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Descriptors:

English version

Eurocode 3 : Design of steel structures

Part 1.8 : Design of joints

Calcul des structures en acier	Bemessung und Konstruktion von Stahlbauten
Partie 1.8 :	Teil 1.8 :
Calcul des assemblages	Bemessung von Anschlüssen

Stage 34 draft

Amendments sent prior to the CEN TC 250 / SC 3 meeting in Vienna are highlighted by gray background color.

Amendments made during to the CEN TC 250 / SC 3 meeting in Vienna are highlighted by *gray background color and italic font*.

CEN

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Foreword

This European Standard EN1993-1-8, Design of Steel Structures : Design of joints, has been prepared on behalf of Technical Committee CEN/TC250/SC3 "Eurocode3", the Secretariat of which is held by BSI. CEN/TC250/SC3 is responsible for Eurocode 3.

The text of the draft standard was submitted to the formal vote and was approved by CEN as EN1993-1-8 on YYYY-MM-DD.

No existing European Standard is superseded.

National Annex for EN1993-1-8

This standard has been drafted on the assumption that it will be complemented by a National Annex to enable it to be used for the design of joints to be constructed in the relevant country.

National choice is allowed in EN 1993-1-8 through clauses

- 2.2(2)
- 2.8 (Group 6: Rivets)
- *3.4.2(3)*
- *6.2.5.2(9).*

1 Introduction

1.1 Scope

(1) This part presents rules for design of joints.

(2) This part contains design methods for joints subject to predominantly static loading.

(3) This part applies to steel grades S 235, S 275, S 355, S 420 and S 460.

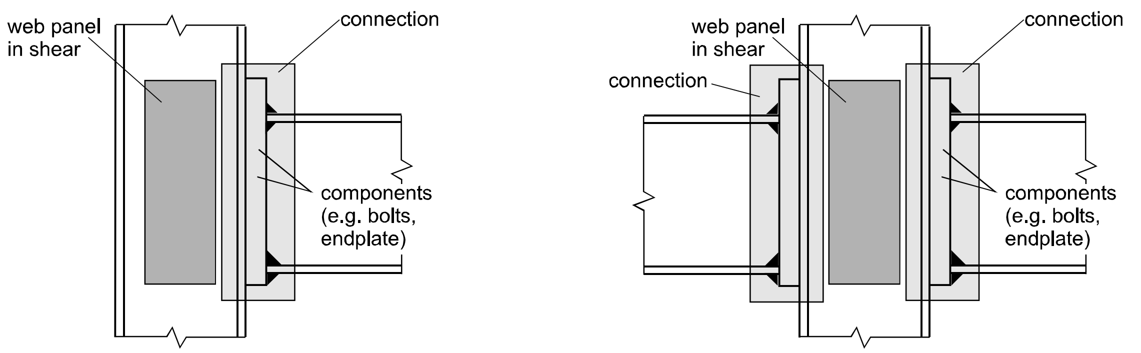
1.2 Distinction between Principles and Application Rules

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

1.3 Definitions

(1) The following definitions apply:

- **basic component** (of a joint): Specific part of a joint that makes an identified contribution to one or more of its structural properties.
- **connection**: Location at which two members are interconnected, and assembly of connection elements and - in case of a major axis joint - the load introduction into the column web panel.
- **connected member**: Member that is supported by the member to which it is connected.
- **joint**: Assembly of basic components that enables members to be connected together in such a way that the relevant internal forces and moments can be transferred between them. A beam-to-column joint consists of a web panel and either one connection (single sided joint configuration) or two connections (double sided joint configuration), see figure 1.1 .
- **joint configuration**: Type or layout of the joint or joints in a zone within which the axes of two or more inter-connected members intersect, see figure 1.2 .
- **structural properties** (of a joint): Its resistance to internal forces and moments in the connected members, its rotational stiffness and its rotation capacity.
- **uniplanar joint**: In a lattice structure an uniplanar joint connects members that are situated in a single plane.



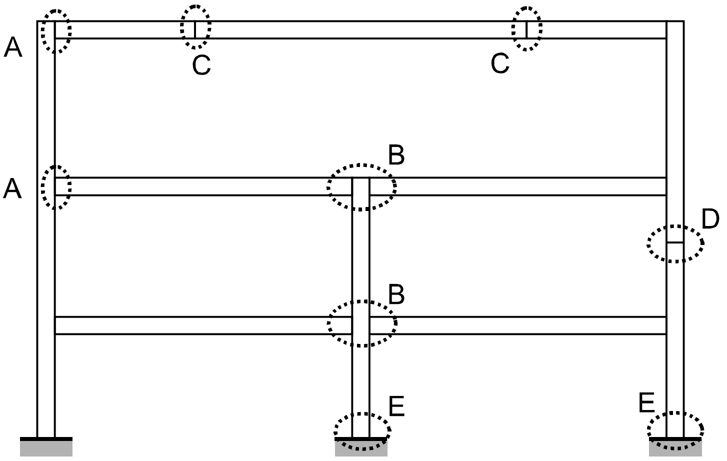
Joint = web panel in shear + connection

Left joint = web panel in shear + left connection
Right joint = web panel in shear + right connection

a) Single-sided joint configuration

b) Double-sided joint configuration

Figure 1.1: Parts of a beam-to-column joint configuration



A Single-sided beam-to-column joint configuration;

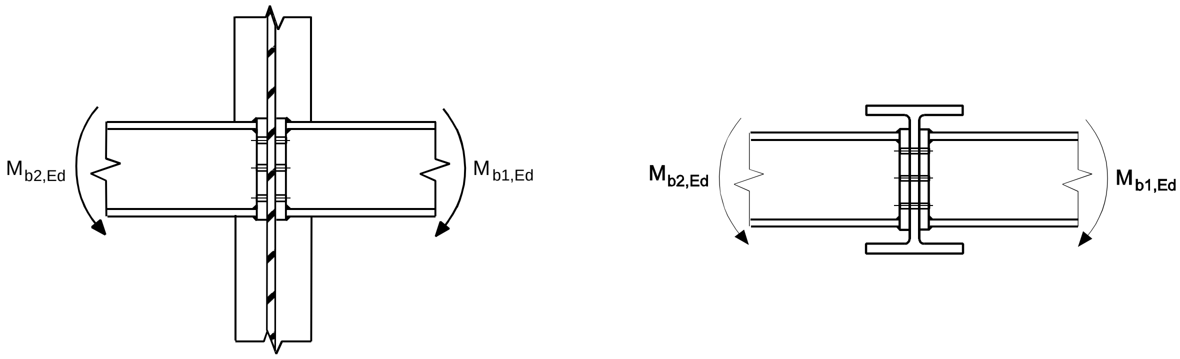
B Double-sided beam-to-column joint configuration;

C Beam splice;

D Column splice;

E Column base.

a) Major-axis joint configurations



Double-sided beam-to-column joint configuration

Double-sided beam-to-beam joint configuration

b) Minor-axis joint configurations (to be used only for balanced moments $M_{b1,Ed} = M_{b2,Ed}$)

Figure 1.2: Joint configurations

1.4 Symbols

(1) In this EN the following symbols are used:

- d is the nominal bolt diameter, the diameter of the pin or the diameter of the fastener;
- d_0 is the hole diameter for a bolt or a rivet or the diameter of the pin hole;
- $d_{o,t}$ is the hole size for the tension face, generally the hole diameter, but for horizontally slotted holes the slot length should be used;
- $d_{o,v}$ is the hole size for the shear face, generally the hole diameter, but for vertically slotted holes the slot length should be used;
- d_c is the clear depth of the column web;
- d_m is the mean of a across points and across flats dimensions of the bolt head or the nut, whichever is smaller;
- $f_{H,Rd}$ is the design value of the Hertz pressure;
- f_{ur} is the specified ultimate tensile strength of the rivet;
- e_1 is the end distance from the centre of a fastener hole to the adjacent end of any part, measured in the direction of load transfer;

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

e_3 is the distance from the axis of a slotted hole to the adjacent end or edge of any part;

e_4 is the distance from the centre of the end radius of a slotted hole to the adjacent end or edge of any part;

ℓ_{eff} is the effective length of fillet weld;

n is the number of the friction surfaces, the number of fastener holes on the shear face;

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

$p_{1,0}$ is the spacing between centres of fasteners in an outer line in the direction of load transfer;

$p_{1,i}$ is the spacing between centres of fasteners in an inner line in the direction of load transfer;

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

r is the bolt row number;

NOTE: In a bolted connection with more than one bolt-row in tension, the bolt-rows are numbered starting from the bolt-row farthest from the centre of compression.

s_s is the length of stiff bearing.

t_a is the thickness of the angle cleat.

t_{fc} is the thickness of the column flange;

t_p is the thickness of the plate under the bolt or the nut;

t_w is the thickness of the web or bracket;

t_{wc} is the thickness of the column web;

A is the gross cross-section area of bolt;

A_0 is the area if the rivet hole;

A_{vc} is the shear area of the column, see EN 1993-1-1;

A_s is the tensile stress area of the bolt;

$A_{v,eff}$ is the effective shear area;

$B_{p,Rd}$ is the design punching shear resistance of the bolt head and the nut

E is the elastic modulus;

$F_{p,Cd}$ is the design preload force;

$F_{t,Ed}$ is design tensile force per bolt for the ultimate limit state;

$F_{t,Rd}$ is design tension resistance per bolt;

$F_{T,Rd}$ is the tension resistance of an equivalent T-stub flange;

$F_{v,Rd}$ is design shear resistance per bolt;

$F_{b,Rd}$ is design bearing resistance per bolt;

$F_{s,Rd,ser}$ is design slip resistance per bolt at the serviceability limit state;

$F_{s,Rd}$ is design slip resistance per bolt at the ultimate limit state;

$F_{v,Ed,ser}$ is design shear force per bolt for serviceability limit state;

$F_{v,Ed}$ is design shear force per bolt for the ultimate limit state;

$M_{j,Rd}$ is the design moment resistance of a joint;

S_j is the rotational stiffness of a joint;

$S_{j,ini}$ is the initial rotational stiffness of a joint;

$V_{wp,Rd}$ is the plastic shear resistance of a column web panel;

z is the lever arm;

μ is the slip factor;

ϕ is the rotation of a joint.

(2) The following standard abbreviations are used in section 7:

CHS for "circular hollow section";

RHS for "rectangular hollow section", which in this context includes square hollow sections.

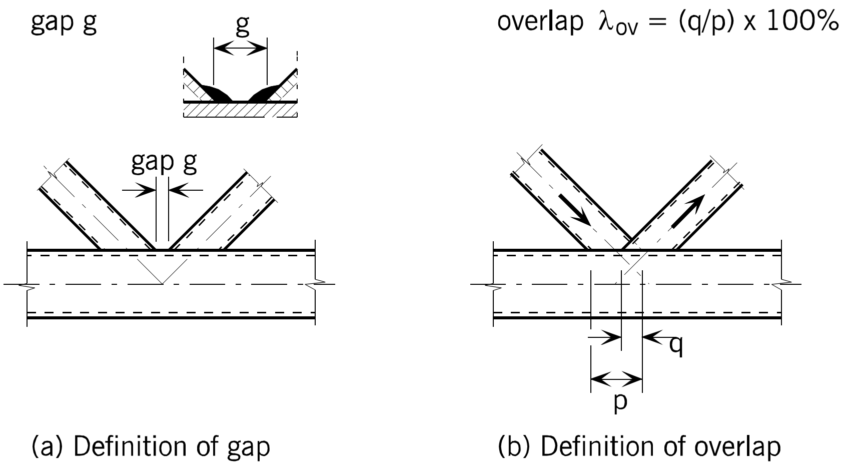


Figure 1.3: Gap and overlap joints.

(3) In section 7 the following symbols are used:

A_i is the cross-sectional area of member i (i = 0, 1, 2 or 3);

A_v is the shear area of the chord;

$A_{v,eff}$ is the effective shear area of the chord;

E is the elastic modulus of steel;

L is the system length of a member;

$M_{ip,i,Rd}$ is the design value of the resistance of the joint, expressed in terms of the in-plane internal moment in member i (i = 0, 1, 2 or 3);

$M_{ip,i,Ed}$ is the design value of the in-plane internal moment in member i (i = 0, 1, 2 or 3);

$M_{op,i,Rd}$ is the design value of the resistance of the joint, expressed in terms of the out-of-plane internal moment in member i (i = 0, 1, 2 or 3);

$M_{op,i,Ed}$ is the design value of the out-of-plane internal moment in member i (i = 0, 1, 2 or 3);

$N_{i,Rd}$ is the design value of the resistance of the joint, expressed in terms of the internal axial force in member i (i = 0, 1, 2 or 3);

$N_{i,Ed}$ is the design value of the internal axial force in member i (i = 0, 1, 2 or 3);

$W_{e\ell,i}$ is the elastic section modulus of member i (i = 0, 1, 2 or 3);

$W_{p\ell,i}$ is the plastic section modulus of member i (i = 0, 1, 2 or 3);

b_i is the overall out-of-plane width of RHS member i (i = 0, 1, 2 or 3);

b_{eff} is the effective width for a brace member to chord connection;

$b_{e,ov}$ is the effective width for an overlapping brace to overlapped brace connection;

$b_{e,p}$ is the effective width for punching shear;

b_p is the width of a plate;

b_w is the effective width for the web of the chord;

d_i is the overall diameter of CHS member i (i = 0, 1, 2 or 3);

d_w is the depth of the web of an I or H section chord member;

e is the eccentricity of a joint;

f_b is the buckling strength of the chord side wall;

f_{yi} is the yield strength of member i (i = 0, 1, 2 or 3);

f_{y0} is the yield strength of a chord member;

g is the gap between the brace members in a K or N joint (negative values of g represent an overlap q); the gap g is measured along the length of the connecting face of the chord, between the toes of the adjacent brace members, see figure 1.3(a);

h_i is the overall in-plane depth of the cross-section of member i (i = 0, 1, 2 or 3);

k is a factor defined in the relevant table, with subscript g, m, n or p;

ℓ is the buckling length of a member;

p is the length of the projected contact area of the overlapping brace member onto the face of the chord, in the absence of the overlapped brace member;

q is the length of overlap, measured at the face of the chord, between the brace members in a K or N joint;

r is the root radius of an I or H section or the corner radius of a rectangular hollow section;

t_f is the flange thickness of an I or H section;

t_i is the wall thickness of member i (i = 0, 1, 2 or 3);

t_p is the thickness of a plate;

t_w is the web thickness of an I or H section;

α is a factor defined in the relevant table;

θ_i is the included angle between brace member i and the chord (i = 1, 2 or 3);

κ is a factor defined where it occurs;

μ is a factor defined in the relevant table;

φ is the angle between the planes in a multiplanar joint.

(4) The integer subscripts used in section 7 are defined as follows:

i is an integer subscript used to designate a member of a joint, i = 0 denoting a chord and i = 1, 2 or 3 the brace members. In joints with two brace members, i = 1 normally denotes the compression brace and i = 2 the tension brace, see figure 1.4(b). For a single brace i = 1 whether it is subject to compression or tension, see figure 1.4(a);

i and j are integer subscripts used in overlap type joints, i to denote the overlapping brace member and j to denote the overlapped brace member, see figure 1.4(c).

(5) The stress ratios used in section 7 are defined as follows:

n is the ratio $(\sigma_{0,Ed}/f_{y0})/\gamma_{M5}$ (used for RHS chords);

n_p is the ratio $(\sigma_{p,Ed}/f_{y0})/\gamma_{M5}$ (used for CHS chords);

$\sigma_{0,Ed}$ is the maximum compressive stress in the chord at a joint;

$\sigma_{p,Ed}$ is the value of $\sigma_{0,Ed}$ excluding the stress due to the horizontal components of the forces in the braces at that joint, see figure 1.4.

(6) The geometric ratios used in section 7 are defined as follows:

β is the ratio of the mean diameter or width of the brace members, to that of the chord:

- for T, Y and X joints:

$$\frac{d_1}{d_0} \;;\; \frac{d_1}{b_0} \text{ or } \frac{b_1}{b_0}$$

- for K and N joints:

$$\frac{d_1 + d_2}{2\,d_0} \;;\; \frac{d_1 + d_2}{2\,b_0} \text{ or } \frac{b_1 + b_2 + h_1 + h_2}{4\,b_0}$$

- for KT joints:

$$\frac{d_1 + d_2 + d_3}{3\,d_0} \;;\; \frac{d_1 + d_2 + d_3}{3\,b_0} \text{ or } \frac{b_1 + b_2 + b_3 + h_1 + h_2 + h_3}{6\,b_0}$$

β_p is the ratio b_i/b_p;

γ is the ratio of the chord width or diameter to twice its wall thickness:

$$\frac{d_0}{2\,t_0} \;;\; \frac{b_0}{2\,t_0} \text{ or } \frac{b_0}{2\,t_f}$$

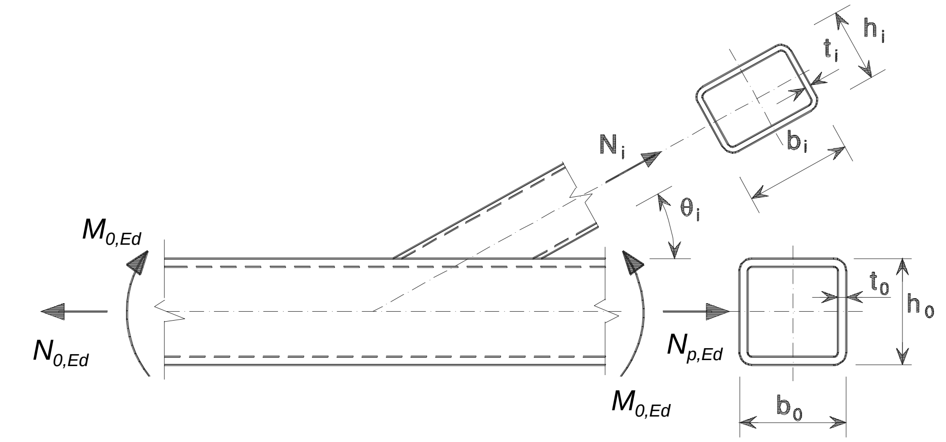
η is the ratio of the brace member depth to the chord diameter or width:

$$\frac{h_i}{d_o} \text{ or } \frac{h_i}{b_o}$$

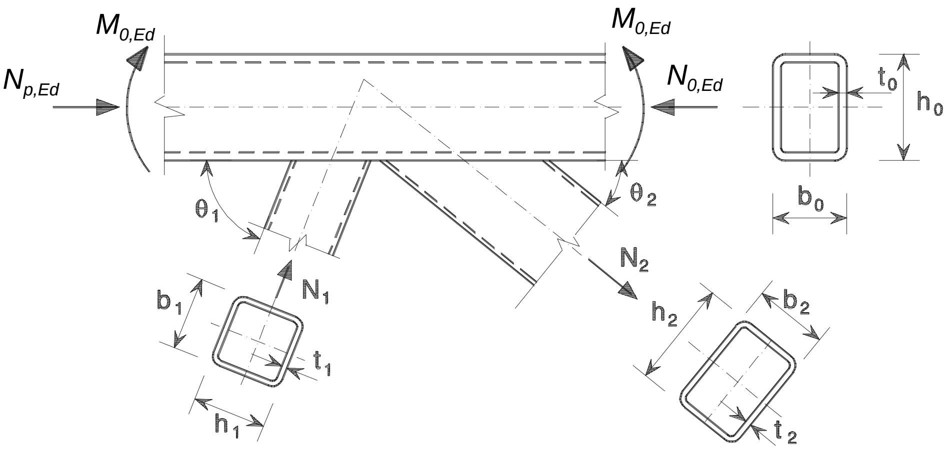
η_p is the ratio h_i/b_p;

λ_{ov} is the overlap ratio, expressed as a percentage (λ_{ov} = (q/p) x 100%) as shown in figure 1.3(b).

