transverse shear force $V_{\text{y,Ed}}$ (using gross cross-section);

$\tau_{\text{Vz,Ed}}$ is the design shear stress due to the transverse shear force $V_{\text{z,Ed}}$ (using gross cross-section);

$\tau_{\text{t,Ed}}$ is the design shear stress due to uniform (St. Venant) torsion (using gross cross-section);

$\tau_{\text{w,Ed}}$ is the design shear stress due to warping (using gross cross-section).

6.1.7 Local transverse forces

6.1.7.1 General

(1)P 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} shall satisfy:

$$F_{\text{Ed}} \leq R_{\text{w,Rd}} \quad \text{... (6.13)}$$

where:

$R_{\text{w,Rd}}$ is the local transverse resistance of the web.

(2) The local transverse resistance of a web $R_{\text{w,Rd}}$ 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 angle of the web relative to the flanges [degrees].

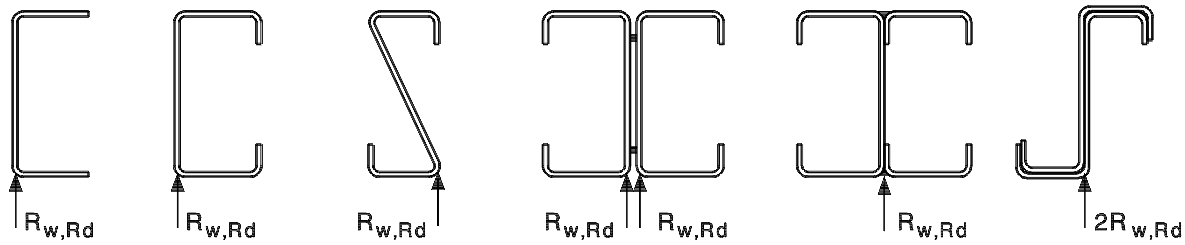


Figure 6.6: Examples of cross-sections with a single web

(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 coefficients 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 $k_2 \leq 1{,}0$

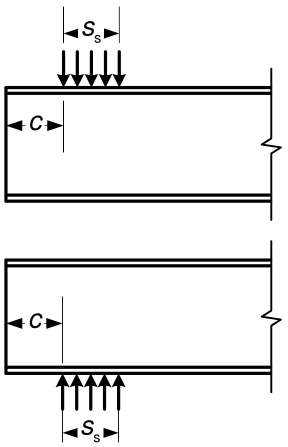
$k_3 = 0{,}7 + 0{,}3\ (\phi / 90)^2$

$k_4 = 1{,}22 - 0{,}22\ k$

$k_5 = 1{,}06 - 0{,}06\ r/t$ but $k_5 \leq 1{,}0$

where:

$k = f_{yb} / 228$ [with f_{yb} in N/mm^2].



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:

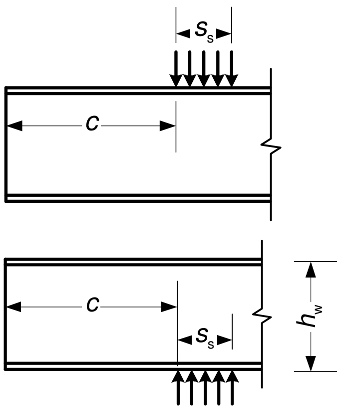
$$R_{w,Rd} = \frac{k_1 k_2 k_3 \left[9{,}04 - \frac{h_w / t}{60} \right] \left[1 + 0{,}01 \frac{s_s}{t} \right] t^2 f_{yb}}{\gamma_{M1}} \qquad (6.15a)$$

- 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 $s_s/t > 60$:

$$R_{w,Rd} = \frac{k_1 k_2 k_3 \left[5{,}92 - \frac{h_w / t}{132} \right] \left[0{,}71 + 0{,}015 \frac{s_s}{t} \right] t^2 f_{yb}}{\gamma_{M1}} \qquad (6.15c)$$



ii) $c > 1{,}5\, h_w$ clear from a free end:

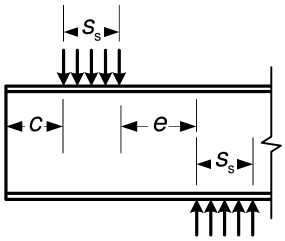
- if $s_s/t \leq 60$:

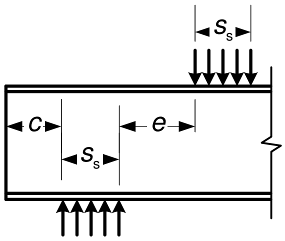
$$R_{w,Rd} = \frac{k_3 k_4 k_5 \left[14{,}7 - \frac{h_w / t}{49{,}5} \right] \left[1 + 0{,}007 \frac{s_s}{t} \right] t^2 f_{yb}}{\gamma_{M1}} \qquad (6.15d)$$

- if $s_s/t > 60$:

$$R_{w,Rd} = \frac{k_3 k_4 k_5 \left[14{,}7 - \frac{h_w / t}{49{,}5} \right] \left[0{,}75 + 0{,}011 \frac{s_s}{t} \right] t^2 f_{yb}}{\gamma_{M1}} \qquad (6.15e)$$

Figure 6.7a) : Local loads and supports — cross-sections with a single web

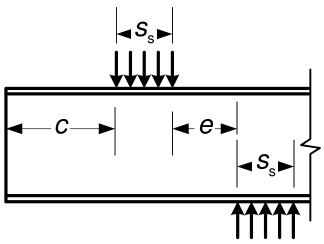


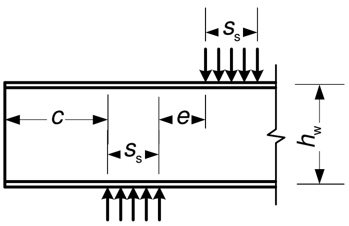


b) For two opposing local transverse forces closer together than $1{,}5\ h_w$:

i) $c\ \leq\ 1{,}5\ h_w$ clear from a free end:

$$R_{w,Rd} = \frac{k_1 k_2 k_3 \left[6{,}66 - \dfrac{h_w/t}{64}\right]\left[1 + 0{,}01\dfrac{s_s}{t}\right] t^2 f_{yb}}{\gamma_{M1}} \tag{6.15f}$$





ii) $c\ >\ 1{,}5\ h_w$ clear from a free end:

$$R_{w,Rd} = \frac{k_3 k_4 k_5 \left[21{,}0 - \dfrac{h_w/t}{16{,}3}\right]\left[1 + 0{,}0013\dfrac{s_s}{t}\right] t^2 f_{yb}}{\gamma_{M1}} \tag{6.15g}$$

Figure 6.7b): Local loads and supports — cross-sections with a single web

(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

$$R_{w,Rd} = \frac{k_7 \left[8{,}8 + 1{,}1\sqrt{\dfrac{s_s}{t}}\right] t^2 f_{yb}}{\gamma_{M1}} \qquad \ldots (6.16a)$$

ii) $c > 1{,}5\ h_w$ (far from free end)

for a cross-section of stiffened and unstiffened flanges

$$R_{w,Rd} = \frac{k_5^* k_6 \left[13{,}2 + 2{,}87\sqrt{\dfrac{s_s}{t}}\right] t^2 f_{yb}}{\gamma_{M1}} \qquad \ldots (6.16b)$$