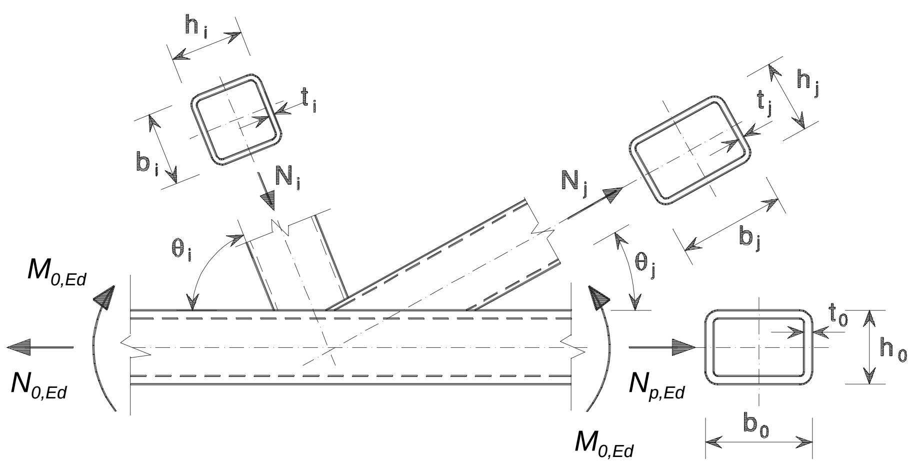
(7) Other symbols are specified in appropriate clause when they are used.



a) Joint with single brace member



b) Gap joint with two brace members



c) Overlap joint with two brace members

NOTE: Symbols for circular sections, see table 7.2.

Figure 1.4: Dimensions and other parameters at a hollow section lattice girder joint

2 Basis of Design

2.1 Assumptions

(1) The following assumptions apply:

- Design procedures are valid if the execution is made according to execution standards (see 2.8).

  NOTE: Execution standards refers to a number of requirements which need to be specified during the design.

- Only construction materials and products are used that are specified in this EN or in the relevant material or product specifications.

2.2 General requirements

(1)P All joints shall have a design resistance such that the structure is capable of satisfying all the basic design requirements given in EN 1993-1-1 and in this part.

(2) The partial safety factor γ_M for joints are given in table 2.1.

Table 2.1: Partial safety factors for joints

Resistance of members and cross-sections	γ_{M0} , γ_{M1} and γ_{M2} see EN 1993-1-1
Resistance of bolts Resistance of rivets Resistance of pins Resistance of welds Resistance of plates in bearing	γ_{M2}
Slip resistance - for hybrid connections or connections under fatigue loading - for other design situations	 γ_{M3} γ_{M3}
Bearing resistance of an injection bolt	γ_{M4}
Resistance of joints in hollow section lattice girder	γ_{M5}
Resistance of pins at serviceability limit state	$\gamma_{M6,ser}$
Preload of high strength bolts	γ_{M7}
Resistance on concrete	γ_c see EN 1992

NOTE: Numerical values for γ_M may be defined in the National Annex. Recommended values are as follows: $\gamma_{M2} = 1,25$; $\gamma_{M3} = 1,25$ for hybrid connections or connections under fatigue loading and $\gamma_{M3} = 1,1$ for other design situations; $\gamma_{M4} = 1,0$; $\gamma_{M5} = 1,0$; $\gamma_{M6,ser} = 1,0$; $\gamma_{M7} = 1,1$.

(3) Joints subject to fatigue should also satisfy the principles given in EN 1993-1-9.

2.3 Applied forces and moments

(1)P The forces and moments applied to joints at the ultimate limit state shall be determined according to the principles in EN 1993-1-1.

2.4 Resistance of joints

(1)P The resistance of a joint shall be determined on the basis of the resistances of the individual fasteners, welds and other components of the joint.

(2) Linear-elastic analysis may be used in the design of the joint. Alternatively elastic-plastic analysis of the joint may be used provided that it takes account of the load deformation characteristics of all the components of the joint.

(3) Where fasteners with different stiffenesses are used to carry a shear load the fasteners with the highest stiffness should be designed to carry the design load. As an exception to this rule, see 3.9.3.

2.5 Design assumptions

(1)P Joints shall be designed by using an assumed distribution of internal forces and moments provided that they fulfil the following assumptions:

(a) the internal forces and moments assumed in the analysis are in equilibrium with the forces and moments applied to the joints,

(b) each element in the joint is capable of resisting the internal forces and moments,

(c) the deformations implied by this distribution do not exceed the deformation capacity of the fasteners or welds and the connected parts,

(d) the assumed distribution of internal forces shall be realistic with regard to relative stiffnesses within the joint,

(e) the deformations assumed in any design model based on elastic-plastic analysis are based on rigid body rotations and/or in-plane deformations which are physically possible, and

(f) any model used is in compliance with the evaluation of test results (see EN 1990).

(2) The application rules given in this part satisfy (*1*).

2.6 Joints loaded in shear subject to impact, vibration and/or load reversal

(1) Where a joint loaded in shear is subject to impact or significant vibration, either welding or bolts with locking devices or preloaded bolts or injection bolts or other types of bolt which effectively prevent movement of the connected parts or rivets should be used.

(2) Where slipping is not acceptable in a joint (because it is subject to reversal of shear load or for any other reason), either preloaded bolts in a Category B or C connection (see 3.4) or fit bolts (see 3.6.1) or rivets or welding should be used.

(3) For wind and/or stability bracings, bolts in Category A connections (see 3.4) may be used.

2.7 Eccentricity at intersections

(1) Where there is eccentricity at intersections, the joints and members should be designed for resulting moments and forces, except in the case of particular types of structures where it has been demonstrated that it is not necessary, see e.g. 3.10.3(2).

(2) In the case of joints with angles or tees connected by at least two bolts at each joint, the setting out lines for the bolts in the angles and tees may be substituted by the centroidal axes for the purpose of determining the intersection at the joints. This is also valid for the case of two setting lines in a wide leg, see figure 2.1. Possible eccentricity should be taken into account as given in (1).

NOTE: For the influence of the eccentricity to angles as web members in compression, see EN 1993-1-1.

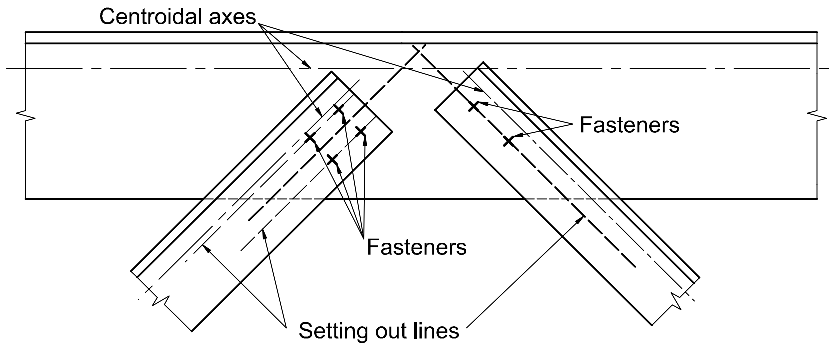


Figure 2.1: Setting out lines

2.8 References

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

Reference Standards, Group 1: Weldable structural steels

prEN 10025:2001 Hot rolled products of non-alloy structural steels - Technical delivery conditions

Reference Standards, Group 2: Tolerances, dimensions and technical delivery conditions

EN 10029:1991 Hot rolled steel plates 3 mm thick or above - Tolerances on dimensions, shape and mass

EN 10034:1993 Structural steel I- and H- sections - Tolerances on shape and dimensions

EN 10051:1991 Continuously hot-rolled uncoated plate, sheet and strip of non-alloy and alloy steels - Tolerances on dimensions and shape

EN 10055:1995 Hot rolled steel equal flange tees with radiused root and toes - Dimensions and tolerances on shape and dimensions

EN 10056-1:1995 Structural steel equal and unequal leg angles - Part 1: Dimensions

EN 10056-2:1993 Structural steel equal and unequal leg angles - Part 2: Tolerances on shape and dimensions

EN 10164:1993 Steel products with improved deformation properties perpendicular to the surface of the product - Technical delivery conditions

Reference Standards, Group 3: Structural hollow sections

EN 10219-1:1997 Cold formed welded structural hollow sections of non-alloy and fine grain steels - Part 1: Technical delivery requirements

EN 10219-2:1997 Cold formed welded structural hollow sections of non-alloy and fine grain steels - Part 2: Tolerances, dimensions and sectional properties

EN 10210-1:1994 Hot finished structural hollow sections of non-alloy and fine grain structural steels - Part 1: Technical delivery requirements

EN 10210-2:1997 Hot finished structural hollow sections of non-alloy and fine grain structural steels - Part 2: Tolerances, dimensions and sectional properties

Reference Standards, Group 4: Bolts, nuts and washers

Pr EN WI 00185215:2001*)	High strength structural bolting for preloading -Part 1 : General Requirements
Pr EN WI 00185220:2001*)	High strength structural bolting for preloading - Part 2 : Suitability Test for preloading
Pr EN WI 00185216:2001*)	High strength structural bolting for preloading - Part 3 : System HR -Hexagon bolt and nut assemblies
Pr EN WI 00185217:2001*)	High strength structural bolting for preloading - Part 4 : System HV -Hexagon bolt and nut assemblies
Pr EN WI 00185218:2001*)	High strength structural bolting for preloading - Part 5 : Plain washers for system HR
Pr EN WI 00185219:2001*)	High strength structural bolting for preloading - Part 6 : Plainchamfered washers for systems HR and HV

*) Numbers and dates to be checked during final editing of prEN 1993-1-8.

EN ISO 898-1:1999	Mechanical properties of fasteners made of carbon steel and alloy steel - Part 1: Bolts, screws and studs (ISO 898-1:1999)
EN 20898-2:1993	Mechanical properties of fasteners - Part 2: Nuts with special proof load values - Coarse thread (ISO 898-2:1992)
EN ISO 2320:1997	Prevailing torque type steel hexagon nuts - Mechanical and performance requirements (ISO 2320:1997)
EN ISO 4014:2001	Hexagon head bolts - Product grades A and B (ISO 4014:1999)
EN ISO 4016:2001	Hexagon head bolts - Product grade C (ISO 4016:1999)
EN ISO 4017:2001	Hexagon head screws - Product grades A and B (ISO 4017:1999)
EN ISO 4018:2001	Hexagon head screws - Product grade C (ISO 4018:1999)
EN ISO 4032:2001	Hexagon nuts, style 1 - Product grades A and B (ISO 4032:1999)
EN ISO 4033:2001	Hexagon nuts, style 2 - Product grades A and B (ISO 4033:1999)
EN ISO 4034:2001	Hexagon nuts - Product grade C (ISO 4034:1999)
EN ISO 7040:1997	Prevailing torque hexagon nuts (with non-metallic insert), style 1 - Property classes 5, 8 and 10
EN ISO 7042:1997	Prevailing torque all-metal hexagon nuts, style 2 - Property classes 5, 8, 10 and 12)
EN ISO 7719:1997	Prevailing torque type all-metal hexagon nuts, style 1 - Property classes 5, 8 and 10)
ISO 286- 2:1988	ISO system of limits and fits - Part 2: Tables of standard tolerance grades and limit deviations for hole and shafts
ISO 1891:1979	Bolts, screws, nuts and accessories - Terminology and nomenclature - Trilingual edition
EN ISO 7089:2000	Plain washers- Nominal series- Product grade A)
EN ISO 7090:2000	Plain washers, chamfered - Normal series - Product grade A
EN ISO 7091:2000	Plain washers - Normal series - Product grade C
EN ISO 10511:1997	Prevailing torque type hexagon thin nuts (with non-metallic insert)
EN ISO 10512:1997	Prevailing torque type hexagon nuts thin nuts, style 1, with metric fine pitch thread - Property classes 6, 8 and 10
EN ISO 10513:1997	Prevailing torque type all-metal hexagon nuts, style 2, with metric fine pitch thread - Property classes 8, 10 and 12

Reference Standards, Group 5: Welding consumable and welding

EN 12345:1998	Welding-Multilingual terms for welded joints with illustrations. September 1998.
EN-ISO 14555:1995	Welding-Arc stud welding of metallic materials. May 1995
PrEN ISO 13918:1997	Welding-Studs for arc stud welding-January 1997
EN 288-3:1992	Specification and approval of welding procedures for metallic materials. Part 3: Welding procedure tests for arc welding of steels. 1992
prEN ISO 5817:2000	Arc-welded joints in steel - Guidance for quality levels for imperfections

Reference Standards, Group 6: Rivets

NOTE: Reference should be given in ***the*** National Annex.

Reference Standard, Group 7: Execution of steel structures

EN xxx[1)] Requirements for the execution of steel structures

1) EN xxx is the conversion of ENV 1090

3 Connections made with bolts, rivets or pins

3.1 Bolts, nuts and washers

3.1.1 General

(1) Bolts, nuts and washers should conform with Reference Standards of Group 4, as appropriate.

(2) The rules in this part are applicable to bolts of grades given in table 3.1.

(3) The nominal values of the yield strength f_{yb} and the ultimate tensile strength f_{ub} are given in table 3.1 and they should be adopted as characteristic values in calculations.

Table 3.1: Nominal values of the yield strength f_{yb} and the ultimate tensile strength f_{ub} for bolts

Bolt grade	4.6	5.6	6.8	8.8	10.9
f_{yb} (N/mm^2)	240	300	480	640	900
f_{ub} (N/mm^2)	400	500	600	800	1000

3.1.2 Preloaded bolts

(1) High strength structural bolts for preloading of bolt grades 8.8 and 10.9 which conform with the requirements for this type of product in Group 4 of the Reference Standards, may be used as preloaded bolts when controlled tightening is carried out in accordance with the relevant requirements in Group 7 of the Reference Standards.

3.2 Rivets

(1) The material properties, dimensions and tolerances of steel rivets should conform with Reference Standards of Group 6.

3.3 Anchor bolts

(1) The following materials may be used for anchor bolts:

- Steel grades according to appropriate Reference Standards of Group 1;
- Steel grades according to appropriate Reference Standards of Group 4;
- Reinforcing bars according to EN 10080,

provided that the nominal yield strength does not exeed 640 N/mm^2.

3.4 Categories of bolted connections

3.4.1 Shear connections

(1) The design of a bolted connection loaded in shear should conform with one of the following categories A, B or C, see table 3.2.

(2) **Category A: Bearing type**
In this category bolts from grade 4.6 up to and including grade 10.9 should be used. No preloading and special provisions for contact surfaces are required. The design ultimate shear load should not exceed the design shear resistance nor the design bearing resistance, obtained from 3.6.

(3) **Category B: Slip-resistant at serviceability limit state**

In this category preloaded 8.8 or 10.9 bolts with controlled tightening in conformity with execution standards (see 2.8) should be used. Slip should not occur at the serviceability limit state. The design serviceability shear load should not exceed the design slip resistance, obtained from 3.9. The design ultimate shear load should not exceed the design shear resistance nor the design bearing resistance, obtained from 3.6.

(4) **Category C: Slip-resistant at ultimate limit state**
In this category preloaded 8.8 or 10.9 bolts with controlled tightening in conformity with execution standards (see 2.8) should be used. Slip should not occur at the ultimate limit state. The design ultimate shear load should not exceed the design slip resistance obtained from 3.9 nor the design bearing resistance obtained from 3.6. In addition for a connection in tension, the design plastic resistance of the net cross-section at bolt holes $N_{net,Rd}$, see 6.2 of EN 1993-1-1, should be checked, at the ultimate limit state

3.4.2 Tension connections

(1) The design of a bolted connection loaded in tension should conform with one of the following categories D and E, see table 3.2.

(2) **Category D: Connections with non-preloaded bolts**
In this category bolts from grade 4.6 up to and including grade 10.9 should be used. No preloading is required. This category should not be used where the connections are frequently subjected to variations of tensile loading. However, they may be used in connections designed to resist normal wind loads.

(3) **Category E: Connections with preloaded 8.8 or 10.9 bolts**
In this category preloaded 8.8 and 10.9 bolts with controlled tightening in conformity with execution standards (see 2.8) should be used.

Table 3.2: Categories of bolted connections

Category	Criteria	Remarks
Shear connections		
A bearing type	$F_{v,Ed} \leq F_{v,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$	No preloading required. All grades from 4.6 to 10.9.
B slip-resistant at serviceability	$F_{v,Ed.ser} \leq F_{s,Rd,ser}$ $F_{v,Ed} \leq F_{v,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$	Preloaded 8.8 or 10.9 bolts. Slip see table 1.1.
C slip-resistant at ultimate	$F_{v,Ed} \leq F_{s,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$ $F_{v,Ed} \leq N_{net,Rd}$	Preloaded 8.8 or 10.9 bolts. Slip see table 1.1. $N_{net,Rd}$ see EN 1993-1-1
Tension connections		
D non-preloaded	$F_{t,Ed} \leq F_{t,Rd}$ $F_{t,Ed} \leq B_{p,Rd}$	No preloading required. All grades from 4.6 to 10.9. $B_{p,Rd}$ see table 3.4.
E preloaded	$F_{t,Ed} \leq F_{t,Rd}$ $F_{t,Ed} \leq B_{p,Rd}$	Preloaded 8.8 or 10.9 bolts. $B_{p,Rd}$ see table 3.4.
The design tensile force $F_{t,Ed}$ should include any force due to prying action, see 3.11. Bolts subjected to both shear force and tensile force shall in addition satisfy the criteria given in table 3.4.		

NOTE: When the preload is not explicitly used in the design calculations for shear resistances but required for execution purposes or as a quality measure (e.g. for fatigue) then the level of preload can be chosen in the National Annex.

3.5 Positioning of holes for bolts and rivets

(1) Minimum and maximum spacing and end and edge distances are given in the table 3.3.

NOTE 1: Connection elements in joints other than indicated in figure 3.1 (such as end plates or cleats) may be treated as joints in tension.

(2) Minimum and maximum spacing, end and edge distances for structures subjected to fatigue, see EN 1993-1-9.

Table 3.3: Minimum and maximum spacing, end and edge distances

Distances and spacings, see figure 3.1	Minimum	Maximum[1) 2) 3)]		
		Structures made of steels according to EN 10025 execpt steels acc. to EN 10025-5		Structures made of steels according to EN 10025-5
		Steel exposed to the weather or other corrosive influences	Steel not exposed to the weather or other corrosive influences	Steel used unprotected
End distance e_1	$1{,}2d_0$	$4t$ + 40 mm		The larger of $8t$ or 125 mm
Edge distance e_2	$1{,}2d_0$	$4t$ + 40 mm		The larger of $8t$ or 125 mm
Distance e_3 in slotted holes	$1{,}5d_0$ [4)]			
Distance e_4 in slotted holes	$1{,}5d_0$ [4)]			
Spacing p_1	$2{,}2d_0$	The smaller of $14t$ or 200 mm	The smaller of $14t$ or 200 mm	The smaller of $14t_{min}$ or 175 mm
Spacing $p_{1,0}$		The smaller of $14t$ or 200 mm		
Spacing $p_{1,i}$		The smaller of $28t$ or 400 mm		
Spacing p_2 [5)]	$2{,}4d_0$	The smaller of $14t$ or 200 mm	The smaller of $14t$ or 200 mm	The smaller of $14t_{min}$ or 175 mm

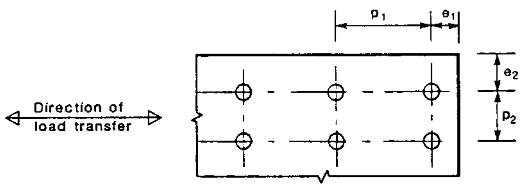
1) Maximum values for spacings, edge and end distances are unlimited, except in the following cases:
- for compression members in order to avoid local buckling and to prevent corrosion in exposed members and;
- for exposed tension members to prevent corrosion.