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 coefficients k_5^* to k_{11} should be determined as follows:

$k_5^* = 1{,}49 - 0{,}53\ k$ but $k_5^* \geq 0{,}6$

$k_6 = 0{,}88 - 0{,}12\ t/1{,}9$

$k_7 = 1 + s_s / t / 750$ if $s_s / t < 150$; $k_7 = 1{,}20$ if $s_s / t > 150$

$k_8 = 1 / k$ if $s_s / t < 66{,}5$; $k_8 = (1{,}10 - s_s / t / 665) / k$ if $s_s / t > 66{,}5$

$k_9 = 0{,}82 + 0{,}15\ t/1{,}9$

$k_{10} = (0{,}98 - s_s / t / 865) / k$

$k_{11} = 0{,}64 + 0{,}31\ t/1{,}9$

where:

$k = f_{yb} / 228$ [with f_{yb} in N/mm^2];

s_s is the nominal 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.

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 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:

$r/t \leq 10$ ... (6.17a)

$h_w/t \leq 200 \sin \phi$ ... (6.17b)

$45° \leq \phi \leq 90°$ ... (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 angle of the web relative to the flanges [degrees].

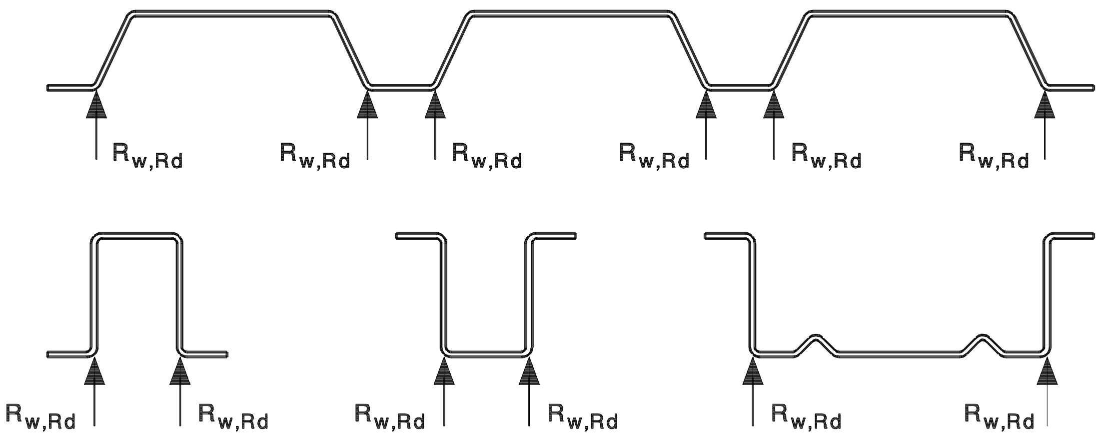


Figure 6.8: Examples of cross-sections with two or more webs

(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 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)

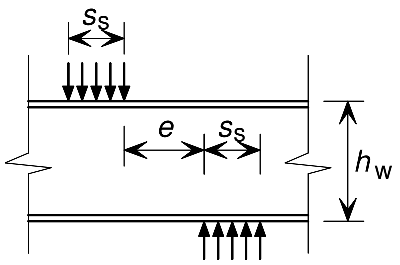
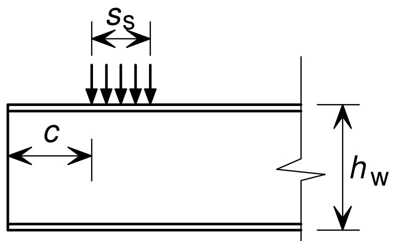
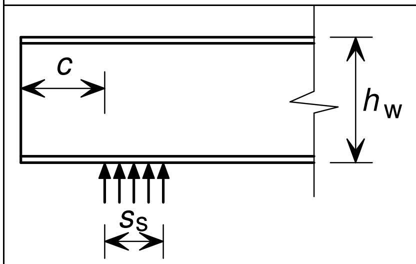
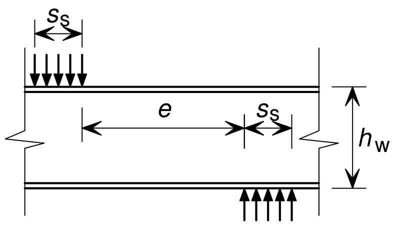
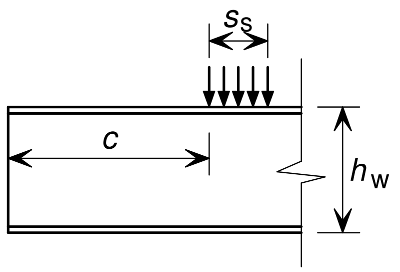
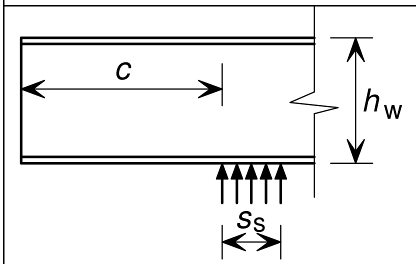
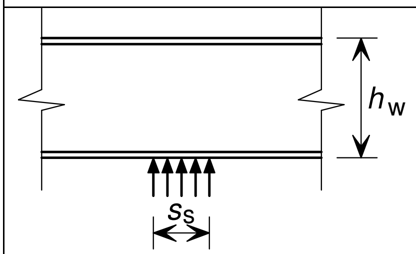
	Category 1 - local load applied with $e \leq 1{,}5\ h_w$ clear from the nearest support;
	Category 1 - local load applied with $c \leq 1{,}5\ h_w$ clear from a free end;
	Category 1 - reaction at end support with $c \leq 1{,}5\ h_w$ clear from a free end.
	Category 2 - local load applied with $e > 1{,}5\ h_w$ 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.

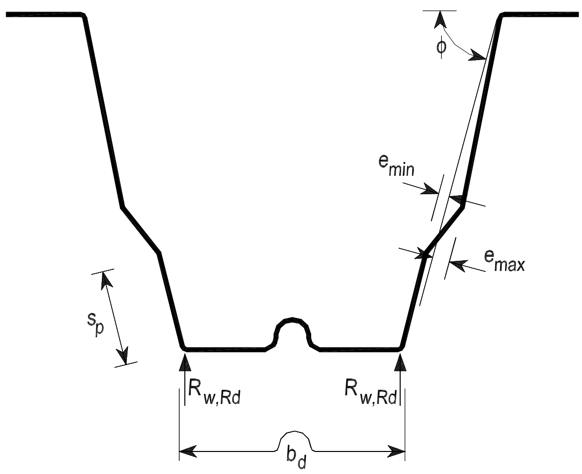


Figure 6.10: Stiffened webs

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 $M_{cy,Rd,ten} \leq {}_{cy,Rd,com}$ 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 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:

$$\frac{N_{Ed}}{N_{Rd}} + \frac{M_{y,Ed}}{M_{y,Rd}} + (1 - \frac{M_{f,Rd}}{M_{pl,Rd}})(\frac{2V_{Ed}}{V_{w,Rd}} - 1)^2 \leq 1{,}0 \qquad \text{...(6.27)}$$

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 moment of resistance of a cross-section consisting of the effective area of flanges only, see EN 1993-1-5;
$M_{pl,Rd}$	is the plastic moment of 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 (6.28c) 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 should be taken of possible lateral buckling of compression flanges and lateral bending of flanges generally.

(2) The effects of local and distortional buckling 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, 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

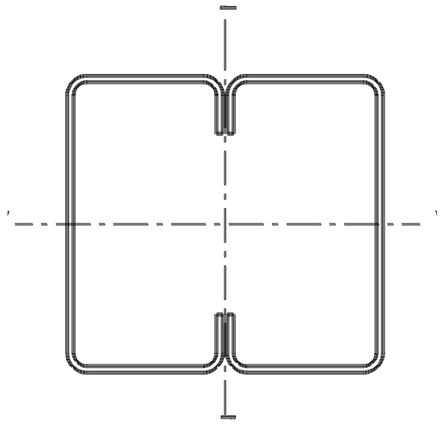
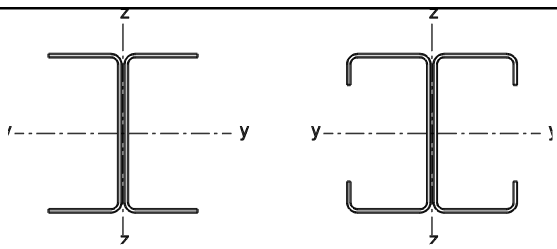
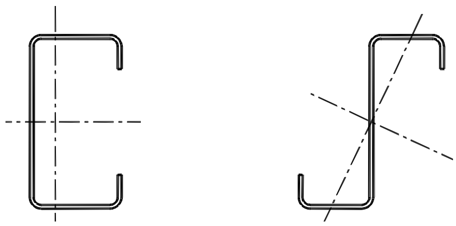
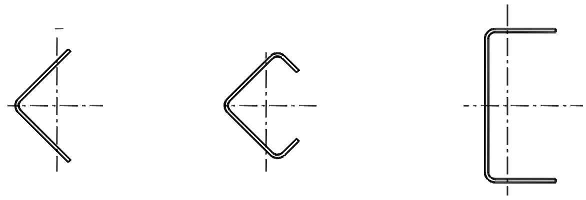
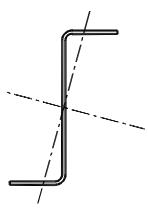
(1) For members with point-symmetric open cross-sections (e.g Z-purlin with equal flanges), account should be taken of the possibility that the resistance of the member to torsional buckling might be less than its resistance to flexural buckling.

(2) For members with mono-symmetric open cross-sections, see figure 6.12, account 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 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 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 $A_{eff} = A_g$

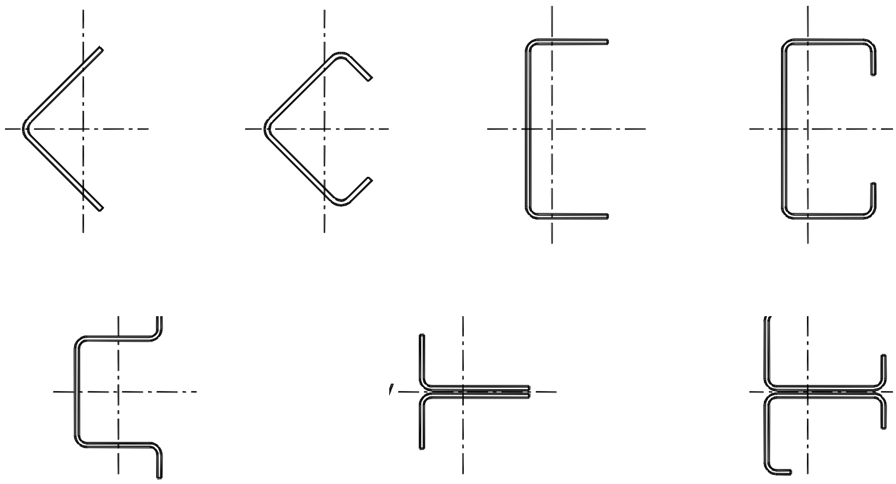


Figure 6.12: Monosymmetric cross-sections susceptible to torsional-flexural buckling

(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. $y_o = z_o = 0$), the elastic critical force $N_{cr,TF}$ for torsional-flexural buckling should be determined from:

$$N_{cr,TF} = N_{cr,T} \qquad \text{...(6.34)}$$

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:

$$N_{cr,TF} = \frac{N_{cr,y}}{2\beta}\left[1 + \frac{N_{cr,T}}{N_{cr,y}} - \sqrt{(1 - \frac{N_{cr,T}}{N_{cr,y}})^2 + 4(\frac{y_o}{i_o})^2 \frac{N_{cr,T}}{N_{cr,y}}}\right] \quad \text{... (6.35)}$$

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).