2) The local buckling resistance of the plate in compression between the fasteners should be calculated according to EN 1993-1-1 as column like buckling by using $0{,}6\,p_i$ as buckling length. Local buckling between the fasteners need not to be checked if p_1/t is smaller than $9\,\varepsilon$. The edge distance should not exceed the maximum to satisfy local buckling requirements for an outstand element in the compression members, see EN 1993-1-1. The end distance is not affected by this requirement.

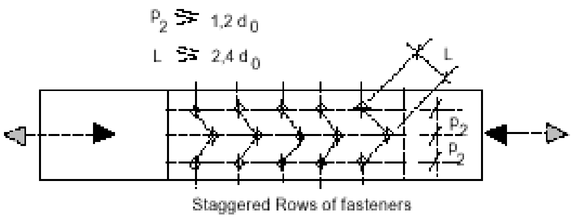
3) t is the thickness of the thinner outer connected part.

4) For the limits for slotted holes, see execution standards in 2.8

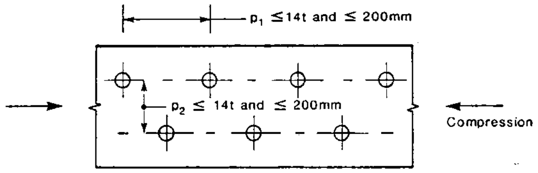
5) For staggered rows of fasteners a minimum line spacing $p_2 = 1{,}2d_0$ may be used, if the minimum distance between any two fasteners in a staggered row $L \geq 2{,}4d_0$, see Figure 3.1b).



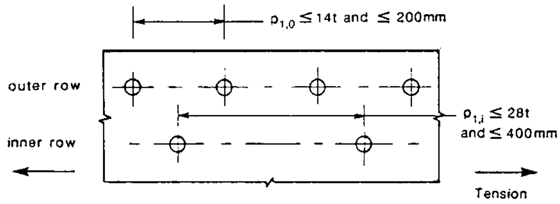
a) Symbols for spacing of fasteners



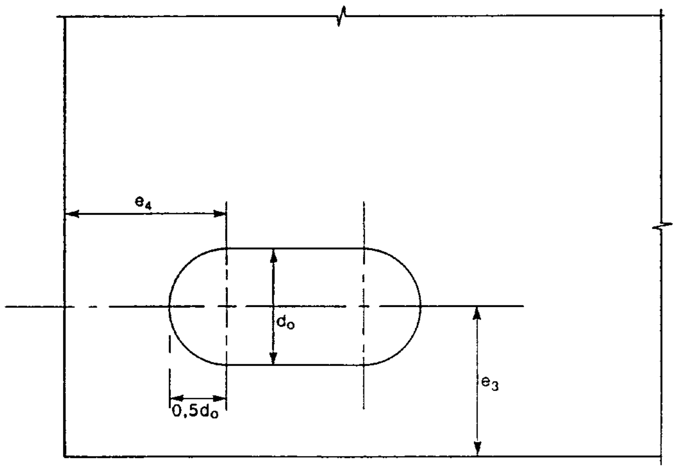
b) Symbols for staggered spacing



c) Staggered spacing - compression



d) Spacing in tension members



e) End and edge distances for slotted holes

Figure 3.1: Symbols for end and edge distances and spacing of fasteners

3.6 Design resistance of individual fasteners

3.6.1 Bolts and rivets

(1) Design resistance for individual fasteners subjected to shear and/or tension are given in table 3.4.

(2) For bolt grades 8.8 and 10.9 conforming with the appropriate Reference Standard of Group 4, with controlled tightening in conformity with execution standards (see 2.8), the design preload $F_{p,Cd}$ to be used in design calculations should be taken as:

$$F_{p,Cd} = 0,7\, f_{ub}\, A_s / \gamma_{M7} \qquad \text{... (3.1)}$$

NOTE: For preload not used in design calculations see table 3.2.

(3) The design resistances for tension and for shear through the threaded portion given in table 3.4 are restricted to bolts manufactured in conformity with appropriate Reference Standard of Group 4. For other items with cut threads, such as anchor bolts or tie rods fabricated from round steel bars where the threads are cut by the steelwork fabricator and not by a specialist bolt manufacturer, the relevant values from table 3.4 should be reduced by multiplying them by a factor of 0,85.

(4) The values for design shear resistance $F_{v,Rd}$ given in table 3.4 apply only where the bolts are used in holes with nominal clearances not exceeding those for standard holes as specified in execution standards (see 2.8).

(5) M12 and M14 bolts may also be used in 2 mm clearance holes provided that the design resistance of the bolt group based on bearing alone is greater or equal to the design resistance of the bolt group based on bolt shear alone. In addition for bolts of grade 4.8, 5.8, 6.8, 8.8 or 10.9 the design shear resistance $F_{v,Rd}$ should be taken as 0,85 times the value given in table 3.4.

(6) The resistances of fit bolts may be calculated by the same way as for bolts in normal clearance holes.

(7) The thread of a fit bolt should not be included in the shear plane.

(8) The length of the threaded portion of the shank of the fit bolt included in the bearing length should not exceed 1/3 of the thickness of the plate, see figure 3.2.

(9) The tolerance of the holes for fit bolts should be in accordance with execution standards (see 2.8).

(10) In single lap joints of flats with only one bolt row, see figure 3.3, the bolts should be provided with washers under both the head and the nut and the bearing resistance $F_{b,Rd}$ for each bolt should be limited to:

$$F_{b,Rd} \leq 1{,}5\, f_u\, d\, t / \gamma_{M2} \qquad \text{... (3.2)}$$

NOTE: Single rivets should not be used in single lap joints.

(11) In the case of 8.8 or 10.9 bolts, hardened washers should be used for single lap joints of flats with only one bolt or one rwo of bolts (normal to the direction of load), even where the bolts are not preloaded.

(12) Where bolts or rivets transmitting load in shear and bearing pass through packings of total thickness t_p greater than one-third of the nominal diameter d, see figure 3.4, the design shear resistance $F_{v,Rd}$ calculated as specified in table 3.4 , should be reduced by multiplying it by a reduction factor β_p given by:

$$\beta_p = \frac{9d}{8d + 3t_p} \quad \text{but} \quad \beta_p \leq 1 \qquad \text{... (3.3)}$$

(13) For double shear connections with packings on both sides of the splice, t_p should be taken as the thickness of the thicker packing.

(14) Riveted connections should be designed to transfer forces essentially in shear. If tension is necessary to satisfy equilibrium, the design tensile force $F_{t.Ed}$ should not exceed the design tension resistance $F_{t,Rd}$ given in table 3.4.

(15) For grade S 235 the "as driven" value of f_{ur} may be taken as 400 N/mm^2.

(16) As a general rule, the grip length of a rivet should not exceed 4,5d for hammer riveting and 6,5d for press riveting.

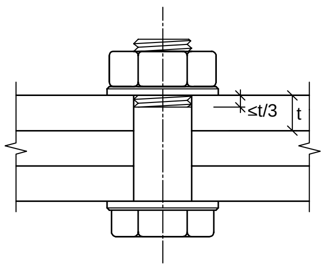


Figure 3.2: Threaded portion of the shank in the bearing length for fit bolts

Table 3.4: Design resistance for individual fasteners subjected to shear and/or tension

Failure mode	Bolts	Rivets
Shear resistance per shear plane	$F_{v,Rd} = \dfrac{\alpha_v\ f_{ub}\ A}{\gamma_{M2}}$ - where the shear plane passes through the threaded portion of the bolt (*A* is the tensile stress area of the bolt A_s): - for strength grades 4.6, 5.6 and 8.8: $\alpha_v = 0{,}6$ - for strength grades 4.8, 5.8, 6.8 and 10.9: $\alpha_v = 0{,}5$ - where the shear plane passes through the unthreaded portion of the bolt (*A* is the gross cross section of the bolt): $\alpha_v = 0{,}6$	$F_{v,Rd} = \dfrac{0{,}6\ f_{ur}\ A_o}{\gamma_{M2}}$
Bearing resistance [1), 2), 3)]	$F_{b,Rd} = \dfrac{k_1\ \alpha_b\ f_u\ d\ t}{\gamma_{M2}}$ where α_b is the smallest of α_d ; $\dfrac{f_{ub}}{f_u}$ or 1,0; in the direction of load transfer: - for end bolts: $\alpha_d = \dfrac{e_1}{3d_o}$, for inner bolts: $\alpha_d = \dfrac{p_1}{3d_o} - \dfrac{1}{4}$ perpendicular to the direction of load transfer: - for edge bolts: k_1 is the smallest of $2{,}8\dfrac{e_2}{d_o} - 1{,}7$ or 2,5 - for inner bolts: k_1 is the smallest of $1{,}4\dfrac{p_2}{d_o} - 1{,}7$ or 2,5	
Tension resistance [2)]	$F_{t,Rd} = \dfrac{k_2\ f_{ub}\ A_s}{\gamma_{M2}}$ where $k_2 = 0{,}63$ for countersunk bolt, otherwise $k_2 = 0{,}9$.	$F_{t,Rd} = \dfrac{0{,}6\ f_{ur}\ A_o}{\gamma_{M2}}$
Punching shear resistance	$B_{p,Rd} = 0{,}6\ \pi\ d_m\ t_p\ f_u\ /\ \gamma_{M2}$	No check needed
Combined shear and tension	$\dfrac{F_{v,Ed}}{F_{v,Rd}} + \dfrac{F_{t,Ed}}{1{,}4\ F_{t,Rd}} \le 1{,}0$	

1) The bearing resistance $F_{b,Rd}$ for bolts
- in oversized holes is 0,8 times the bearing resistance for bolts in normal round holes.
- in slotted holes, where the longitudinal axis of the slotted hole is perpendicular to the direction of the force transfer, is 0,6 times the bearing resistance for bolts in normal round holes

2) For countersunk bolt:
- for the determination of the bearing resistance $F_{b,Rd}$ the plate thickness *t* of the relevant part joined should be deducted with half the depth of the countersink.
- for the determination of the tension resistance $F_{t,Rd}$ the angle and depth of countersinking should conform with appropriate Reference Standards of Group 4, otherwise the tension resistance $F_{t,Rd}$ should be adjusted accordingly.

3) When the load on a bolt is not parallel to the edge, the bearing resistance may be verified separately for the bolt load components parallel and normal to the end.

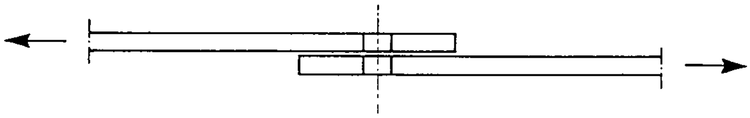


Figure 3.3: Single lap joint with one row of bolts

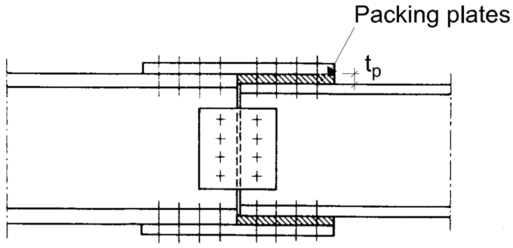


Figure 3.4: Fasteners through packings

3.6.2 Injection bolts

3.6.2.1 General

(1) If slip is not allowed in a connection, injection bolts when injected with resin give an alternative to slip resistant connections for both bolts and rivets.

(2) Only bolt grades 8.8 and 10.9 are covered.

(3) Fabrication and erection details for hexagon injection bolts are given in execution standards (see 2.8).

3.6.2.2 Design resistance

(1) Injection bolts used in connections of primary structural members may be designed as category A, B or C as specified in 3.4.

(2)P The design ultimate shear load shall not exceed either the design shear resistance of the bolt, or the bearing resistance of the resin.

(3) For connections of category B and C, preloaded injection bolts with controlled tightening should be used.

(4)P The design serviceability shear load for category B and the design ultimate shear load for Category C shall not exceed the design slip resistance of the connection plus the design bearing resistance of the resin at the respective limit states. The design ultimate shear load shall not exceed the design shear resistance of the bolt, nor the bearing resistance of the steel plates, calculated for a connection without resin.

(5) The bearing resistance of the resin, $F_{b,Rd.resin}$, should be determined according to the following equation:

$$F_{b,Rd,resin} = \frac{k_t\ k_s\ d\ t_{min}\ \beta\ f_{b,resin}}{\gamma_{M4}} \qquad \text{... (3.4)}$$

where: $F_{b,Rd,resin}$ is the bearing strength of an injection bolt

ß is a coefficient depending of the thickness ratio of the connected plates as given in table 3.5 and figure 3.5

$f_{b,resin}$ is the bearing strength of the resin to be determined according to the execution standards (see 2.8).

t_{min} is the minimum plate thickness, given in table 3.5

k_t is 1,0 for serviceability limit state (long duration)

is 1,2 for ultimate limit state

k_s is taken as 1,0 for holes with standard clearances or (1,0 - 0,1 m), for oversized holes

m is the difference (in mm) between the normal and oversized hole dimensions. In the case of short slotted holes as specified in execution standards (see 2.8), m = 0.5 x (the difference (in mm) between the hole length and width).

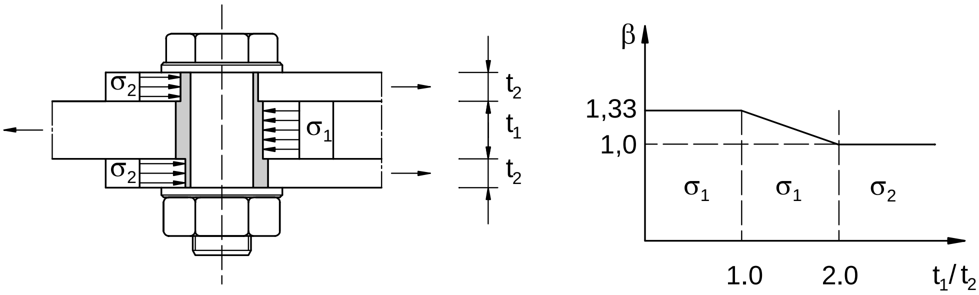


Figure 3.5: Increase of $f_{b,resin}$ by a factor ß in case of thick cover plates

Table 3.5: Values of *ß* and t_{min}

t_1 / t_2	ß	t_{min}
$\geq$2,0	1,0	2 t_2
1,0 < t_1 / t_2 < 2,0	1,66 - 0,33 (t_1 / t_2)	t_1
$\leq$1,0	1,33	t_1

(6) For bolts with a clamping length exceeding 3d, a value of ℓ of not more than 3d should be taken into account for calculation of the bearing resistance (see figure 3.6)

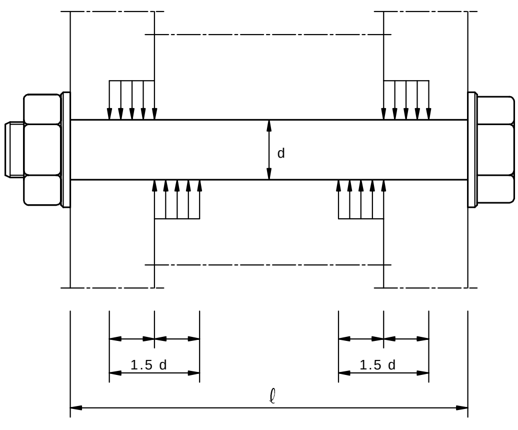


Figure 3.6: Limiting effective length for long injection bolts

3.7 Group of fasteners

(1) The resistance of a group of fasteners may be determined as the sum of the bearing resistances $F_{b,Rd}$ of the individual fasteners provided that the design shear resistance $F_{v,Rd}$ of each individual fastener is greater or equal to the design bearing resistance $F_{b,Rd}$. Otherwise the resistance of a group of fasteners should be determined by using the smallest resistance of the individual fasteners multiplied by the number of fasteners.

3.8 Long joints

(1) Where the distance L_j between the centres of the end fasteners in a joint, measured in the direction of the transfer of force (see figure 3.7), is more than 15 d, the design shear resistance $F_{v,Rd}$ of all the fasteners calculated according to table 3.4 should be reduced by multiplying it by a reduction factor β_{Lf}, given by:

$$\beta_{Lf} = 1 - \frac{L_j - 15\,d}{200\,d} \quad \text{... (3.5)}$$

but $\beta_{Lf} \leq 1{,}0$ and $\beta_{Lf} \geq 0{,}75$

(2) This provision in (1) does not apply where there is a uniform distribution of force transfer over the length of the joint, e.g. the transfer of shear force between the web and the flange of a section.

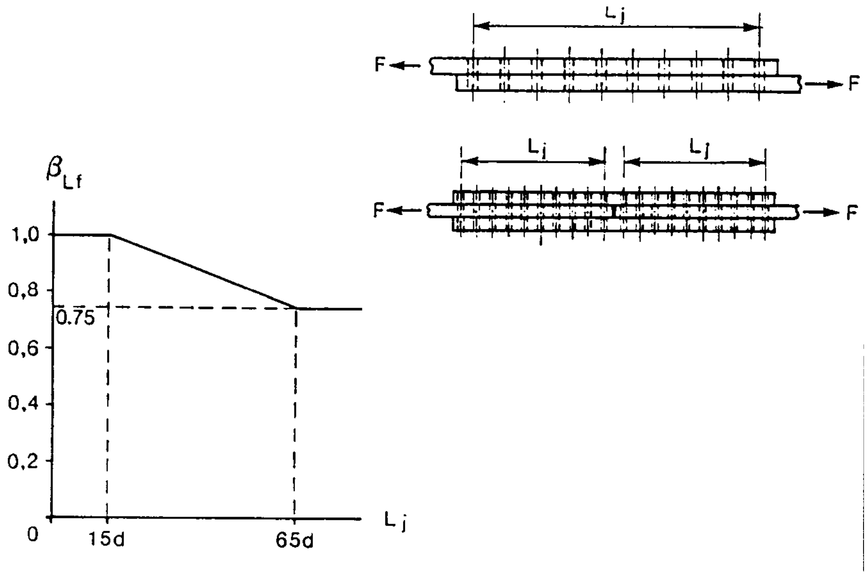


Figure 3.7: Long joints

3.9 Slip-resistant connections using 8.8 or 10.9 bolts

3.9.1 Slip resistance

(1) The design slip resistance of a preloaded 8.8 or 10.9 bolt should be taken as:

$$F_{s,Rd} = \frac{k_s\, n\, \mu}{\gamma_{M3}} F_{p,C} \quad \text{... (3.6)}$$

where: k_s should be taken as given in table 3.6

n is the number of the friction surfaces

μ is the slip factor should be taken as given in table 3.7.

(2) For bolt grades 8.8 and 10.9 conforming with the appropriate Reference Standard of Group 4, with controlled tightening in conformity with execution standards (see 2.8), the ~~*design*~~ preloading force $F_{p,C}$ ***to be used in equations (3.6) or (3.7)*** should be taken as:

$$F_{p,C} = 0{,}7\, f_{ub}\, A_s \quad \text{... (3.7)}$$

(3) The relevant combination of actions for serviceability limit states should be used.

(4) Additional checks at the serviceability limit state and at the ultimate limit state, see 3.4.