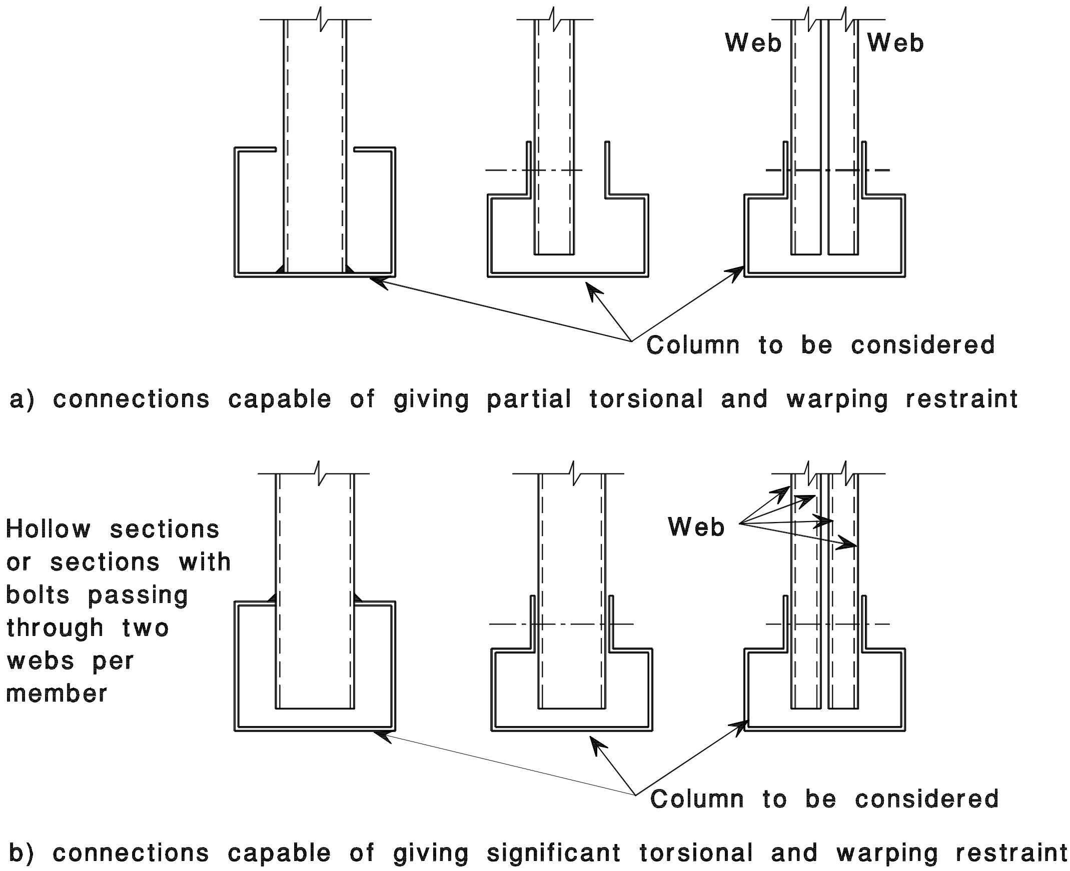


Figure 6.13: Torsional and warping restraint from practical connections

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 6.3.2.2 using the lateral buckling curve b.

(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.

Bracing systems that supply positional restraint to more than one member should be designed to resist the sum of the restraint forces from each member that they restrain, reduced by the factor k_r obtained from:

$$k_r = (0.2 + 1/N_r)^{0.5}$$

in which N_r is the number of parallel members restrained.

4.7.2 Slenderness

The slenderness λ of a compression member should generally be taken as its effective length L_E divided by its radius of gyration r about the relevant axis, except as given in **4.7.9**, **4.7.10** or **4.7.13**.

In the case of a single-angle strut with lateral restraints to its two legs alternately, the slenderness for buckling about every axis should be increased by 20 %.

4.7.3 Effective lengths

Except for angles, channels or T-sections designed in accordance with **4.7.10** the effective length L_E of a compression member should be determined from the segment length L centre-to-centre of restraints or intersections with restraining members in the relevant plane as follows.

a) Generally, in accordance with Table 22, depending on the conditions of restraint in the relevant plane, members carrying more than 90 % of their reduced plastic moment capacity M_r in the presence of axial force (see **I.2**) being taken as incapable of providing directional restraint.

b) For continuous columns in multistorey buildings of simple design, in accordance with Table 22, depending on the conditions of restraint in the relevant plane, directional restraint being based on connection stiffness as well as member stiffness.

c) For compression members in trusses, lattice girders or bracing systems, in accordance with Table 22, depending on the conditions of restraint in the relevant plane.

d) For columns in single storey buildings of simple design, see **D.1**.

e) For columns supporting internal platform floors of simple design, see **D.2**.

f) For columns forming part of a continuous structure, see Annex E.

Table 22 — Nominal effective length L_E for a compression member[a]

<table>
<tr><td colspan="4">a) non-sway mode</td></tr>
<tr><td colspan="3">Restraint (in the plane under consideration) by other parts of the structure</td><td>L_E</td></tr>
<tr><td rowspan="4">Effectively held in position at both ends</td><td colspan="2">Effectively restrained in direction at both ends</td><td>0.7L</td></tr>
<tr><td colspan="2">Partially restrained in direction at both ends</td><td>0.85L</td></tr>
<tr><td colspan="2">Restrained in direction at one end</td><td>0.85L</td></tr>
<tr><td colspan="2">Not restrained in direction at either end</td><td>1.0L</td></tr>
<tr><td colspan="4">b) sway mode</td></tr>
<tr><td>One end</td><td colspan="2">Other end</td><td>L_E</td></tr>
<tr><td rowspan="3">Effectively held in position and restrained in direction</td><td rowspan="3">Not held in position</td><td>Effectively restrained in direction</td><td>1.2L</td></tr>
<tr><td>Partially restrained in direction</td><td>1.5L</td></tr>
<tr><td>Not restrained in direction</td><td>2.0L</td></tr>
<tr><td colspan="4">[a] Excluding angle, channel or T-section struts designed in accordance with 4.7.10.</td></tr>
</table>

Table 3 — Coefficients of friction μ

	Wood	Steel	Concrete
Wood	0,4	0,4	0,6
Steel	0,4	0,1	0,2
Concrete	0,6	0,2	0,5
Clay[a]	0,25	0,2	0,25
Loam[a]	0,4	0,2	0,4
Sand and gravel	0,65	0,2	0,65

[a] At least of stiff consistency in accordance with ENV 1997-1.

It shall be borne in mind that loosening by vibration may occur in the case of supports subjected to vibrating stress.

If stability is not obtained by static friction alone, then the structure shall be anchored in the ground. In such cases, the safety against sliding shall be calculated in conjunction with the action of soil anchors. Under these conditions, the coefficients of friction in accordance with Table 3 shall only be entered in the calculation at 70 % of the listed values.

$$\sum \gamma \, \overline{\mu} \, N_{\mathrm{k}} + Z_{\mathrm{h,d}} \geq \sum \gamma \times H_{\mathrm{k}} \tag{55}$$

$$\overline{\mu} = 0{,}7 \, \mu \tag{56}$$

where

$Z_{\mathrm{h,d}}$ is the horizontal design capacity of the anchor (see 5.5.2);

μ is the coefficient of friction in accordance with Table 3.

5.5.1.4 Safety against lifting shall be calculated from:

$$\sum \gamma \, N_{\mathrm{St,\,k}} \geq \sum \gamma \, N_{\mathrm{a,\,k}} \tag{57}$$

where

γ is the safety factor in accordance with Table 2;

$N_{\mathrm{St,\,k}}$ are the vertical stabilizing load components;

$N_{\mathrm{a,\,k}}$ are the vertical lifting load components.

With anchor ties the following relationship shall be applied:

$$\sum \gamma \, N_{\mathrm{St,\,k}} + Z_{v,\,d} \geq \sum \gamma \, N_{\mathrm{a,\,k}} \tag{58}$$

where

$Z_{\mathrm{v,d}}$ is the vertical design capacity of the anchor (see 5.5.2).

Table 6.1 - Categories of use

Category	Specific Use	Example
A	Areas for domestic and residential activities	Rooms in residential buildings and houses; bedrooms and wards in hospitals; bedrooms in hotels and hostels kitchens and toilets.
B	Office areas	
C	Areas where people may congregate (with the exception of areas defined under category A, B, and D[1)])	**C1**: Areas with tables, etc. e.g. areas in schools, cafés, restaurants, dining halls, reading rooms, receptions. **C2**: Areas with fixed seats, e.g. areas in churches, theatres or cinemas, conference rooms, lecture halls, assembly halls, waiting rooms, railway waiting rooms. **C3:** Areas without obstacles for moving people, e.g. areas in museums, exhibition rooms, etc. and access areas in public and administration buildings, hotels, hospitals, railway station forecourts. **C4**: Areas with possible physical activities, e.g. dance halls, gymnastic rooms, stages. **C5**: Areas susceptible to large crowds, e.g. in buildings for public events like concert halls, sports halls including stands, terraces and access areas and railway platforms.
D	Shopping areas	**D1:** Areas in general retail shops **D2**: Areas in department stores
[1)] Attention is drawn to 6.3.1.1(2), in particular for C4 and C5. See EN 1990 when dynamic effects need to be considered. For Category E, see Table 6.3		
NOTE 1 Depending on their anticipated uses, areas likely to be categorised as C2, C3, C4 may be categorised as C5 by decision of the client and/or National annex. NOTE 2 The National annex may provide sub categories to A, B, C1 to C5, D1 and D2 NOTE 3 See 6.3.2 for storage or industrial activity		

6.3.1.2 Values of actions

(1)P The categories of loaded areas, as specified in Table 6.1, shall be designed by using characteristic values q_k (uniformly distributed load) and Q_k (concentrated load).