Table 3.6: Values of k_s

	k_s
Where the holes in all the plies have standard nominal clearance as specified in execution standards (see 2.8)	1,0
For oversize holes, or short slotted holes, as specified in execution standards (see 2.8). The axis of the slot perpendicular to the direction of force transfer.	0,85
For long slotted holes, as specified in execution standards (see 2.8). The axis of the slot perpendicular to the direction of force transfer.	0,7
For oversize holes, or short slotted holes, as specified in execution standards (see 2.8). The axis of the slot parallel to the direction of force transfer.	0,76
For long slotted holes, as specified in execution standards (see 2.8). The axis of the slot parallel to the direction of force transfer.	0,63

Table 3.7: The design value of the slip factor μ

Classes of friction surfaces according to the execution standards (see 2.8)	Design value of the slip factor μ
A	0,5
B	0,4
C	0,3
D	0,2

NOTE 1: The requirements of testing and inspection, see execution standards in 2.8.
NOTE 2: The classification of any other surface treatment should be based on tests on specimens representative of the surfaces used in the structure using the procedure set out in execution standards (see 2.8).
NOTE 3: The definitions of the classes of friction surfaces, see execution standards in 2.8.
NOTE 4: The grade of the cleanliness of the surfaces should be given in the project specification.
NOTE 5: With painted surface treatments account may have to be made for the pre-load losses which occur over time.

3.9.2 Combined tension and shear

(1) If a slip-resistant connection is subjected to an applied tensile force $F_{t,Ed}$ in addition to the shear force $F_{v,Ed}$ tending to produce slip, the slip resistance per bolt should be taken as follows:

$$F_{s,Rd} = \frac{k_s\ n\ \mu\ (F_{p,C}\ -\ 0{,}8\ F_{t,Ed})}{\gamma_{M3}} \qquad \text{... (3.8)}$$

(2) If, in a moment connection, the applied tensile force is counterbalanced by a contact force on the compression side, no reduction of the slip resistance is required.

3.9.3 Hybrid connections

(1) As an exception to 2.4(3) , preloaded 8.8 and 10.9 bolts in connections designed as slip-resistant at the ultimate limit state (Category C in 3.4) may be assumed to share load equally with welds, provided that the final tightening of the bolts is carried out after the welding is complete.

3.10 Deductions for fastener holes

3.10.1 General

(1) Deduction for holes in the member design should be made according to EN 1993-1-1.

3.10.2 Design for block tearing

(1) The design value for block tearing $V_{eff,1,Rd}$ for symmetric bolt groups under centric loading should be determined from:

$$V_{eff,1,Rd} = f_u A_{nt} / \gamma_{M2} + (1 / \sqrt{3}) f_y A_{nv} / \gamma_{M0} \quad \text{... (3.9)}$$

where: A_{nt} is net area subjected to tension;

A_{nv} is net area subjected to shear.

(2) For beam end with a shear force acting eccentric relative to the bolt group the design value for block tearing $V_{eff,2,Rd}$ should be determined from:

$$V_{eff,2,Rd} = 0{,}5\, f_u A_{nv} / \gamma_{M2} + (1 / \sqrt{3}) f_y A_{nt} / \gamma_{M0} \quad \text{... (3.10)}$$

NOTE: Tensile failure occurs along the horizontal limit of the block, and shear plastic yielding occurs along the left vertical limit of the block. The block is hatched in figure 3.8.

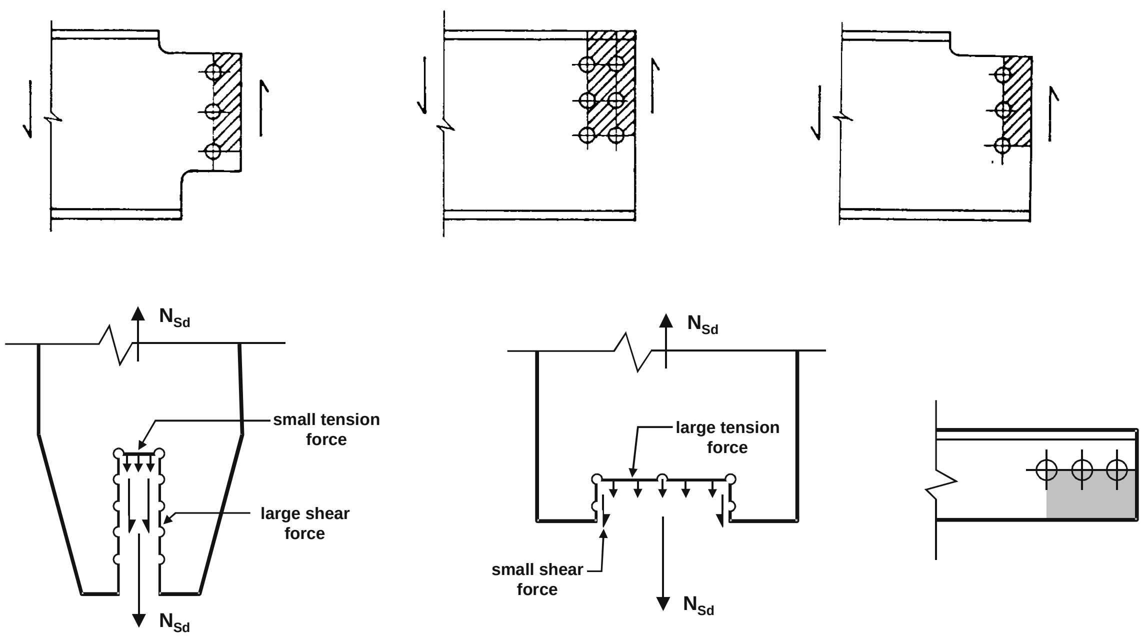


Figure 3.8: Block tearing

3.10.3 Angles connected by one leg and other unsymmetrically connected members in tension

(1)P The eccentricity of fasteners in end connections, and the effects of the spacing and edge distances of the bolts, shall be taken into account in determining the design resistance of:

- unsymmetrical members;
- symmetrical members that are connected unsymmetrically, such as angles connected by one leg.

NOTE: Design of angles connected by one leg and other unsymmetrically connected members for compression, see EN 1993-1-1.

(2) A single angles connected by a single row of bolts in one leg, see figure 3.9, may be treated as concentrically loaded and the design ultimate resistance of the net section determined as follows:

with 1 bolt: $$N_{u,Rd} = \frac{2{,}0\ (e_2 - 0{,}5d_o)\ t\ f_u}{\gamma_{M2}} \qquad \text{... (3.11)}$$

with 2 bolts: $$N_{u,Rd} = \frac{\beta_2\ A_{net}\ f_u}{\gamma_{M2}} \qquad \text{... (3.12)}$$

with 3 or more bolts: $$N_{u,Rd} = \frac{\beta_3\ A_{net}\ f_u}{\gamma_{M2}} \qquad \text{... (3.13)}$$

where: β_2 and β_3 are reduction factors dependent on the pitch p_1 as given in table 3.8. For intermediate values of p_1 the value of β may be determined by linear interpolation;

A_{net} is the net area of the angle. For an unequal-leg angle connected by its smaller leg, A_{net} should be taken as equal to the net section area of an equivalent equal-leg angle of leg size equal to that of the smaller leg.

Table 3.8: Reduction factors β_2 and β_3

Pitch	p_1	$\leq 2{,}5\ d_o$	$\geq 5{,}0\ d_o$
2 bolts	β_2	0,4	0,5
3 bolts or more	β_3	0,5	0,7

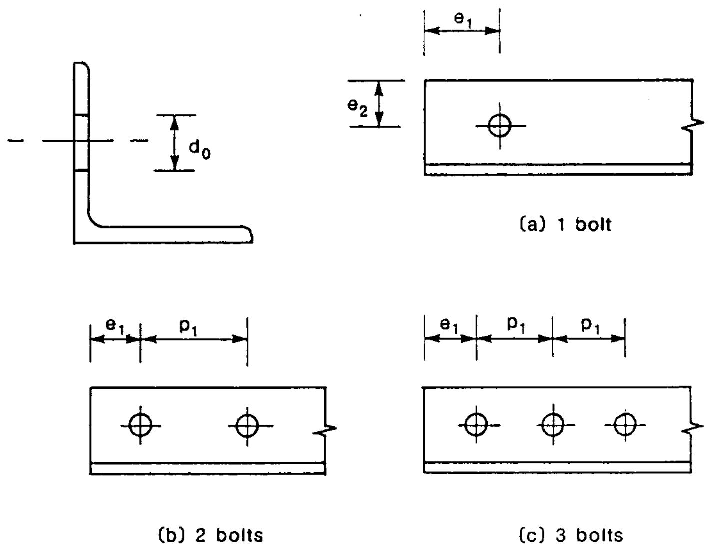


Figure 3.9: Connections of angles

3.10.4 Lug angles

(1) Lug angles, see figure 3.10, connecting angle members and their fastenings to a gusset or other supporting part should be designed to transmit a force 20% greater than the force in the outstand of the angle connected.

(2) The fastenings connecting the lug angle to the outstand of the angle member should be designed to transmit a force 40 % greater than the force in the outstand of the angle member.

(3) Lug angles connecting a channel or a similar member should be designed to transmit a force 10% greater than the force in the component of the member not directly connected.

(4) The fastenings connecting the lug angle to the channel or similar member should be designed to transmit a force 20 % greater than force excess as specified in (3) above.

(5) In no case should less than two bolts or rivets be used to attach a lug angle to a gusset or other supporting part.

(6) The connection of lug angles to gusset plates or other supporting parts should terminate at the end of the member connected. The connection of the lug angle to the member should run from the end of the member to a point beyond the direct connection of the member to the gusset or other supporting part.

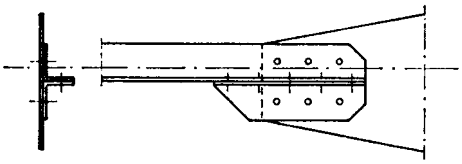


Figure 3.10: Lug angles

3.11 Prying forces

(1) Where fasteners are required to carry an applied tensile force, they shall be proportioned to also resist the additional force due to prying action, where this can occur.

NOTE: Rules given in 6.2.2 implicitly account for prying forces.

3.12 Distribution of forces between fasteners at the ultimate limit state

(1) When a moment is applied to a joint, the distribution of internal forces may be either linear proportional to the distance from the centre of rotation or else plastic, i.e. any distribution that is in equilibrium is acceptable provides that the resistances of the components are not exceeded and the ductility of the components are sufficient.

(2) The distribution of internal forces should be proportional to the distance from the centre of rotation:

- when bolts are used creating a category C slip-resistant connection,
- in other shear connections where the design shear resistance $F_{v,Rd}$ of a fastener is less than the design bearing resistance $F_{b,Rd}$,
- where connections are subjected to impact, vibration or load reversal (except wind loads), except in connections of secondary structural elements.

(3) When a joint is loaded by a non-eccentric shear only, the load may be assumend to be uniformly distributed amongst the fasteners, if the size and the class of fasteners is the same.

3.13 Connections made with pins

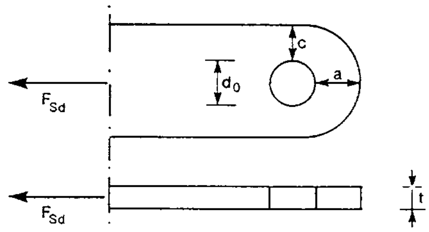
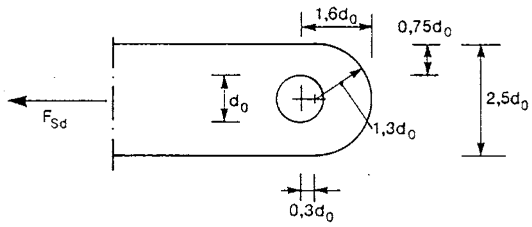
3.13.1 General

(1) Wherever there is a risk of pins becoming loose, they should be secured.

(2) Pin connections in which no rotation is required may be designed as single bolted connections, if the length of the pin is less than 3 times the pin diameter, see 3.6.1. For other cases the rules below should be applied.

(3) The geometry of plates in pin connections should be in accordance with the dimensional requirements given in table 3.9.

Table 3.9: Geometrical conditions for plates in pin connections

Type A: Given thickness t	 $a \geq \frac{F_{Sd}\,\gamma_{M0}}{2tf_y} + \frac{2d_0}{3}$: $c \geq \frac{F_{Sd}\,\gamma_{M0}}{2tf_y} + \frac{d_0}{3}$
Type B: Given geometry	 $t \geq 0{,}7\left[\frac{F_{Sd}\,\gamma_{M0}}{f_y}\right]^{1/2}$: $d_0 \leq 2{,}5t$

(4) Pin plates that are provided to increase the net cross-sectional area of a member, or to increase the bearing resistance of a pin, should be arranged to avoid eccentricity and should be of sufficient size to distribute the load from the pin into the member.

3.13.2 Design of pins

(1) The design requirements for solid circular pins are given in table 3.10.

(2) The moment in a pin should be calculated on the basis that the connected parts form simple supports, and that the reactions between the pin and the connected parts are uniformly distributed along the length in contact on each part as indicated in figure 3.11.

(3) If the pin is intended to be replaceable, in addition to the provisions given in 3.13.1 to 3.13.2, the contact bearing stress should satisfy:

$$\sigma_{h,Ed} \leq f_{h,Rd} \qquad \text{... (3.14)}$$

where:

$$\sigma_{h,Ed} = 0{,}591\sqrt{\frac{E\,F_{Ed,ser}\,(d_0 - d)}{d^2\,t}} \qquad \text{... (3.15)}$$

$$f_{h,Ed} = 2{,}5 f_y/\gamma_{M6,ser} \qquad \text{... (3.16)}$$

where: d is the diameter of the pin;

d_0 is the diameter of the pin hole;

$F_{Ed,ser}$ is the design value of the force to be transferred in bearing, under the characteristic load combination for serviceability limit states.

Table 3.10: Design criteria for pin connections

Failure mode	Design requirements
Shear of the pin	$F_{v,Rd} = 0{,}6\, A\, f_{up} / \gamma_{M2} \geq F_{v,Ed}$
Bearing of the plate and the pin If the pin is intended to be replaceable, in addition:	$F_{b,Rd} = 1{,}5\, t\, d\, f_y / \gamma_{M0} \geq F_{b,Ed}$ $F_{b,Rd,ser} = 0{,}6\, t\, d\, f_y / \gamma_{M6,ser} \geq F_{b,Ed,ser}$
Bending of the pin If the pin is intended to be replaceable, in addition:	$M_{Rd} = 1{,}5\, W_{e\ell}\, f_{yp} / \gamma_{M0} \geq M_{Ed}$ $M_{Rd,ser} = 0{,}8\, W_{e\ell}\, f_{yp} / \gamma_{M6,ser} \geq M_{Ed,ser}$
Combined shear and bending of the pin	$\left[\dfrac{M_{Ed}}{M_{Rd}}\right]^2 + \left[\dfrac{F_{v,Ed}}{F_{v,Rd}}\right]^2 \leq 1$
d is the diameter of the pin; f_y is the lower of the design strengths of the pin and the connected part; f_{up} is the ultimate tensile strength of the pin; f_{yp} is the yield strength of the pin; t is the thickness of the connected part; A is the cross-sectional area of a pin.	

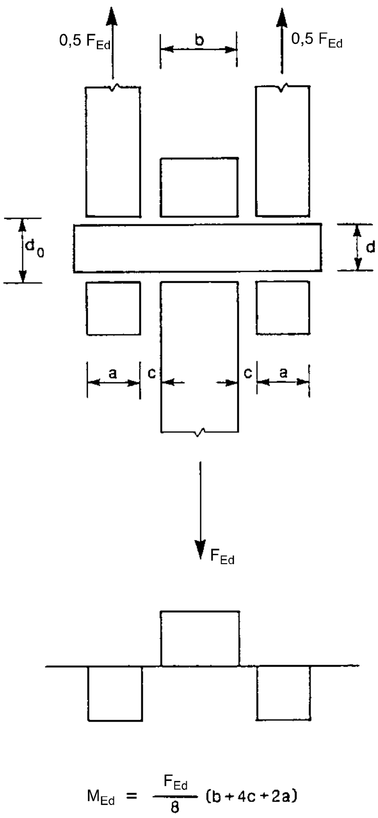


Figure 3.11: Bending moment in a pin

4 Welded connections

4.1 General

(1) The provisions in this section apply to:

- Weldable structural steels meeting the requirements of EN 1993-1-1.
- Material thicknesses of 4 mm and over. For welds in thinner material refer to EN 1993 part 1.3. For welds in structural hollow sections in material thickness of 2,5 mm and over, see section 7.
- Joints in which the weld metal is compatible with the parent metal in terms of mechanical properties, see 4.2.
- For stud welding reference is made to EN 1994-1-1.

NOTE: Guidance for stud welding are given in the standards EN ISO 14555 and EN ISO 13918.

(2)P Welds subject to fatigue shall also satisfy the principles given in EN 1993-1-9.

(3) Quality level C according to EN ISO 25817 is usually required, if not otherwise specified in the project specification. The frequence of inspection of welds should be given in the project specification by using the rules in execution standards (see 2.8). The quality level of welds should be chosen according to EN ISO 25817. The quality level of fatigue loaded structures, see EN 1993-1-9.

(4)P Lamellar tearing shall be avoided.

(5) Guidance for lamellar tearing are given in EN 1993-1-10.

4.2 Welding consumables

(1) All welding consumables should confirm with the Reference Standards of Group 5.

(2) The specified yield strength, ultimate tensile strength, elongation at failure and minimum Charpy V-notch energy value of the filler metal, should all be such that the performance of the weld metal should not be less than the corresponding values specified for steel grade being welded.

NOTE: Generally it is safe to use electrodes that are overmatched with regard to the steel grades being used.

4.3 Geometry and dimensions

4.3.1 Type of weld

(1) This EN covers the design of fillet welds, fillet welds all round, butt welds, plug welds or flare groove welds. Butt welds may be either full penetration butt welds or partial penetration butt welds. Both fillet welds all round and plug welds may be in either in circular holes or in elongated holes.

(2) The most common types of joints and welds with pictoral representation are given in EN 12345.

4.3.2 Fillet welds

4.3.2.1 General

(1) Fillet welds may be used for connecting parts where the fusion faces form an angle of between 60° and 120°.

(2) Smaller angles than 60° are also permitted. However, in such cases the weld should be considered to be a partial penetration butt weld.

(3) For angles greater than 120° the resistance of fillet welds should be determined by testing in accordance with EN 1990.

(4) Fillet welds should not terminate at corners of parts or members, but should be returned continuously, full sized, around the corner for a length equal to twice the leg size of the weld, wherever such a return can be made in the same plane.

NOTE: In case of intermittent welds this rule applies only to the last intermittent fillet welds at corners.

(5) End returns should be indicated on the drawings.

(6) For eccentricity of single-sided fillet welds, see 4.12.

4.3.2.2 Intermittent fillet welds

(1) Intermittent fillet welds shall not be used in corrosive conditions.

(2) In an intermittent fillet weld, the gaps (L_1 or L_2) between the ends of each length of weld L_w should fulfil the requirement given in figure 4.1.

(3) In an intermittent fillet weld, the gap should be measured between the ends of welds on opposing sides or on the same side, whichever is shorter.

(4) In any run of intermittent fillet welds there should always be a length of weld at each end of the part connected.

(5) In a build-up member in which plates are connected by means of intermittent fillet welds, a continuous fillet weld should be provided on each side of the plate for a length at each end equal to at least three-quarters of the width of the narrower plate concerned (see figure 4.1).

4.3.3 Fillet welds all round

(1) Fillet welds all round, comprising fillet welds in circular or elongated holes, can be used only to transmit shear or to prevent the buckling or separation of lapped parts.

(2) The diameter of a circular hole, or width of an elongated hole, for a fillet welds all round should not be less than four times the thickness of the part containing it.

(3) The ends of elongated holes should be half-circular, except for those ends which extend to the edge of the part concerned.

(4) The centre to centre spacing of fillet welds all round should not exceed the value necessary to prevent local buckling, see table 3.3.