NOTE Values for q_k and Q_k are given in Table 6.2 below. Where a range is given in this table, the value may be set by the National annex. The recommended values, intended for separate application, are underlined. q_k is intended for determination of general effects and Q_k for local effects. The National annex may define different conditions of use of this Table.

Table 6.2 - Imposed loads on floors, balconies and stairs in buildings

Categories of loaded areas	q_k [kN/m^2]	Q_k [kN]
Category A		
- Floors	1,5 to2,0	2,0 to 3,0
- Stairs	2,0 to4,0	2,0 to 4,0
- Balconies	2,5 to 4,0	2,0 to 3,0
Category B	2,0 to 3,0	1,5 to 4,5
Category C		
- C1	2,0 to 3,0	3,0 to 4,0
- C2	3,0 to 4,0	2,5 to 7,0 (4,0)
- C3	3,0 to 5,0	4,0 to 7,0
- C4	4,5 to 5,0	3,5 to 7,0
- C5	5,0 to 7,5	3,5 to 4,5
category D		
- D1	4,0 to 5,0	3,5 to 7,0 (4,0)
- D2	4,0 to 5,0	3,5 to 7,0

(2) Where necessary q_k and Q_k should be increased in the design (e.g. for stairs and balconies depending on the occupancy and on dimensions).

(3) For local verifications a concentrated load Q_k acting alone should be taken into account.

(4) For concentrated loads from storage racks or from lifting equipment, Q_k should be determined for the individual case, see 6.3.2.

(5)P The concentrated load shall be considered to act at any point on the floor, balcony or stairs over an area with a shape which is appropriate to the use and form of the floor.

NOTE The shape may normally be assumed as a square with a width of 50 mm. See also 6.3.4.2(4)

(6)P The vertical loads on floors due to traffic of forklifts shall be taken into account according to 6.3.2.3.

(7)P Where floors are subjected to multiple use, they shall be designed for the most unfavourable category of loading which produces the highest effects of actions (e.g. forces or deflection) in the member under consideration.

(8) Provided that a floor allows a lateral distribution of loads, the self-weight of movable partitions may be taken into account by a uniformly distributed load q_k which should be added to the imposed loads of floors obtained from Table 6.2. This defined uniformly distributed load is dependent on the self-weight of the partitions as follows:
– for movable partitions with a self-weight ≤ 1,0 kN/m wall length: q_k =0,5 kN/m^2;

Category of spectator activity	Horizontal load (percentage of vertical live load)
Category 1 Nominal potential for spectator movement, which excludes synchronised and periodic crowd movement, such as at: • Lectures/exhibitions • Displays/shows • Minor athletic events • Golf tournaments • Agricultural shows • Military tournaments	6%
Category 2 Potential for spectator movement more vigorous than Category 1. For instance as at: • Major athletics meetings where music accompanies the individual events • Major musical concerts • Rugby or football matches	7.5%
Category 3 Stands with a potential for synchronised and periodic crowd movement and having vertical and horizontal fundamental frequencies which avoid resonance effects, e.g. at most pop concerts where strong musical beats are expected.	10%
Notes **a** Partial factors for dead and imposed loads for use in the limit state design should correspond to the structural Code of Practice relevant to the material (steel, aluminium, etc.). For notional horizontal loads, the partial factor should be 1.5 for the load combination case with factored values of vertical dead and imposed loads. **b** The notional horizontal load should be combined with the operational wind load for designing the structural elements, but not in the design against overturning as a result of wind action (Section 7.6). **c** Synchronised and periodic crowd movements may include dancing, jumping, rhythmic stamping, etc.	

of 5kN/m^2, and a simultaneous notional load commensurate with the use applied in any one horizontal direction at the stage surface. This value should be a minimum of 5% of the design vertical imposed load (or such higher value taken from Table 7.2 as required by the anticipated activity). This notional load should be applied to the area of stage floor on which the activity takes place. Other parts of the stage floor, including wings and extension platforms used for workers and equipment, should use a value of 2.5% of the vertical load for the notional horizontal load. The values of the design's vertical and horizontal loads used for each area should be clearly defined in the engineering documents and shown on relevant drawings.

However, assuming that the self-weight of the rostra and stage sets does not exceed 2.5kN/m^2, the design vertical static equivalent load on rostra and stage sets should generally be 2.5kN/m^2, unless defined otherwise in the engineering documentation. Appropriate risk assessments should be prepared for the rostra and stage sets, and the operating limits clearly defined in the engineering documentation.

Stage floor surfaces should be constructed in such a way as to remove any tripping or undesirable slipping hazards. It is recommended that stages should be designed to carry a point load of 3.6kN over an area 50 × 50mm without causing any damage to the floor and without causing excessive deflection of the floor panels (e.g. deflection of more than 10mm relative to the adjoining panels).

If moving objects are attached to the structure, an allowance for the dynamic movement of the object should be considered. Unless specific measures are put in place (such as 'soft-start' circuits or other techniques), the allowance should be a 25% increase for electrically-operated equipment and 10% for hand-operated equipment as appropriate.

The quoted values are characteristic values. All roofs, floors and spectator decks should be designed to carry the uniformly distributed load derived using appropriate load factors.

Vertical imposed crowd loads should be taken into account as quasi-static actions. It may also be necessary to consider the dynamic effect of imposed loads, especially for long-span structures and grandstands. It has been demonstrated that groups of people dancing and jumping in unison can generate forces up to 3.5 times their own weight. This means that people jumping in unison can potentially generate forces well in excess of the design load for proprietary stage decks. As such, the use of such decks for a suspended dance floor is not recommended[7.3,7.4].

The design of temporary structures to resist earthquakes is not usually required in areas of low seismicity. However, if the Client and/or Enforcing Authority require the structure to be subject to seismic design, the complexity of the design is much higher. Reference should be made to National Standards which may include BS EN 1998-1[7.5], ASCE/SEI 7[7.6], ASCE/SEI 37[7.7] and ANSI E1.21[7.8] or other appropriate and equivalent Standards, as well as relevant guidance documents — including the Institution's *Manual for the seismic design of steel and concrete buildings to Eurocode 8*[7.9].

E.6.3.3 Nominal values of γ_M based on past experience

The values of γ_M given in Table E.9 shall be used.