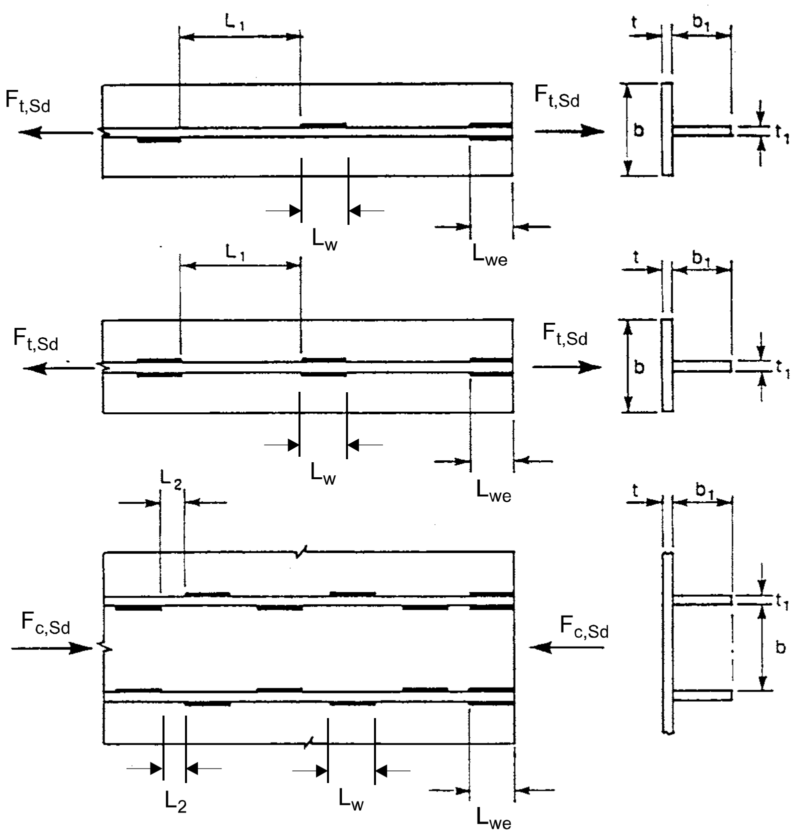
4.3.4 Butt welds

(1) A full penetration butt weld is defined as a butt weld that has complete penetration and fusion of weld and parent metal throughout the thickness of the joint.

(2) A partial penetration butt weld is defined as a butt weld that has joint penetration which is less than the full thickness of the parent material.

(3) Intermittent butt welds should not be used.

(4) For eccentricity in single-sided partial penetration butt welds, see 4.12.



The larger of $L_{we} \geq 0{,}75\ b$ and $0{,}75\ b_1$

For build-up members in tension:
The smallest of $L_1 \leq 16\ t$ and $16\ t_1$ and 200 mm

For build-up members in compression or shear:
The smallest of $L_2 \leq 12\ t$ and $12\ t_1$ and $0{,}25\ b$ and 200 mm

Figure 4.1: Intermittent fillet welds

4.3.5 Plug welds

(1) Plug welds shall not be used to resist externally applied tension, but they may be used:

- to transmit shear, or
- to prevent the buckling or separation of lapped parts, or
- to inter-connect the components of built-up members.

(2) The diameter of a circular hole, or width of an elongated hole, for a plug weld should be at least 8 mm more than the thickness of the part containing it.

(3) The ends of elongated holes should either be half-circular or else should have corners which are rounded to a radius of not less than the thickness of the part containing the slot, except for those ends which extend to the edge of the part concerned.

(4) The thickness of a plug weld in parent material up to 16 mm thick should be equal to the thickness of the parent material. The thickness of a plug weld in parent material over 16 mm thick should be at least half the thickness of the parent material and not less than 16 mm.

(5) The centre to centre spacing of plug welds should not exceed the value necessary to prevent local buckling, see table 3.3.

4.3.6 Flare groove welds

(1) For solid bars the design throat thickness of flare groove welds, when fitted flush to the surface of the solid section of the bars, is defined in figure 4.2. The definition of the design throat thickness of flare groove welds in rectangular hollow sections is given in 7.3.1(7).

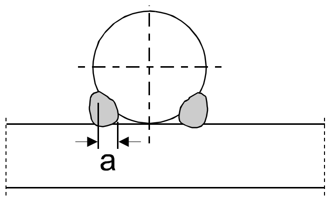


Figure 4.2: Effective throat of flare groove welds in solid sections

4.4 Welds with packings

(1) In the case of welds with packing, the packing should be trimmed flush with the edge of the part that is to be welded.

(2) Where two parts connected by welding are separated by packing having a thickness less than the leg length of weld necessary to transmit the force, the required leg length should be increased by the thickness of the packing.

(3) Where two parts connected by welding are separated by packing having a thickness equal to, or greater than, the leg length of weld necessary to transmit the force, each of the parts should be connected to the packing by a weld capable of transmitting the design force.

4.5 Design resistance of a fillet weld

4.5.1 Length of welds

(1) The effective length of a fillet weld should be taken as the overall length of the full-size fillet, including end returns. Provided that the weld is full size throughout this length, no reduction in effective length need be made for either the start or the termination of the weld. If the reduction has to be made, the actual length of the weld should be reduced and this reduction may be assumed to be as twice the effective throat thickness a. The design length of a fillet weld l is the length of the weld required by calculations and given in the drawings including end returns. Long joint, see 4.11.

(2) Welds with effective lengths shorter than 30 mm or 6 times the throat thickness, whichever is larger, should be ignored for transmission of forces.

(3) Where the stress distribution along a weld is significantly influenced by the stiffness of the members or parts joined, the non-uniformity of the stress distribution may be neglected, provided that the design resistance is correspondingly reduced.

(4) The effective widths of welded joints designed to transfer transverse loads to an unstiffened flange of an I, H or other section should be reduced as specified in 4.10.

(5) The design resistances of welds in long overlapped joints and in long transverse stiffeners should be reduced as specified in 4.11.

4.5.2 Effective throat thickness

(1) The effective throat thickness, a, of a fillet weld should be taken as the height of the largest triangle (with equal or unequal legs) which can be inscribed within the fusion faces and the weld surface, measured perpendicular to the outer side of this triangle, see figure 4.3.

(2) The effective throat thickness of a fillet weld should not be less than 3 mm.

(3) In determining the resistance of a deep penetration fillet weld, account may be taken of its additional throat thickness, see figure 4.4, provided that it is shown by preliminary tests that the required penetration can consistently be achieved.

(4) The effective throat thickness and the length required by calculations irrespective of the welding process should be given in the project specification and/or in the drawings.

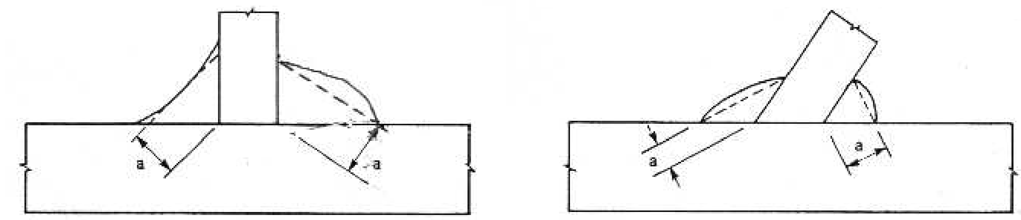


Figure 4.3: Throat thickness of a fillet weld

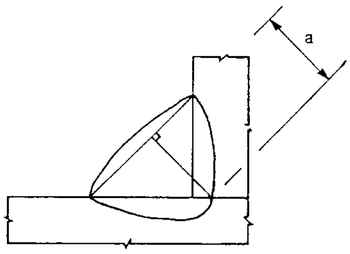


Figure 4.4: Throat thickness of a deep penetration fillet weld

4.5.3 Resistance of fillet welds

(1) The design resistance per unit length of a fillet weld should be determined using either the method given below, or else the alternative method given in 4.5.4.

(2) The design throat area A_w should be taken as $A_w = \sum a\, \ell_{eff}$.

(3) The location of the design throat area should be assumed to be concentrated in the root.

(4) In this method, the forces transmitted by a unit length of weld are resolved into components parallel and transverse to the longitudinal axis of the weld and normal and transverse to the plane of its throat.

(5) A uniform distribution of stress is assumed on the throat section of the weld, leading to the normal stresses and shear stresses shown in figure 4.5, as follows:

- $\sigma_{\perp}$ is the normal stress perpendicular to the throat
- $\sigma_{/\!/}$ is the normal stress parallel to the axis of the weld
- $\tau_{\perp}$ is the shear stress (in the plane of the throat) perpendicular to the axis of the weld
- $\tau_{/\!/}$ is the shear stress (in the plane of the throat) parallel to the axis of the weld.

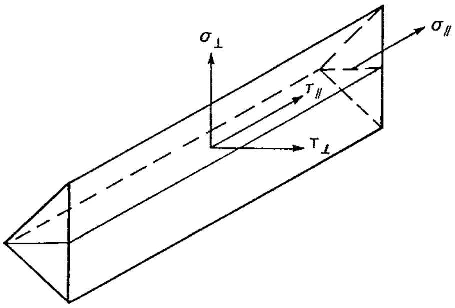


Figure 4.5: Stresses on the throat section of a fillet weld

(6) The normal stress $\sigma_{/\!/}$ parallel to the axis is not considered when verifying the resistance of the weld.

(7) The resistance of the fillet weld will be sufficient if the following are both satisfied:

$$[\sigma_{\perp}^{2} + 3\,(\tau_{\perp}^{2} + \tau_{/\!/}^{2})]^{0,5} \leq f_u / (\beta_w\, \gamma_{M2}) \quad \text{and} \quad \sigma_{\perp} \leq f_u / \gamma_{M2} \qquad \text{... (4.1)}$$

where: f_u is the nominal ultimate tensile strength of the weaker part joined;

β_w is is the appropriate correlation factor according to the table 4.1.

(8) Welds between parts with different material strength grades should be designed using the lower strength properties.

Table 4.1: Correlation factor β_w for fillet welds

Standard and steel grade			Correlation factor β_w
EN 10025	**EN 10210**	**EN 10219**	
S 235 S 235 W	S 235 H	S 235 H	0,8
S 275 S 275 N/NL S 275 M/ML	S 275 H S 275 NH/NLH	S 275 H S 275 NH/NLH S 275 MH/MLH	0,85
S 355 S 355 N/NL S 355 M/ML S 355 W	S 355 H S 355 NH/NLH	S 355 H S 355 NH/NLH S 355 MH/MLH	0,9
S 420 N/NL S 420 M/ML		S 420 MH/MLH	1,0
S 460 N/NL S 460 M/ML S 460 Q/QL/QL1	S 460 NH/NLH	S 460 NH/NLH S 460 MH/MLH	1,0

4.5.4 Simplified method for resistance of fillet welds

(1) The resistance of a fillet weld may be verified by the following method as an alternative to the method given in 4.5.3.

(2) The resistance of a fillet weld may be assumed to be adequate if, at every point in its length, the resultant of all the forces per unit length transmitted by the weld satisfiy the criterion:

$$F_{w,Ed} \leq F_{w,Rd} \quad \text{... (4.2)}$$

where: $F_{w,Ed}$ is the design value of the weld force per unit length;

$F_{w,Rd}$ is the design weld resistance per unit length.

(3) Independent of the orientation of the weld, the design resistance per unit length $F_{w,Rd}$ should be determined from:

$$F_{w,Rd} = f_{vw.d}\, a \quad \text{... (4.3)}$$

where: $f_{vw.d}$ is the design shear strength of the weld.

(4) The design shear strength $f_{vw.d}$ of the weld should be determined from:

$$f_{vw.d} = \frac{f_u/\sqrt{3}}{\beta_w \gamma_{M2}} \quad \text{... (4.4)}$$

where: f_u and β_w are defined in 4.5.3(7).

4.6 Design resistance of fillet welds all round

(1) The design resistance of a fillet weld all round should be determined from 4.5.

4.7 Design resistance of butt welds

4.7.1 Full penetration butt welds

(1) The design resistance of a full penetration butt weld should be taken as equal to the design resistance of the weaker of the parts connected, provided that the weld is made with a suitable consumable which will produce all-weld tensile specimens having both a minimum yield strength and a minimum tensile strength not less than those specified for the parent metal.

4.7.2 Partial penetration butt welds

(1) The resistance of a partial penetration butt weld should be determined as for a deep penetration fillet weld, see 4.5.2(3).

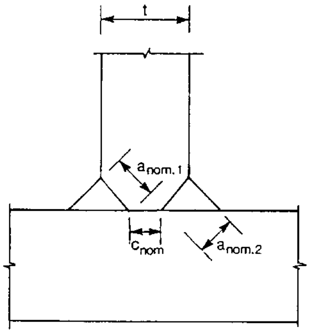
(2) The throat thickness of a partial penetration butt weld should not be taken more than the depth of penetration that can consistently be achieved, see 4.5.2(3).

(3) The throat thickness that can be consistently achieved should be determined by preliminary trials.

4.7.3 T-butt joints

(1) The resistance of a T-butt joint, consisting of a pair of partial penetration butt welds reinforced by superimposed fillet welds, may be determined as for a full penetration butt weld (see 4.7.1) if the total nominal throat thickness, exclusive of the unwelded gap, is not less than the thickness t of the part forming the stem of the tee joint, provided that the unwelded gap is not more than (t / 5) or 3 mm, whichever is less, see figure 4.6(a).

(2) The resistance of a T-butt joint which does not meet the requirements given in (1) should be determined as for a fillet weld or as for a deep penetration fillet weld (see 4.5) depending on the amount of preparation. The throat thickness should be determined in conformity with the provisions for both fillet welds (see 4.5.2) and partial penetration butt welds (see 4.7.2).



$a_{nom,1} + a_{nom,2} \geq t$
The smaller of $c_{nom} \leq t/5$ and 3 mm

Figure 4.6: Effective full penetration of T-butt welds

4.8 Design resistance of plug welds

(1) The design resistance $F_{w,Rd}$ of a plug weld (see 4.3.5) should be taken as $f_{vw,d}$ A_w, where $f_{vw.d}$ is the design shear strength of a weld given in 4.5.4(4).

(2) The design throat area A_w of a plug weld should be taken as the area of the hole.

4.9 Distribution of forces

(1) The distribution of forces in a welded connection may be calculated on the assumption of either elastic or plastic behaviour in conformity with 2.4 and 2.5.

(2) It is acceptable to assume a simplified load distribution within the welds.

(3) Residual stresses and stresses not subjected to transfer of load need not be included when checking the resistance of a weld. This applies specifically to the normal stress parallel to the axis of a weld.

(4) Welded joints should be designed to have adequate deformation capacity. However, ductility of the welds should not be relied upon.

(5) In joints where plastic hinges may form, the welds should be designed to provide at least the same design resistance as the weakest of the connected parts.

(6) In other joints where deformation capacity for joint rotation is required due to the possibility of excessive straining, the welds require sufficient strength not to rupture before general yielding in the adjacent parent material.

(7) If the design resistance of an intermittent weld is determined by using the total length ℓ_{tot}, the weld shear force per unit length $F_{w,Ed}$ should be multiplied by the factor $(e+\ell)/\ell$, see figure 4.7.

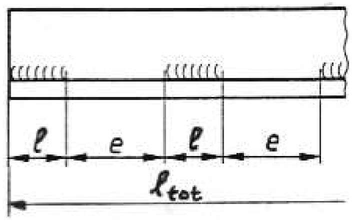


Figure 4.7: Calculation of weld forces for intermittent welds

4.10 Connections to unstiffened flanges

(1) In a T-joint of a plate to an unstiffened flange of an I, H or other section, see figure 4.8, an effective width should be taken into account both for the parent material and for the welds.

(2) For an I or H section the effective width b_{eff} should be obtained from:

$$b_{eff} = t_w + 2s + 7k\,t_f \qquad \text{... (4.5a)}$$

where: $k = (t_f / t_p)(f_{y,f} / f_{y,p})$ but $k \leq 1$ ... (4.5b)

$f_{y,f}$ is the yield strength of the flange of the I or H section;

$f_{y,p}$ is the yield strength of the plate welded to the I or H section.

(3) The dimension s should be obtained from:

- for a rolled I or H section: $s = r$ ... (4.5c)
- for a welded I or H section: $s = \sqrt{2}\,a$ ... (4.5d)

(4) For an unstiffened flange of an I or H section and for box section, the following criterion should be satisfied:

$$b_{eff} \geq (f_{y,p} / f_{u,p})\, b_p \qquad \text{... (4.6)}$$

where: $f_{u,p}$ is the ultimate strength of the plate welded to the I or H section.

b_p is the width of the plate welded to the I or H section.

Otherwise the joint should be stiffened.

(5) For other sections such as box sections or channel sections where the width of the connected plate is similar to the width of the flange, the effective width b_{eff} should be obtained from:

$$b_{eff} = 2t_w + 5t_f \quad \text{but} \quad b_{eff} \leq 2t_w + 5\,k\,t_f \qquad \text{... (4.7)}$$

NOTE: For hollow sections, see table 7.13.

(6) Even if $b_{eff} \leq b_p$, the welds connecting the plate to the flange should be designed to resist a force equal to the resistance of the plate $b_p\, t_p\, f_{y,p} / \gamma_{M0}$.

4.11 Long joints

(1) In lap joints the design resistance of a fillet weld should be reduced by multiplying it by a reduction factor β_{Lw} to allow for the effects of non-uniform distribution of stress along its length.

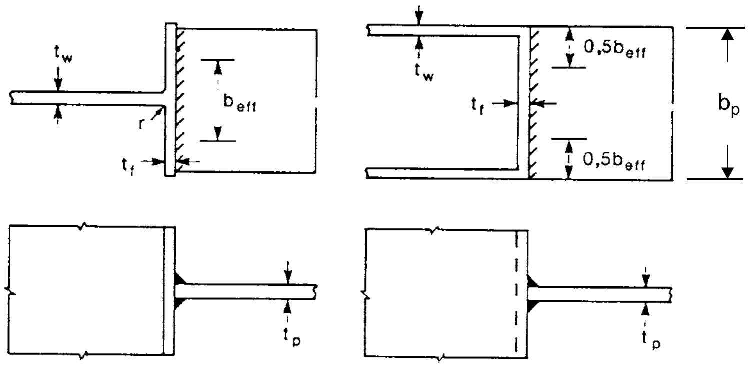


Figure 4.8: Effective width of an unstiffened T-joint

(2) This provision does not apply when the stress distribution along the weld corresponds to the stress distribution in the adjacent base metal, as, for example, in the case of a weld connecting the flange and the web of a plate girder.

(3) Generally in lap joints longer than $150a$ the reduction factor β_{Lw} should be taken as $\beta_{\mathrm{Lw.1}}$ given by:

$$\beta_{\mathrm{Lw.1}} = 1{,}2 - 0{,}2L_{\mathrm{j}}/(150a) \quad \text{but} \quad \beta_{\mathrm{Lw.1}} \leq 1{,}0 \qquad \text{... (4.8)}$$

where: L_{j} is the overall length of the lap in the direction of the force transfer.

(4) For fillet welds longer than 1,7 metres connecting transverse stiffeners in plated members, the reduction factor β_{Lw} may be taken as $\beta_{\mathrm{Lw.2}}$ given by:

$$\beta_{\mathrm{Lw.2}} = 1{,}1 - L_{\mathrm{w}}/17 \quad \text{but} \quad \beta_{\mathrm{Lw.2}} \leq 1{,}0 \quad \text{and} \quad \beta_{\mathrm{Lw.2}} \geq 0{,}6 \qquad \text{... (4.9)}$$

where: L_{w} is the length of the weld (in metres).