NOTE The material safety factors given in Table E.9 are examples of values that may be obtained for a product with relatively consistent properties such as continuously laminated PUR or PIR. They may be unsafe for products with less consistent properties.

Table E.9 – Material safety factors γ_M

Property to which γ_M applies	Limit state	
	Ultimate limit state	Serviceability limit state
Yielding of a metal face	1,1	1,0
Wrinkling of a metal face in the span ($v \leq 0{,}09$)	1,25	1,1
Wrinkling of a metal face at an intermediate support (interaction with support reaction)	1,25 [a]	1,1
Shear of the core ($v \leq 0{,}16$)	1,5	1,1
Shear failure of a profiled face	1,1	1,0
Crushing of the core ($v \leq 0{,}13$)	1,4	1,1
Support reaction capacity of a profiled face	1,1	1,0
Failure of a fastener	1,33 [b]	1,0 [b]
Failure of an element at a point of connection	1,33 [b]	1,0 [b]

[a] The material factor for wrinkling at the ultimate limit state is needed if the design is based on elastic analysis or if a non-zero bending resistance at intermediate supports is utilised in a design based on plastic analysis.

[b] If the characteristic value of the strength of a fastening is not based on a sufficient number of tests for a statistically reliable value to be obtained, higher values of the material safety factors shall be used.

E.7 Calculation of the effects of actions

E.7.1 General

In the determination of the internal stress resultants and deflections, the shear flexibility of the core shall be taken into account. For this purpose, a constant value of the shear modulus of the core, corresponding to an average value at normal indoor temperature, shall be used. The stress resultants shall then be determined using the methods described in E.7.2.

E.7.2 Methods of analysis

E.7.2.1 General

One or other of the following methods of analysis shall be used:

— elastic analysis;

— plastic analysis.

Elastic analysis shall be used for the serviceability limit state and may be used for the ultimate limit state.

Plastic analysis shall only be used for the ultimate limit state and shall be used whenever the design is controlled by bending stresses at an internal support. Plastic analysis shall not be used when the first failure mode is a shear failure of the core, unless the core material has adequate plastic shear capacity.

γ_M is the global partial factor for the particular resistance.

NOTE For a more general formula for the design resistance, see EN 1990.

(2) The partial factors γ_M should be applied to members and joints.

NOTE 1 The values of γ_{Mi} are given in Table 4.1(NDP) unless the National Annex gives different values.

Table 4.1(NDP) — Partial factors for resistance for wrought aluminium alloys

Type of structure and failure	**Partial factor**	**Recommended value**
Resistance of cross-sections and resistance of members to instability (based on f_o):	γ_{M1}	1,1
Resistance of cross-sections to fracture and resistance of joints in tension, shear and bearing (based on f_u or f_w):	γ_{M2} γ_{Mw}	1,25 1,25
Resistance of pin connections for ultimate limit state: Resistance of pin connections for serviceability limit state:	γ_{Mp} $\gamma_{Mp,ser}$	1,25 1,0
Slip resistance of connections for ultimate limit state: Slip resistance of connections for serviceability limit state:	γ_{Ms} $\gamma_{Ms,ser}$	1,4 1,0
Resistance of adhesive bonded connections	γ_{Ma}	≥ 3,0
Resistance in case of fire and fatigue	See EN 1999-1-2 and EN 1999-1-3	

NOTE 2 The values of γ_{M1} and γ_{M2} are also applicable for cold-formed aluminium structural sheeting and shells, see EN 1999-1-4 and EN 1999-1-5 respectively.

4.5 Design assisted by testing

(1) Values for resistances R_k or R_d may be determined using design assisted by testing methods.

NOTE 1 Guidance on design assisted by testing is given in Annex D of prEN 1990:2020.

NOTE 2 The National Annex can give rules for testing.

4.6 Execution requirements

(1) A specification for execution of the work shall be prepared in order to include all necessary technical information to carry out the work. This information should include execution class(es), whether any non-normative tolerances in EN 1090-3 should apply, complete geometrical information and information on materials to be used in members and joints, types and sizes of fasteners, weld requirements and requirements for execution of work. EN 1090-3 contains a checklist for information to be provided.

5 Materials

5.1 General

(1) For design verification, the material properties given in this section should be considered as characteristic values. They are based on the nominal values given in the relevant product standard.

Table 5.8 — Characteristic values of 0,2% proof strength, f_o, and ultimate tensile strength, f_u, for cast aluminium alloys – Gravity castings[c]

Alloy	Casting process	Temp-er	f_o (f_{oc}) N/mm²	f_u (f_{uc}) N/mm²	A_{50} %[a, b]
EN AC-42100	Permanent mould	T6	147	203	2,0
	Permanent mould	T64	126	175	4
EN AC-42200	Permanent mould	T6	168	224	1,5
	Permanent mould	T64	147	203	3
EN AC-43000	Permanent mould	F	63	126	1,25
EN AC-43300	Permanent mould	T6	147	203	2,0
	Sand cast	T6	133	161	1,0
	Permanent mould	T64	126	175	3
EN AC-44200	Permanent mould	F	56	119	3
	Sand cast	F	49	105	2,5
EN AC-51300	Permanent mould	F	70	126	2,0
	Sand cast	F	63	112	1,5

[a] For elongation requirements for the design of cast components, see E.3.2(1).

[b] The values are 70 % of the values of EN 1706:2010, which should be used only for separately casted test specimens (see 6.3.3.2 of EN 1706:2010).

[c] The values for A50 are 50 % of the elongation values of EN 1706:2010, which should be used only for separately casted test specimens (see 7.3.3.2 of EN 1706:2010).

5.2.4 Dimensions, mass and tolerances

(1) The dimensions and tolerances of structural extruded products, sheet and plate products, drawn tubes, wires and forgings should conform to the ENs listed in 2.3.2.

(2) The dimensions and tolerances of structural cast products should conform to the ENs listed in 2.3.3.

5.2.5 Design values of material constants

(1) The material constants to be adopted in calculations for the aluminium alloys covered by this document should be taken as follows:

— modulus of elasticity $E = 70\ 000$ N/mm²;

— shear modulus $G = 27\ 000$ N/mm²;

— Poisson's ratio $\nu = 0{,}3$;

— coefficient of linear thermal expansion $\alpha = 23 \times 10^{-6}$ per °C;

— unit mass $\rho = 2\ 700$ kg/m³.

(2) For material properties in structures subject to elevated temperatures associated with fire EN 1999-1-2 should be used.

2.3 Basic variables

2.3.1 Actions and environmental influences

2.3.1.1 General

(1) Actions to be used in design may be obtained from the relevant parts of EN 1991.

Note 1: The relevant parts of EN 1991 for use in design include:
EN 1991-1-1 Densities, self-weight and imposed loads
EN 1991-1-3 Snow loads
EN 1991-1-4 Wind actions
EN 1991-1-5 Thermal actions
EN 1991-1-6 Actions during execution
EN 1991-1-7 Accidental actions

(2)P Duration of load and moisture content affect the strength and stiffness properties of timber and wood-based elements and shall be taken into account in the design for mechanical resistance and serviceability.

(3)P Actions caused by the effects of moisture content changes in the timber shall be taken into account.