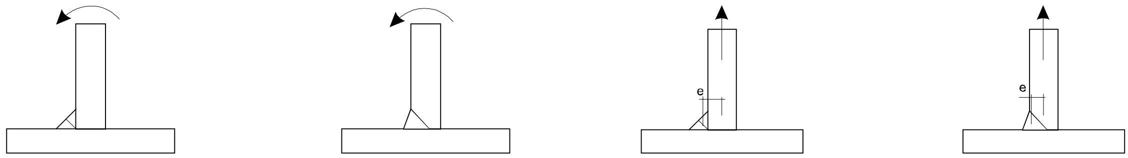
4.12 Eccentrically loaded single fillet or single-sided partial penetration butt welds

(1) Local eccentricity (relative to the line of action of the force to be resisted) should be taken into account in the following cases:

- A bending moment is transmitted about the longitudinal axis of the weld if it produces tension at the root of the weld, see figure 4.9(a);
- A tensile force is transmitted perpendicular to the longitudinal axis of the weld which would effectively produce a bending moment , figure 4.9(b).

NOTE: Local eccentricity should be avoided whenever it is possible.

(2) Local eccentricity need not be taken into account if a weld is used as part of a weld group around the perimeter of a structural hollow section, but should not be used in the situation indicated in figure 4.9(a).



(a) Bending moment produces tension at the root of the weld

(b) Tensile force produces tension at the root of the weld

Figure 4.9: Single fillet welds and single-sided partial penetration butt welds

4.13 Angles connected by one leg

(1) In angles connected by one leg, the eccentricity of welded lap joint end connections may be allowed for by adopting an effective cross-sectional area and then treating the member as concentrically loaded.

(2) For an equal-leg angle, or an unequal-leg angle connected by its larger leg, the effective area may be taken as equal to the gross area.

(3) For an unequal-leg angle connected by its smaller leg, the effective area should be taken as equal to the gross cross-sectional area of an equivalent equal-leg angle of leg size equal to that of the smaller leg, when determining the design resistance of the cross-section, see EN 1993-1-1. However when determining the design buckling resistance of a compression member, see EN 1993-1-1, the actual gross cross-sectional area should be used.

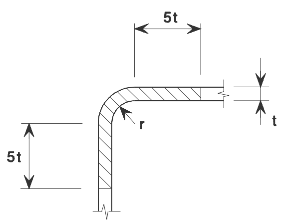
4.14 Welding in cold-formed zones

(1) Welding may be carried out in the cold-formed zones or within the adjacent width of $5t$ each side, see table 4.2, if one of the following conditions is fulfilled:

- the cold-formed zones are normalized after cold-forming but before welding;
- the r/t-ratio satisfy the relevant value obtained from table 4.2.

Table 4.2: Conditions for welding cold-formed zones and adjacent material

r/t	Strain due to cold forming (%)	Maximum thickness (mm)		
		Generally		Fully killed Aluminium-killed steel (Al ≥ 0,02 %)
		Predominantly static loading	Where fatigue predominates	
≥ 25	≥ 2	any	any	any
≥ 10	≥ 5	any	16	any
≥ 3,0	≥ 14	24	12	24
≥ 2,0	≥ 20	12	10	12
≥ 1,5	≥ 25	8	8	10
≥ 1,0	≥ 33	4	4	6



5 Analysis, classification and modelling

5.1 Global analysis

5.1.1 General

(1) The effects of the behaviour of the joints on the distribution of internal forces and moments within a structure, and on the overall deformations of the structure, should generally be taken into account, but where these effects are sufficiently small they may be neglected.

(2) To identify whether the effects of joint behaviour on the analysis need be taken into account, a distinction may be made between three simplified joint models as follows:

- simple, in which the joint may be assumed not to transmit bending moments;
- continuous, in which the behaviour of the joint may be assumed to have no effect on the analysis;
- semi-continuous, in which the behaviour of the joint needs to be taken into account in the analysis.

(3) The appropriate type of joint model should be determined from table 5.1, depending on the classification of the joint and on the chosen method of analysis.

(4) The design moment-rotation characteristic of a joint used in the analysis may be simplified by adopting any appropriate curve, including a linearised approximation (e.g. bi-linear or tri-linear), provided that the approximate curve lies wholly below the design moment-rotation characteristic. Exceptions to this rule, as those indicated in 5.1.2(4) and 5.1.4(5), may be contemplated as long as their validity is demonstrated.

Table 5.1: Type of joint model

Method of global analysis	Classification of joint		
Elastic	Nominally pinned	Rigid	Semi-rigid
Rigid-Plastic	Nominally pinned	Full-strength	Partial-strength
Elastic-Plastic	Nominally pinned	Rigid and full-strength	Semi-rigid and partial-strength Semi-rigid and full-strength Rigid and partial-strength
Type of joint model	Simple	Continuous	Semi-continuous

5.1.2 Elastic global analysis

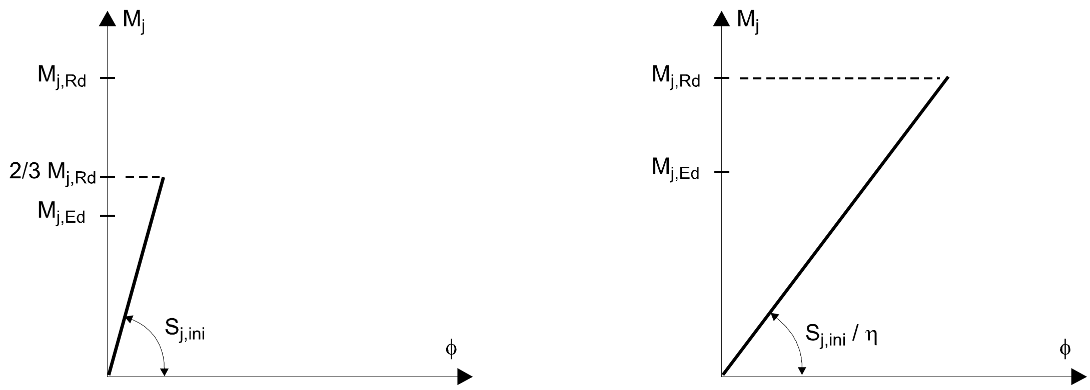
(1) The joints should be classified according to their rotational stiffness, see 5.2.2.

(2)P The strength of the joints shall be verified to be capable to transmit the forces and moments acting at the joints resulting from the analysis.

(3) In the case of a semi-rigid joint, the rotational stiffness S_{j} corresponding to the bending moment $M_{\mathrm{j,Ed}}$ should generally be used in the analysis. If $M_{\mathrm{j,Ed}}$ does not exceed 2/3 $M_{\mathrm{j,Rd}}$ the initial rotational stiffness $S_{\mathrm{j,ini}}$ may be taken in the global analysis, see figure 5.1(a).

(4) As a simplification to (3), the rotational stiffness may be taken as $S_{\mathrm{j,ini}}/\eta$ in the analysis, for all values of the moment $M_{\mathrm{j,Ed}}$, as shown in figure 5.1(b), where η is the stiffness modification coefficient from table 5.2.

(5) For joints connecting H or I sections S_{j} is given in 6.3.1.



a) $M_{j,Ed} \leq 2/3\ M_{j,Rd}$ b) $M_{j,Ed} \leq M_{j,Rd}$

Figure 5.1: Rotational stiffness to be used in elastic global analysis

Table 5.2: Stiffness modification coefficient η

Type of connection	Beam-to-column joints	Other types of joints (beam-to-beam joints, beam splices, column base joints)
Welded	2	3
Bolted end-plate	2	3
Bolted flange cleats	2	3,5
Base plates	-	3

5.1.3 Rigid-plastic global analysis

(1) The joints should be classified according to their strength, see 5.2.3.

(2) For joints connecting H or I sections $M_{j,Rd}$ is given in 6.2.

(3) For joints connecting hollow sections rules to determine the strength are given in section 7.

(4)P The rotation capacity of the joints shall be verified to be capable to accept the rotations of the joints resulting from the analysis.

(5) For joints connecting H or I sections the rotation capacity should be checked according to 6.4.

5.1.4 Elastic-plastic global analysis

(1) The joints should be classified according to both stiffness (see 5.2.2) and strength (see 5.2.3).

(2) For joints connecting H or I sections $M_{j,Rd}$ is given in 6.2, S_j is given in 6.3.1 and ϕ_{Cd} is given in 6.4.

(3) For joints connecting hollow sections rules to determine the strength are given in section 7.

(4) The moment rotation characteristic of the joints should be taken into account for the determination of the distribution of the internal forces and moments.

(5) As a simplification, the bi-linear design moment-rotation characteristic shown in figure 5.2 may be adopted. The stiffness modification coefficient η should be obtained from table 5.2.

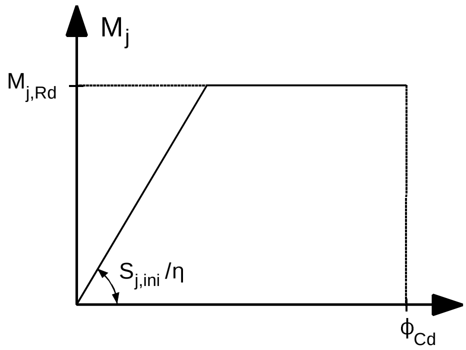


Figure 5.2: Simplified bi-linear design moment-rotation characteristic

5.1.5 Global analysis of lattice girders

(1) The distribution of axial forces in a lattice girder may be determined on the assumption that the members are connected by pinned joints (see also 2.7).

(2) The following provisions in this section apply for structure where the joints are verified according to section 7.

(3) Secondary moments at the joints, caused by the rotational stiffnesses of the joints, may be neglected both in the design of the members and in the design of the joints, provided that both of the following conditions are satisfied:

- the joint geometry is within the range of validity specified in tables 7.1, 7.8, 7.9 or 7.20 as appropriate;
- the ratio of the system length to the depth of the member in the plane of the lattice girder is not less than the appropriate minimum value. For building structures, the appropriate minimum value may be assumed to be 6. Larger values may apply in other parts of EN 1993.

(4) The moments resulting from transverse loads (whether in-plane or out-of-plane) that are applied between panel points, should be taken into account in the design of the members to which they are applied. Provided that the conditions given in (3) are satisfied:

- the brace members may be considered as pin-connected to the chords, so moments resulting from transverse loads applied to chord members need not be distributed into brace members, and vice versa;
- the chords may be considered as continuous beams, with simple supports at panel points.

(5) Moments resulting from eccentricities may be neglected in the design of tension chord members and brace members. They may also be neglected in the design of connections if the eccentricities are within the following limits:

- $-0{,}55\ d_0 \le e \le 0{,}25\ d_0$ ... (5.1a)
- $-0{,}55\ h_0 \le e \le 0{,}25\ h_0$ ... (5.1b)

where: e is the eccentricity defined in figure 5.3;

d_0 is the diameter of the chord;

h_0 is the depth of the chord, in the plane of the lattice girder.

(6) When the eccentricities are within the limits given in (5), the moments resulting from the eccentricities should be taken into account in the design of compression chord members. In this case the eccentricity moments should be distributed between the compression chord members on each side of the joint, on the basis of their relative stiffness coefficients I/L, where L is the system length of the member, measured between panel points.

(7) When the eccentricities are outside the limits given in (5), the moments resulting from the eccentricities should be taken into account in the design of the connections and the compression chord members. In this case the eccentricity moments should be distributed between all the members meeting at the joint, on the basis of their relative stiffness coefficients I/L.

(8) The stresses in a chord resulting from moments taken into account in the design of the chord, should also be taken into account in determining the factors k_m, k_n and k_p used in the design of the connections, see tables 7.2 to 7.5, 7.10 and 7.12 to 7.14.

(9) The cases where moments should be taken into account are summarized in table 5.3.

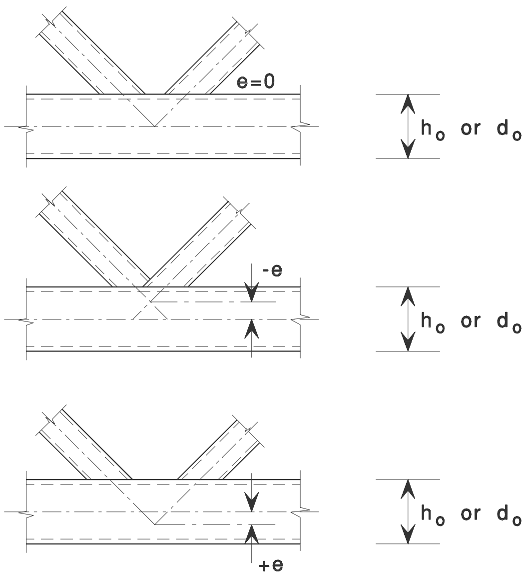


Figure 5.3: Eccentricity of joints.

Table 5.3: Allowance for bending moments

Type of component	Source of the bending moment		
	Secondary effects	Transverse loading	Eccentricity
Compression chord	Not if 5.1.5(3) is satisfied	Yes	Yes
Tension chord			No
Brace member			No
Connection			Not if 5.1.5(5) is satisfied

5.2 Classification of joints

5.2.1 General

(1)P The structural properties of all joints shall be such as to achieve the assumptions made in the analysis of the structure and in the design of the members.

(2) Joints may be classified by stiffness (see 5.2.2) and by strength (see 5.2.3).

5.2.2 Classification by stiffness

5.2.2.1 General

(1) A joint may be classified as rigid, nominally pinned or semi-rigid according to its rotational stiffness, by comparing its initial rotational stiffness $S_{j,ini}$ with classification boundaries, see 5.2.2.5.

> **NOTE:** Rules for the determination of $S_{j,ini}$ for joints connecting H or I sections are given in from 6.3.1. Rules for the determination of $S_{j,ini}$ for joints connecting hollow section are not given in this part.

(2) A joint may be classified on the basis of particular or general experimental evidence, or significant experience of previous satisfactory performance in similar cases or by calculations based on test evidence.

5.2.2.2 Nominally pinned joints

(1)P A nominally pinned joint shall be capable of transmitting the internal forces, without developing significant moments which might adversely affect members of the structure.

(2)P A nominally pinned joint shall be capable of accepting the resulting rotations under the design loads.

5.2.2.3 Rigid joints

(1) Joints classified as rigid may be assumend to have no significant influence on the distribution of internal forces and moments in the structure, nor on its overall deformation.

5.2.2.4 Semi-rigid joints

(1) A joint which does not meet the criteria for a rigid joint or a nominally pinned joint shall be classified as a semi-rigid joint.

> **NOTE:** Semi-rigid joints provide a predictable degree of interaction between members, based on the design moment-rotation characteristics of the joints.

(2) Semi-rigid joints should be capable of transmitting the internal forces and moments.

5.2.2.5 Classification boundaries

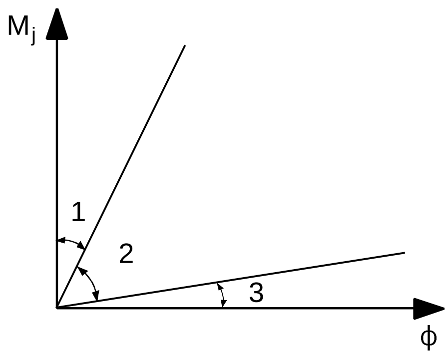
(1) Classification boundaries, see 5.2.2.1(1), for other joints than column bases are given in figure 5.4.

(2) Column bases may be classified as rigid:

- in frames where the bracing system reduces the horizontal desplacement by at least 80 % and where the effects of deformation may be neglected
  - if $\bar{\lambda}_o \leq 0{,}5$; ... (5.2a)
  - if $0{,}5 < \bar{\lambda}_o < 3{,}93$ and $S_{j,ini} \geq 7 (2 \bar{\lambda}_o - 1) EI_c / L_c$; ... (5.2b)
  - if $\bar{\lambda}_o \geq 3{,}93$ and $S_{j,ini} \geq 48 EI_c / L_c$. ... (5.2c)
- otherwise if $S_{j,ini} \geq 30 EI_c / L_c$. ... (5.2d)

where: $\bar{\lambda}_o$ is the slenderness of the column assumed as pinned at both end;

I_c, L_c are as in figure 5.4.



Zone 1: rigid, if $S_{j,ini} \geq k_b E I_b / L_b$

where $k_b = 8$ for frames where the bracing system reduces the horizontal desplacement by at least 80 %

$k_b = 25$ for other frames, provided that in every storey $K_b/K_c \geq 0{,}1$ *)

Zone 2: semi-rigid All joints in zone 2 should be classified as semi-rigid. Joints in zones 1 or 3 may optionally also be treated as semi-rigid.

Zone 3: nominally pinned, if $S_{j,ini} \leq 0{,}5 E I_b / L_b$

*) For frames where $K_b/K_c < 0{,}1$ the joints should be classified as semi-rigid.

Key:

K_b	is	the mean value of I_b/L_b for all the beams at the top of that storey;
K_c	is	the mean value of I_c/L_c for all the columns in that storey;
I_b	is	the second moment of area of a beam;
I_c	is	the second moment of area of a column;
L_b	is	the span of a beam (centre-to-centre of columns);
L_c	is	the storey height of a column.

Figure 5.4: Boundaries for stiffness classification of joints

5.2.3 Classification by strength

5.2.3.1 General

(1) A joint may be classified as full-strength, nominally pinned or partial strength by comparing its moment resistance $M_{j,Rd}$ with the moment resistances of the members that it joins. When classifying joints, the resistance of a member should be taken as that adjacent to the joint.

5.2.3.2 Nominally pinned joints

(1)P A nominally pinned joint shall be capable of transmitting the internal forces, without developing significant moments which might adversely affect members of the structure.

(2)P A nominally pinned joint shall be capable of accepting the resulting rotations under the design loads.

(3) A joint may be classified as nominally pinned if its moment resistance $M_{j,Rd}$ is not greater than 0,25 times the moment resistance required for a full-strength joint, provided that it also has sufficient rotation capacity.

5.2.3.3 Full-strength joints

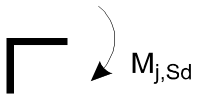
(1)P The design resistance of a full strength joint shall be not less than that of the members connected.

(2) A joint may be classified as full-strength if it meets the criteria given in figure 5.5.

5.2.3.4 Partial-strength joints

(1) A joint which does not meet the criteria for a full-strength joint or a nominally pinned joint should be classified as a partial-strength joint.

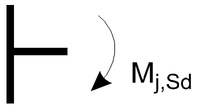
a) Top of column



Either $M_{j,Rd} \geq M_{b,p\ell,Rd}$

or $M_{j,Rd} \geq M_{c,p\ell,Rd}$

b) Within column height



Either $M_{j,Rd} \geq M_{b,p\ell,Rd}$

or $M_{j,Rd} \geq 2\,M_{c,p\ell,Rd}$

Key:

$M_{b,p\ell,Rd}$ is the plastic moment resistance of a beam;
$M_{c,p\ell,Rd}$ is the plastic moment resistance of a column.

Figure 5.5: Full-strength joints

5.3 Modelling of beam-to-column joints

(1) To model the deformational behaviour of a joint, account should be taken of the shear deformation of the web panel and the rotational deformation of the connections.

(2) Joint configurations should be designed to resist the internal bending moments $M_{b1,Ed}$ and $M_{b2,Ed}$, normal forces $N_{b1,Ed}$ and $N_{b2,Ed}$ and shear forces $V_{b1,Ed}$ and $V_{b2,Ed}$ applied to the connections by the connected beams, see figure 5.6.

(3) The resulting shear force $V_{wp,Ed}$ in the web panel should be obtained using:

$$V_{wp,Ed} = (M_{b1,Ed} - M_{b2,Ed})/z - (V_{c1,Ed} - V_{c2,Ed})/2 \qquad \text{... (5.3)}$$

where: z is the lever arm, see 6.2.5.

(4) To model a joint in a way that closely reproduces the expected behaviour, the web panel in shear and each of the connections should be modelled separately, taking account of the internal moments and forces in the members, acting at the periphery of the web panel, see figure 5.6(a) and figure 5.7.

NOTE: This type of modelling is not considered further in this section.

(5) As a simplified alternative to the method of modelling defined in (4), a single-sided joint configuration may be modelled as a single joint, and a double-sided joint configuration may be modelled as two separate but inter-acting joints, one on each side. As a consequence a double-sided beam-to-column joint configuration has two moment-rotation characteristics, one for the right-hand joint and another for the left-hand joint.

(6) Each of these joints should be modelled as a separate rotational spring, as shown in figure 5.8, in which each spring has a moment-rotation characteristic that takes into account the behaviour of the web panel in shear as well the influence of the relevant connection.

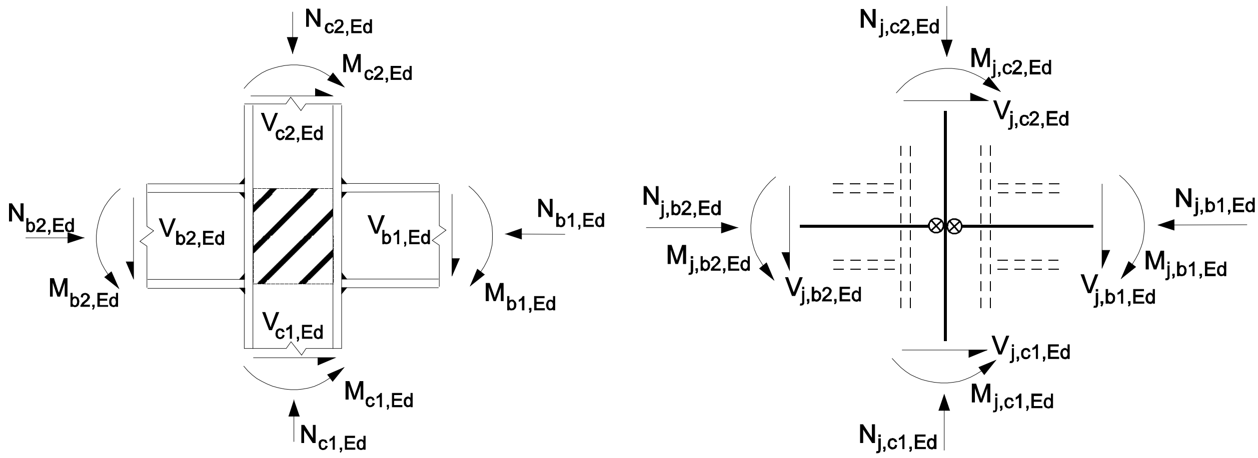
(7) When determining the moment resistance and rotational stiffness for each of the joints, the possible influence of the web panel in shear should be taken into account by means of the transformation parameters β_1 and β_2, where:

β_1 is the value of the transformation parameter β for the right-hand side joint;

β_2 is the value of the transformation parameter β for the left-hand side joint.

NOTE: The transformation parameters β_1 and β_2 are used directly in 6.2.5.2(7) and 6.3.2(1). They are also used in 6.2.4.2(4) and 6.2.4.3(4) in connection with table 6.3 to obtain the reduction factor ω for shear.

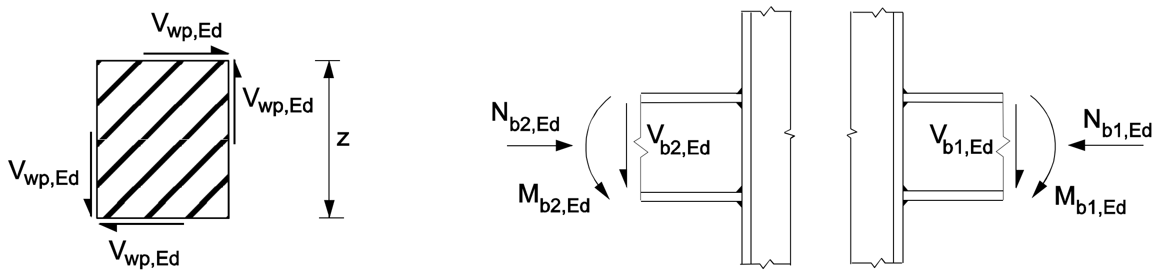
(8) Conservative values for β_1 and β_2 based on the values of the beam moments $M_{b1,Ed}$ and $M_{b2,Ed}$ at the periphery of the web panel, see figure 5.6(a), may be obtained from table 5.4.



a) Values at periphery of web panel   b) Values at intersection of member centrelines

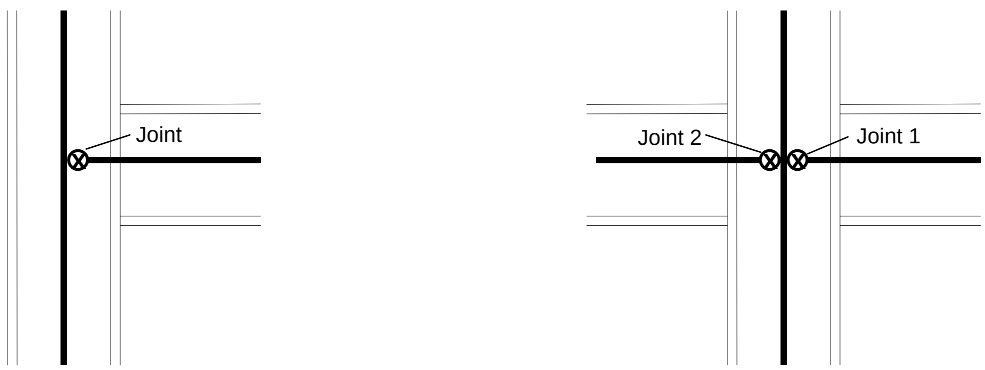
Direction of forces and moments are considered as positive in related equations (5.3) and (5.4)

Figure 5.6: Forces and moments acting on the joint



a) Shear forces in web panel   b) Connections, with forces and moments in beams

Figure 5.7: Forces and moments acting on the web panel at the connections



Single-sided joint configuration   Double-sided joint configuration

Figure 5.8: Modelling the joint

(9) Alternatively, more accurate values of β_1 and β_2 based on the values of the beam moments $M_{j,b1,Ed}$ and $M_{j,b2,Ed}$ at the intersection of the member centrelines, may be determined from the simplified model shown in figure 5.6(b) as follows:

$$\beta_1 = \left| 1 - M_{j,b2,Ed} / M_{j,b1,Ed} \right| \leq 2 \quad \text{... (5.4a)}$$

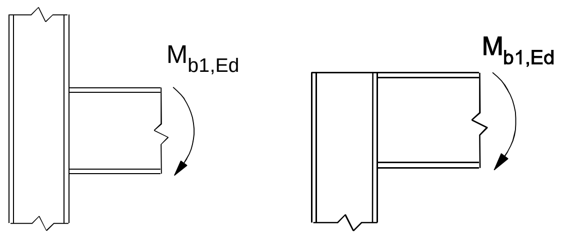
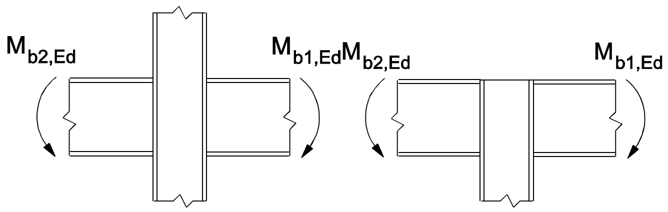
$$\beta_2 = \left| 1 - M_{j,b1,Ed} / M_{j,b2,Ed} \right| \leq 2 \quad \text{... (5.4b)}$$

where: $M_{j,b1,Ed}$ is the moment at the intersection from the right hand beam;

$M_{j,b2,Ed}$ is the moment at the intersection from the left hand beam.

(10) In the case of an unstiffened double-sided beam-to-column joint configuration in which the depths of the two beams are not equal, the actual distribution of shear stresses in the column web panel should be taken into account when determining the design moment resistance.

Table 5.4: Approximate values for the transformation parameter β

<table>
<tr><th>Type of joint configuration</th><th>Action</th><th>Value of β</th></tr>
<tr><td></td><td>$M_{b1,Ed}$</td><td>$\beta \approx 1$</td></tr>
<tr><td rowspan="4"></td><td>$M_{b1,Ed} = M_{b2,Ed}$</td><td>$\beta = 0$ *)</td></tr>
<tr><td>$M_{b1,Ed} / M_{b2,Ed} > 0$</td><td>$\beta \approx 1$</td></tr>
<tr><td>$M_{b1,Ed} / M_{b2,Ed} < 0$</td><td>$\beta \approx 2$</td></tr>
<tr><td>$M_{b1,Ed} + M_{b2,Ed} = 0$</td><td>$\beta \approx 2$</td></tr>
<tr><td colspan="3">*) In this case the value of β is the exact value rather than an approximation.</td></tr>
</table>

6 Structural joints connecting H or I sections

6.1 General

6.1.1 Basis

(1) This section contains design methods to determine the structural properties of joints in frames of any type. To apply these, a joint should be modelled as an assembly of basic components, see 1.3(1).

(2) For components identified in this section, see table 6.1, their properties should be determined in accordance with the provisions herein. For components not so identidfied, their properties should be based on test or analytical or numerical methods supported by tests, see EN 1990.

> **NOTE:** The design methods for basic joint components given in this section are of general application and can also be applied to similar components in other joint configurations. However the specific design methods given for determining the moment resistance, rotational stiffness and rotation capacity of a joint are based on an assumed distribution of internal forces for joint configurations indicated in figure 1.2 . For other joint configurations, design methods for determining the moment resistance, rotational stiffness and rotation capacity should be based on appropriate assumptions for the distribution of internal forces.

6.1.2 Structural properties

6.1.2.1 Design moment-rotation characteristic

(1) A joint may be represented by a rotational spring connecting the centre lines of the connected members at the point of intersection, as indicated in figure 6.1(a) and (b) for a single-sided beam-to-column joint configuration. The spring behaviour is expressed in the form of a design moment-rotation characteristic that describes the relationship between the bending moment $M_{j,Ed}$ applied to a joint and the corresponding rotation ϕ_{Ed} between the connected members. Generally the design moment-rotation characteristic is non-linear as indicated in figure 6.1(c).

(2) A design moment-rotation characteristic, see figure 6.1(c) should

define three main structural properties, as follows:

- moment resistance;
- rotational stiffness;
- rotation capacity.

> **NOTE:** In certain cases the moment-rotation behaviour of a joint includes some initial rotation due to bolt slip or lack of fit. This can result in a significant amount of initial hinge rotation that may need to be included in the design moment-rotation characteristic.

(3)P The design moment-rotation characteristics of a beam-to-column joint shall be consistent with the assumptions made in the global analysis of the structure and with the assumptions made in the design of the members, see EN 1993-1-1.

6.1.2.2 Moment resistance

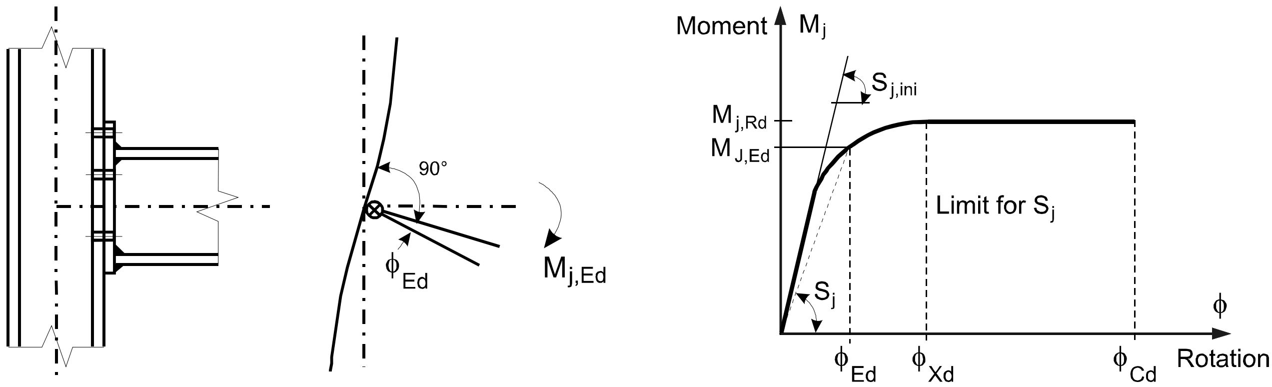
(1) The design moment resistance $M_{j,Rd}$ is equal to the maximum moment of the design moment-rotation characteristic, see figure 6.1(c).

6.1.2.3 Rotational stiffness

(1) The rotational stiffness S_j should be taken as the secant stiffness as indicated in figure 6.1(c). For a design moment-rotation characteristic this definition of S_j applies up to the rotation ϕ_{Xd} at which $M_{j,Ed}$ first reaches $M_{j,Rd}$, but not for larger rotations, see figure 6.1(c). The initial rotational stiffness $S_{j,ini}$ is the slope of the elastic range of the design moment-rotation characteristic.

6.1.2.4 Rotation capacity

(1) The design rotation capacity ϕ_{Cd} of a joint is the maximum rotation of the design moment-rotation characteristic, see figure 6.1(c).



a) Joint b) Model c) Design moment-rotation characteristic

Figure 6.1: Design moment-rotation characteristic for a joint

6.1.3 Basic components of a joint

(1) The design moment-rotation characteristic of a joint depends on the properties of its basic components.

(2) The basic joint components identified in this section are given in table 6.1, together with the reference to the application rules for the evaluation of their structural properties.

(3) Certain joint components may be reinforced. Details are given in 6.2.2.3 and 6.2.4.

(4) Relationships between the properties of the basic components of a joint and the structural properties of the joint are given:

- for moment resistance in 6.2.5;
- for rotational stiffness in 6.3.1;
- for rotation capacity in 6.4.

Table 6.1: Basic joint components

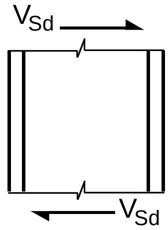
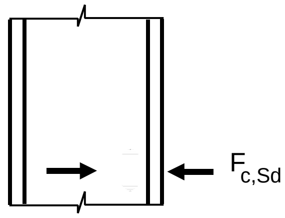
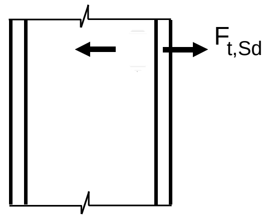
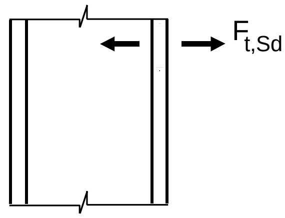
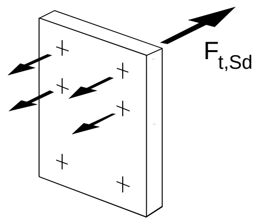
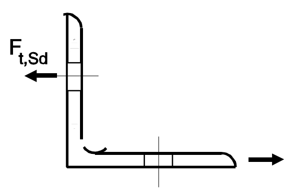
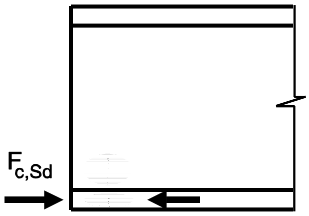
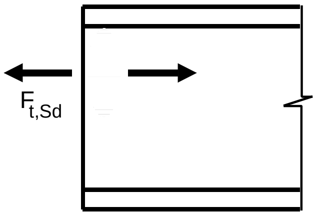
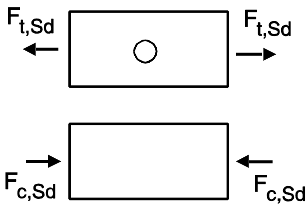
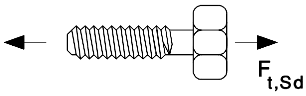
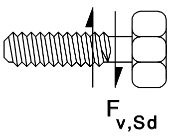
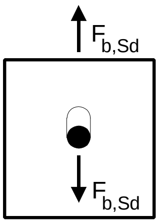
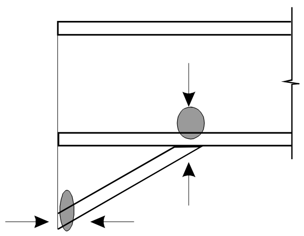
Component			Reference to application rules		
			Resistance	Stiffness coefficient	Rotation capacity
1	Column web panel in shear		6.2.4.1	6.3.2	6.4(4)
2	Column web in transverse compression		6.2.4.2	6.3.2	6.4(5) and 6.4(6)
3	Column web in transverse tension		6.2.4.3	6.3.2	6.4(5)
4	Column flange in bending		6.2.4.4	6.3.2	6.4(7)
5	End-plate in bending		6.2.4.5	6.3.2	6.4(7)
6	Flange cleat in bending		6.2.4.6	6.3.2	6.4(7)

Table 6.1: Basic joint components (continued)

Component			Reference to application rules		
			Resistance	Stiffness coefficient	Rotation capacity
7	Beam or column flange and web in compression		6.2.4.7	6.3.2	*)
8	Beam web in tension		6.2.4.8	6.3.2	*)
9	Plate in tension or compression		in tension: - EN 1993-1-1 in compression: - EN 1993-1-1	6.3.2	*)
10	Bolts in tension		with column flange: - 6.2.4.4 with end-plate: - 6.2.4.5 with flange cleat: - 6.2.4.6	6.3.2	6.4(7)
11	Bolts in shear		3.6	6.3.2	6.4(2)
12	Bolts in bearing (on beam flange, column flange, end-plate or cleat)		3.6	6.3.2	*)

*) No information available in this part.

Table 6.1: Basic joint components (continued)

Component			Reference to application rules		
			Resistance	Stiffness coefficient	Rotation capacity
13	Concrete in compression including grout		6.2.4.9	6.3.2	*)
14	Base plate in bending under compression		6.2.4.10		*)
15	Base plate in bending under tension		6.2.4.11	6.3.2	*)
16	Anchor bolts in tension		6.2.4.12	6.3.2	*)
17	Anchor bolts in shear		6.2.1.2	6.3.2	*)
18	Anchor bolts in bearing		6.2.1.2	6.3.2	*)
19	Welds		4	6.3.2	*)
20	Haunched beam		6.2.4.7	6.3.2	*)

*) No information available in this part.

6.2 Resistance

6.2.1 General

6.2.1.1 Internal forces

(1) The stresses due to the internal forces and moments in the members may be assumed not to affect the resistances of the basic components of a joint, except as specified in (2) and (3).

(2) The longitudinal stress in a column should be taken into account when determining the resistance of the column web in compression, see 6.2.4.2(2).

(3) The shear in a column web panel should be taken into account when determining the resistance of the following basic components:

- column web in transverse compression, see 6.2.4.2;
- column web in transverse tension, see 6.2.4.3.

6.2.1.2 Resistance to shear forces

(1) In welded connections, and in bolted connections with end-plates, the welds connecting the beam web should be designed to transfer the shear force from the connected beam to the joint, without any assistance from the welds connecting the beam flanges.

(2) In bolted connections with end-plates, the resistance of each bolt-row to combined shear and tension should be verified with the criterion given in table 3.4, taking into account the total tensile force in the bolt, including any force due to prying action.

NOTE : As a simplification it may be verified that the shear force transferred by the bolts does not exceed the sum of:

a) the total shear resistance of those bolts that are not required to resist tension;

b) (0,4/1,4) times the total shear resistance of those bolts that are also required to resist tension.