2.3.1.2 Load-duration classes

(1)P The load-duration classes are characterised by the effect of a constant load acting for a certain period of time in the life of the structure. For a variable action the appropriate class shall be determined on the basis of an estimate of the typical variation of the load with time.

(2)P Actions shall be assigned to one of the load-duration classes given in Table 2.1 for strength and stiffness calculations.

Table 2.1 – Load-duration classes

Load-duration class	Order of accumulated duration of characteristic load
Permanent	more than 10 years
Long-term	6 months – 10 years
Medium-term	1 week – 6 months
Short-term	less than one week
Instantaneous	

NOTE: Examples of load-duration assignment are given in Table 2.2. Since climatic loads (snow, wind) vary between countries, the assignment of load-duration classes may be specified in the National annex.

Table 2.2 – Examples of load-duration assignment

Load-duration class	Examples of loading
Permanent	self-weight
Long-term	storage
Medium-term	imposed floor load, snow
Short-term	snow, wind
Instantaneous	wind, accidental load

2.3.1.3 Service classes

(1)P Structures shall be assigned to one of the service classes given below:

NOTE 1: The service class system is mainly aimed at assigning strength values and for calculating deformations under defined environmental conditions.

NOTE 2: Information on the assignment of structures to service classes given in (2)P, (3)P and (4)P may be given in the National annex.

(2)P Service class 1 is characterised by a moisture content in the materials corresponding to a temperature of 20°C and the relative humidity of the surrounding air only exceeding 65 % for a few weeks per year.

NOTE: In service class 1 the average moisture content in most softwoods will not exceed 12 %.

(3)P Service class 2 is characterised by a moisture content in the materials corresponding to a temperature of 20°C and the relative humidity of the surrounding air only exceeding 85 % for a few weeks per year.

NOTE: In service class 2 the average moisture content in most softwoods will not exceed 20 %.

(4)P Service class 3 is characterised by climatic conditions leading to higher moisture contents than in service class 2.

2.3.2 Materials and product properties

2.3.2.1 Load-duration and moisture influences on strength

(1) Modification factors for the influence of load-duration and moisture content on strength, see 2.4.1, are given in 3.1.3.

(2) Where a connection is constituted of two timber elements having different time-dependent behaviour, the calculation of the design load-carrying capacity should be made with the following modification factor k_{mod}:

$$k_{\mathrm{mod}} = \sqrt{k_{\mathrm{mod},1}\,k_{\mathrm{mod},2}} \tag{2.6}$$

where $k_{\mathrm{mod},1}$ and $k_{\mathrm{mod},2}$ are the modification factors for the two timber elements.

2.3.2.2 Load-duration and moisture influences on deformations

(1) For serviceability limit states, if the structure consists of members or components having different time-dependent properties, the final mean value of modulus of elasticity, $E_{\mathrm{mean,fin}}$, shear modulus $G_{\mathrm{mean,fin}}$, and slip modulus, $K_{\mathrm{ser,fin}}$, which are used to calculate the final deformation should be taken from the following expressions:

2.4 Verification by the partial factor method

2.4.1 Design value of material property

(1)P The design value X_d of a strength property shall be calculated as:

$$X_d = k_{mod} \frac{X_k}{\gamma_M} \tag{2.14}$$

where:

X_k is the characteristic value of a strength property;

γ_M is the partial factor for a material property;

k_{mod} is a modification factor taking into account the effect of the duration of load and moisture content.

NOTE 1: Values of k_{mod} are given in 3.1.3.

NOTE 2: The recommended partial factors for material properties (γ_M) are given in Table 2.3. Information on the National choice may be found in the National annex.

Table 2.3 – Recommended partial factors γ_M for material properties and resistances

Fundamental combinations:	
Solid timber	1,3
Glued laminated timber	1,25
LVL, plywood, OSB,	1,2
Particleboards	1,3
Fibreboards, hard	1,3
Fibreboards, medium	1,3
Fibreboards, MDF	1,3
Fibreboards, soft	1,3
Connections	1,3
Punched metal plate fasteners	1,25
Accidental combinations	1,0

(2)P The design member stiffness property E_d or G_d shall be calculated as:

$$E_d = \frac{E_{mean}}{\gamma_M} \tag{2.15}$$

$$G_d = \frac{G_{mean}}{\gamma_M} \tag{2.16}$$

where:

E_{mean} is the mean value of modulus of elasticity;

G_{mean} is the mean value of shear modulus.

Section 3 Material properties

3.1 General

3.1.1 Strength and stiffness parameters

(1)P Strength and stiffness parameters shall be determined on the basis of tests for the types of action effects to which the material will be subjected in the structure, or on the basis of comparisons with similar timber species and grades or wood-based materials, or on well-established relations between the different properties.

3.1.2 Stress-strain relations

(1)P Since the characteristic values are determined on the assumption of a linear relation between stress and strain until failure, the strength verification of individual members shall also be based on such a linear relation.

(2) For members or parts of members subjected to compression, a non-linear relationship (elastic-plastic) may be used.

3.1.3 Strength modification factors for service classes and load-duration classes

(1) The values of the modification factor k_{mod} given in Table 3.1 should be used.

(2) If a load combination consists of actions belonging to different load-duration classes a value of k_{mod} should be chosen which corresponds to the action with the shortest duration, e.g. for a combination of dead load and a short-term load, a value of k_{mod} corresponding to the short-term load should be used.

3.1.4 Deformation modification factors for service classes

(1) The values of the deformation factors k_{def} given in Table 3.2 should be used.

3.2 Solid timber

A1⟩ (1)P Timber members shall comply with EN 14081-1.

NOTE: Strength classes for timber are given in EN 338. ⟨A1

(2) The effect of member size on strength may be taken into account.

(3) For rectangular solid timber with a characteristic timber density $\rho_{\mathrm{k}} \leq 700$ kg/m^3, the reference depth in bending or width (maximum cross-sectional dimension) in tension is 150 mm. For depths in bending or widths in tension of solid timber less than 150 mm the characteristic values for $f_{\mathrm{m,k}}$ and $f_{\mathrm{t,0,k}}$ may be increased by the factor k_{h}, given by:

$$k_{\mathrm{h}} = \min \begin{cases} \left(\dfrac{150}{h}\right)^{0,2} \\ 1,3 \end{cases} \tag{3.1}$$

where h is the depth for bending members or width for tension members, in mm.

[A1) Table 3.1 – Values of k_{mod}

Material	Standard	Service class	Load-duration class				
			Permanent action	Long term action	Medium term action	Short term action	Instantaneous action
Solid timber	EN 14081-1	1	0,60	0,70	0,80	0,90	1,10
		2	0,60	0,70	0,80	0,90	1,10
		3	0,50	0,55	0,65	0,70	0,90
Glued laminated timber	EN 14080	1	0,60	0,70	0,80	0,90	1,10
		2	0,60	0,70	0,80	0,90	1,10
		3	0,50	0,55	0,65	0,70	0,90
LVL	EN 14374, EN 14279	1	0,60	0,70	0,80	0,90	1,10
		2	0,60	0,70	0,80	0,90	1,10
		3	0,50	0,55	0,65	0,70	0,90
Plywood	EN 636						
	Type EN 636-1	1	0,60	0,70	0,80	0,90	1,10
	Type EN 636-2	2	0,60	0,70	0,80	0,90	1,10
	Type EN 636-3	3	0,50	0,55	0,65	0,70	0,90
OSB	EN 300						
	OSB/2	1	0,30	0,45	0,65	0,85	1,10
	OSB/3, OSB/4	1	0,40	0,50	0,70	0,90	1,10
	OSB/3, OSB/4	2	0,30	0,40	0,55	0,70	0,90
Particle-board	EN 312						
	Type P4, Type P5	1	0,30	0,45	0,65	0,85	1,10
	Type P5	2	0,20	0,30	0,45	0,60	0,80
	Type P6, Type P7	1	0,40	0,50	0,70	0,90	1,10
	Type P7	2	0,30	0,40	0,55	0,70	0,90
Fibreboard, hard	EN 622-2						
	HB.LA, HB.HLA 1 or 2	1	0,30	0,45	0,65	0,85	1,10
	HB.HLA1 or 2	2	0,20	0,30	0,45	0,60	0,80
Fibreboard, medium	EN 622-3						
	MBH.LA1 or 2	1	0,20	0,40	0,60	0,80	1,10
	MBH.HLS1 or 2	1	0,20	0,40	0,60	0,80	1,10
	MBH.HLS1 or 2	2	–	–	–	0,45	0,80
Fibreboard, MDF	EN 622-5						
	MDF.LA, MDF.HLS	1	0,20	0,40	0,60	0,80	1,10
	MDF.HLS	2	–	–	–	0,45	0,80

(A1]

(4) For timber which is installed at or near its fibre saturation point, and which is likely to dry out under load, the values of k_{def}, given in Table 3.2, should be increased by 1,0.

(5)P Finger joints shall comply with EN 385.

3.3 Glued laminated timber

(1)P Glued laminated timber members shall comply with EN 14080.

NOTE: In EN 1194 values of strength and stiffness properties are given for glued laminated timber allocated to strength classes, see annex D (Informative).

(2) The effect of member size on strength may be taken into account.

(3) For rectangular glued laminated timber, the reference depth in bending or width in tension is 600 mm. For depths in bending or widths in tension of glued laminated timber less than 600 mm

the characteristic values for $f_{\mathrm{m,k}}$ and $f_{\mathrm{t,0,k}}$ may be increased by the factor k_{h}, given by

$$k_{\mathrm{h}} = \min \begin{cases} \left(\dfrac{600}{h} \right)^{0,1} \\ 1{,}1 \end{cases} \tag{3.2}$$

where h is the depth for bending members or width for tensile members, in mm.

A1⟩ (4)P Large finger joints complying with the requirements of EN 387 shall not be used for products to be installed in service class 3, where the direction of grain changes at the joint. ⟨A1

(5)P The effect of member size on the tensile strength perpendicular to the grain shall be taken into account.

A1⟩ Table 3.2 – Values of k_{def} for timber and wood-based materials

Material	Standard	Service class 1	Service class 2	Service class 3
Solid timber	EN 14081-1	0,60	0,80	2,00
Glued Laminated timber	EN 14080	0,60	0,80	2,00
LVL	EN 14374, EN 14279	0,60	0,80	2,00
Plywood	EN 636			
	Type EN 636-1	0,80	–	–
	Type EN 636-2	0,80	1,00	–
	Type EN 636-3	0,80	1,00	2,50
OSB	EN 300			
	OSB/2	2,25	–	–
	OSB/3, OSB/4	1,50	2,25	–
Particleboard	EN 312			
	Type P4	2,25	–	–
	Type P5	2,25	3,00	–
	Type P6	1,50	–	–
	Type P7	1,50	2,25	–
Fibreboard, hard	EN 622-2			
	HB.LA	2,25	–	–
	HB.HLA1, HB.HLA2	2,25	3,00	–
Fibreboard, medium	EN 622-3			
	MBH.LA1, MBH.LA2	3,00	–	–
	MBH.HLS1, MBH.HLS2	3,00	4,00	–
Fibreboard, MDF	EN 622-5			
	MDF.LA	2,25	–	–
	MDF.HLS	2,25	3,00	–

⟨A1

3.4 Laminated veneer lumber (LVL)

(1)P LVL structural members shall comply with EN 14374.

(2)P For rectangular LVL with the grain of all veneers running essentially in one direction, the effect of member size on bending and tensile strength shall be taken into account.

(3) The reference depth in bending is 300 mm. For depths in bending not equal to 300 mm the characteristic value for $f_{\mathrm{m,k}}$ should be multiplied by the factor k_{h}, given by

$$k_{\mathrm{h}} = \min\begin{cases}\left(\dfrac{300}{h}\right)^{s} \\ 1{,}2\end{cases} \tag{3.3}$$

where:

h is the depth of the member, in mm;

s is the size effect exponent, refer to 3.4(5)P.

(4) The reference length in tension is 3000 mm. For lengths in tension not equal to 3000 mm the characteristic value for $f_{t,0,k}$ should be multiplied by the factor k_{ℓ} given by

$$k_{\ell} = \min\begin{cases}\left(\dfrac{3000}{\ell}\right)^{s/2} \\ 1{,}1\end{cases} \tag{3.4}$$

where ℓ is the length, in mm.

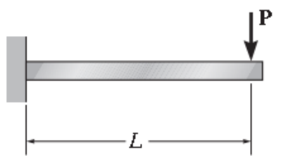
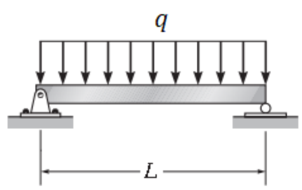
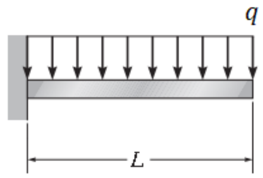
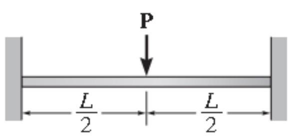
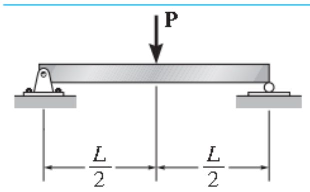
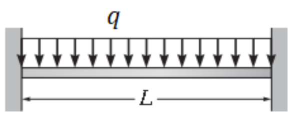
(5)P The size effect exponent s for LVL shall be taken as declared in accordance with EN 14374.

A1) (6)P Large finger joints complying with the requirements of EN 387 shall not be used for products to be installed in service class 3, where the direction of grain changes at the joint. (A1

(7)P For LVL with the grain of all veneers running essentially in one direction, the effect of member size on the tensile strength perpendicular to the grain shall be taken into account.

Deflection:

$$\delta = \frac{PL^3}{B_1 D_{eq}} + \frac{PL}{B_2 G_c A_c}$$

Mode of loading	B_1	B_2	Mode of loading	B_1	B_2
	3	1		$\frac{384}{5}$	8
	8	2		192	4
	48	4	 $P = qL$	384	8