(3) In bolted connections with angle flange cleats, the cleat connecting the compression flange of the beam may be assumed to transfer the shear force in the beam to the column, provided that:

- the gap g between the end of the beam and the face of the column does not exceed the thickness t_a of the angle cleat;
- the force does not exceed the shear resistance of the bolts connecting the cleat to the column;
- the web of the beam satisfies requirement in EN 1993-1-1.

(4) The shear resistance of a joint may be derived from the distribution of internal forces within that joint, and the resistances of its basic components to these forces, see table 6.1.

(5) If no special elements for resisting shear are provided, such as block or bar shear connectors, it should be demonstrated that either the friction resistance of the base plate, see (6), or in case where the bolt holes are not oversized the shear resistance of the anchor bolts, see (7), is sufficient to transfer the design shear force. The bearing resistance of the block or bar shear connectors with respect to the concrete should be checked according to EN 1992-1.

(6) In a column base the design friction resistance $F_{f,Rd}$ between base plate and grout should be derived as follows:

$$F_{f,Rd} = C_{f,d} \; N_{c,Ed} \qquad \text{... (6.1)}$$

where: $C_{f,d}$ is the coefficient of friction between base plate and grout layer. The following values may be used:

- for sand-cement mortar $C_{f,d} = 0{,}20$
- for special grout $C_{f,d} = 0{,}30$

$N_{c,Ed}$ is the design value of the normal compressive force in the column.

(7) In a column base design shear resistance of an anchor bolt $F_{vb,Rd}$ should be taken as the smaller of $F_{1,vb,Rd}$ and $F_{2,vb,Rd}$ where

- $F_{1,vb,Rd}$ is the bearing resistance of the anchor bolt, see 3.6.1
- $$F_{2,vb,Rd} = \frac{\alpha_b\, f_{ub}\, A_s}{\gamma_{Mb}} \quad \text{... (6.2)}$$

where: $\alpha_b = 0{,}44 - 0{,}0003\, f_{yb}$

f_{yb} is the yield strength of the anchor bolt, where $235\ \text{N/mm}^2 \leq f_{yb} \leq 640\ \text{N/mm}^2$

NOTE: If the column is loaded by a tensile normal force, $F_{f,Rd} = 0$.

(8) In a column base with a flat base plate the design shear resistance $F_{v,Rd}$ should be derived as follows:

$$F_{v,Rd} = F_{f,Rd} + n\, F_{vb,Rd} \quad \text{... (6.3)}$$

where: n is the number of anchor bolts in the base plate.

(9) For the design of concrete and reinforcement reference is made to EN 1992.

6.2.1.3 Resistance to bending moments

(1) The moment resistance of any joint may be derived from the distribution of internal forces within that joint and the resistances of its basic components to these forces, see table 6.1.

(2) Provided that the axial force N_{Ed} in the connected member does not exceed 5% of the resistance $N_{p\ell,Rd}$ of its cross-section, the moment resistance $M_{j,Rd}$ of a beam-to column joint or beam splice may be determined using the method given in 6.2.5.

(3) The moment resistance $M_{j,Rd}$ of a column base may be determined using the method given in 6.2.6.

(4) In all joints, the sizes of the welds should be such that the moment resistance of the joint $M_{j,Rd}$ is always limited by the resistance of its other basic components, and not by the resistance of the welds.

(5) In a beam-to-column joint or beam splice in which a plastic hinge is required to form and rotate under any relevant load case, the welds should be designed to resist the effects of a moment equal to the smaller of:

- the plastic moment resistance of the connected member $M_{p\ell,Rd}$
- α times the moment resistance of the joint $M_{j,Rd}$

  where for frames where the bracing system reduces the horizontal desplacement by at least 80 % $\alpha = 1{,}4$ otherwise $\alpha = 1{,}7$.

(6) In a bolted connection with more than one bolt-row in tension, as a simplification the contribution of any bolt-row may be neglected, provided that the contributions of all other bolt-rows closer to the centre of compression are also neglected.

NOTE: This leads to an under-estimate of the moment resistance, but generally the reduction is relatively small.

6.2.2 Equivalent T-stub in tension

6.2.2.1 General

(1) In bolted connections an equivalent T-stub in tension may be used to model the resistance of the following basic components:

- column flange in bending;
- end-plate in bending;
- flange cleat in bending.
- base plate in bending under tension.

(2) Methods for modelling these basic components as equivalent T-stub flanges, including the values to be used for e_{min}, ℓ_{eff} and m, are given in 6.2.4.

(3) The possible modes of failure of the flange of an equivalent T-stub may be assumed to be similar to those expected to occur in the basic component that it represents.

(4) The total effective length $\Sigma\ell_{eff}$ of an equivalent T-stub, see figure 6.2, should be such that the resistance of its flange is equivalent to that of the basic joint component that it represents.

NOTE: The effective length of an equivalent T-stub is a notional length and does not necessarily correspond to the physical length of the basic joint component that it represents.

(5) The tension resistance of a T-stub flange should be determined from table 6.2.

NOTE: Prying effects are implicitly taken into account when determining the tension resistance according to table 6.2 .

(6) In cases where prying forces may develop, see table 6.2 , the tension resistance of a T-stub flange $F_{T,Rd}$ should be taken as the smallest value for the three possible failure modes 1, 2 and 3.

(7) In cases where prying forces may not develop, see table 6.2 , the tension resistance of a T-stub flange $F_{T,Rd}$ should be taken as the smallest value for the two possible failure modes 12 and 3.

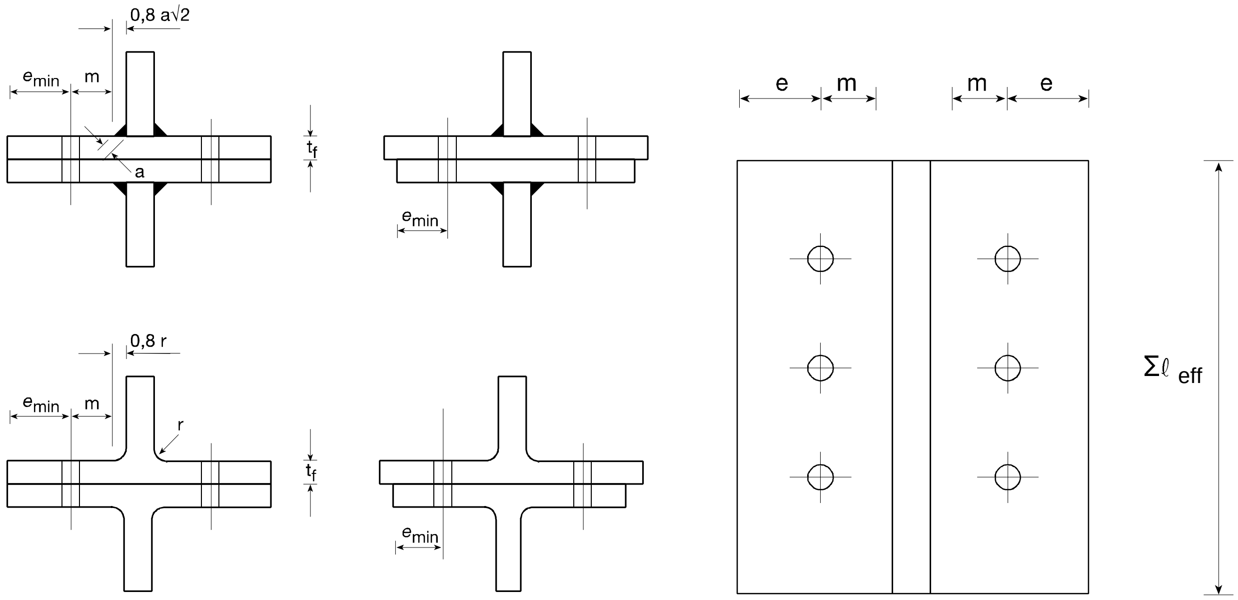


Figure 6.2: Dimensions of an equivalent T-stub flange

Table 6.2: Resistance of a T-stub flange

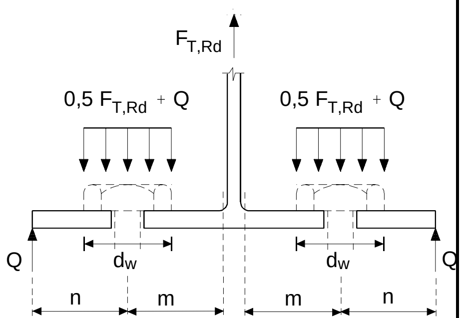
	Prying forces may develop, i.e. $L_b \le L_b^*$		No prying forces
Mode 1	Method 1	Method 2 (alternative method)	
without backing plates	$F_{T,1,Rd} = \dfrac{4M_{p\ell,1,Rd}}{m}$	$F_{T,1,Rd} = \dfrac{(8n - 2e_w)M_{p\ell,1,Rd}}{2mn - e_w(m+n)}$	$F_{T,12,Rd} = \dfrac{2M_{p\ell,1,Rd}}{m}$
with backing plates	$F_{T,1,Rd} = \dfrac{4M_{p\ell,1,Rd} + 2M_{bp,Rd}}{m}$	$F_{T,1,Rd} = \dfrac{(8n - 2e_w)M_{p\ell,1,Rd} + 4nM_{bp,Rd}}{2mn - e_w(m+n)}$	
Mode 2	$F_{T,2,Rd} = \dfrac{2M_{p\ell,2,Rd} + n\Sigma F_{t,Rd}}{m+n}$		
Mode 3	$F_{T,3,Rd} = \Sigma F_{t,Rd}$		

Mode 1: Complete yielding of the flange
Mode 2: Bolt failure with yielding of the flange
Mode 3: Bolt failure

L_b is
- the bolt elongation length, taken as equal to the grip length (total thickness of material and washers), plus half the sum of the height of the bolt head and the height of the nut or
- the anchor bolt elongation length, taken as equal to the sum of 8 times the nominal bolt diameter, the grout layer, the plate thickness, the washer and half of the height of the nut

$L_b^* = \dfrac{8{,}8\, m^3 A_s}{\Sigma \ell_{eff,1}\, t_f^3}$

$F_{T,Rd}$ is the tension resistance of a T-stub flange
Q is is the prying force
$M_{p\ell,1,Rd} = 0{,}25 \Sigma \ell_{eff,1}\, t_f^2 f_y / \gamma_{M0}$
$M_{p\ell,2,Rd} = 0{,}25 \Sigma \ell_{eff,2}\, t_f^2 f_y / \gamma_{M0}$
$M_{bp,Rd} = 0{,}25\, \Sigma \ell_{eff,1}\, t_{bp}^{\;2} f_{y,bp} / \gamma_{M0}$
$n = e_{min}$ but $n \le 1{,}25m$
$F_{t,Rd}$ is the tension resistance of a bolt, see table 3.4;
$\Sigma F_{t,Rd}$ is the total value of $F_{t,Rd}$ for all the bolts in the T-stub;
$\Sigma \ell_{eff,1}$ is the value of $\Sigma \ell_{eff}$ for mode 1;
$\Sigma \ell_{eff,2}$ is the value of $\Sigma \ell_{eff}$ for mode 2;
e_{min} , m and t_f are as indicated in figure 6.2.
$f_{y,bp}$ is the yield strength of the backing plates;
t_{bp} is the thickness of the backing plates;
$e_w = d_w / 4$;
d_w is the diameter of the washer, or the width across points of the bolt head or nut, as relevant.



NOTE 1: In bolted beam-to-column joints or beam splices it may be assumed that prying forces will develop.
NOTE 2: In method 2, the force applied to the T-stub flange by a bolt should be assumed to be uniformly distributed under the washer, the bolt head or the nut, as appropriate, see figure, instead of concentrated at the centre-line of the bolt. This assumption leads to a higher value of the resistance for mode 1, but leaves the values for $F_{T,12,Rd}$ and modes 2 and 3 unchanged.

6.2.2.2 Individual bolt-rows, bolt-groups and groups of bolt-rows

(1) Although in an actual T-stub flange the forces at each bolt-row are generally equal, when an equivalent T-stub flange is used to model a basic component listed in 6.2.2.1(1), allowance should be made for the forces are generally different at each bolt-row.

(2) When modelling a basic joint component by equivalent T-stub flanges in accordance with 6.2.4, if necessary more than one equivalent T-stub may be used, with the bolt-rows divided into separate bolt-groups corresponding to each equivalent T-stub flange.

(3) The following conditions should be satisfied:

a) the force at each bolt-row should not exceed the resistance determined considering only that individual bolt-row;

b) the total force on each group of bolt-rows, comprising two or more adjacent bolt-rows within the same bolt-group, should not exceed the resistance of that group of bolt-rows.

(4) Accordingly, when obtaining the tension resistance of the basic component represented by an equivalent T-stub flange, the following parameters should generally be determined:

a) the maximum resistance of an individual bolt-row, determined considering only that bolt-row;

b) the contribution of each bolt-row to the maximum resistance of two or more adjacent bolt-rows within a bolt-group, determined considering only those bolt-rows.

(5) In the case of an individual bolt-row $\Sigma\ell_{eff}$ should be taken as equal to the effective length ℓ_{eff} tabulated in 6.2.4 for that bolt-row as an individual bolt-row.

(6) In the case of a group of bolt-rows $\Sigma\ell_{eff}$ should be taken as the sum of the effective lengths ℓ_{eff} tabulated in 6.2.4 for each relevant bolt-row as part of a bolt-group.

6.2.2.3 Backing plates

(1) Column flanges in bending may be reinforced by backing plates as indicated in figure 6.3.

(2) Each backing plate should extend at least to the edge of the column flange, and to within 3 mm of the toe of the root radius or fillet weld.

(3) The backing plate should at least extend beyond the furthermost bolt rows active in tension as defined in figure 6.3.

(4) Where backing plates are used, the resistance of the T-stub $F_{T,Rd}$ should be determined as given in table 6.2.

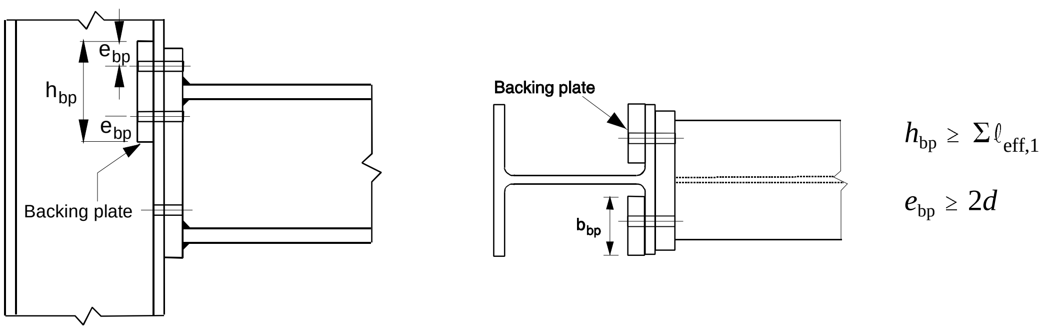


Figure 6.3: Column flange with backing plates

6.2.3 Equivalent T-stub in compression

(1) In steel- to-concrete connections, the flange of an equivalent T-stub in compression may be used to model the resistance of the basic components:

- base plate in bending under compression in combination with
- concrete in compression including grout.

(2) The total length l_{eff} and the total width b_{eff} of an equivalent T-stub should be such that the compression resistance of the T-stub is equivalent to that of the basic joint component it represents.

NOTE: The effective length and the effective width of an equivalent T-stub are a notional lengths and are smaller than or equal to the physical dimensions of the basic joint component it represents.

(3) The compression resistance of a T-stub flange $F_{C,Rd}$ should be determined as follows:

$$F_{C,Rd} = f_{jd}\, b_{eff}\, l_{eff} \qquad \text{... (6.4)}$$

where: b_{eff} is the effective width of the T-stub flange, see (5) and (6)

l_{eff} is the effective length of the T-stub flange, see (5) and (6)

f_{jd} is the design bearing strength of the joint, see (7)

(4) The forces transferred through a T-stub should be assumed to spread uniformly as shown in figure 6.4(a) and (b). The pressure on the resulting bearing area should not exceed the design bearing strength f_j and the additional bearing width should not exceed:

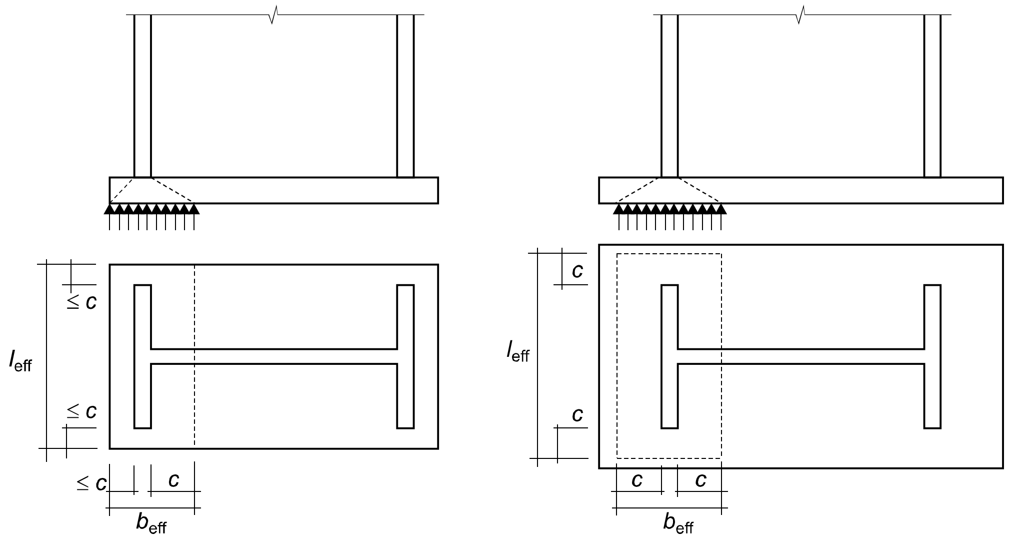
$$c = t\,[f_y / (3\, f_j\, \gamma_{M0})]^{0.5} \qquad \text{... (6.5)}$$

where: t is the thickness of the T-stub flange;

f_y is the yield strength of the T-stub flange.

(5) Where the projection of the physical length of the basic joint component represented by the T-stub is less than c, the effective area should be assumed as indicated in figure 6.4(a)

(6) Where the projection of the physical length of the basic joint component represented by the T-stub exceeds c, the additional projection should be neglected, see figure 6.4(b).



(a) Short projection (b) Large projection

Figure 6.4: Area of equivalent T-Stub in compression

(7) The design bearing strength of the joint f_{jd} should be determined from:

$$f_{jd} = \beta_j F_{Rdu} / (b_{eff}\, l_{eff}) \qquad \text{... (6.6)}$$

where: β_j is the joint coefficient, which may be taken as 2/3 provided that the characteristic strength of the grout is not less than 0,2 times the characteristic strength of the concrete foundation and the thickness of the grout is not greater than 0,2 times the smallest width of the steel base plate. In case that the thickness of the grout is more than 50 mm, the characteristic strength of the grout should be at least the same as that of the concrete foundation.

F_{Rdu} is the concentrated resistance force given in EN 1992, where A_{c0} is to be taken as $(b_{eff}\, l_{eff})$.

6.2.4 Resistance of basic components

6.2.4.1 Column web panel in shear

(1) The design methods are valid if $d/t_w \leq 69\varepsilon$.

(2) For a single-sided joint, or for a double-sided joint in which the beam depths are similar, the shear resistance $V_{wp,Rd}$ of an unstiffened column web panel, subject to a design shear force $V_{wp,Ed}$, see 5.3(3), should be obtained using:

$$V_{wp,Rd} = \frac{0{,}9\, f_{y,wc}\, A_{vc}}{\sqrt{3}\, \gamma_{M0}} \qquad \text{... (6.7)}$$

where: A_{vc} is the shear area of the column, see EN 1993-1-1.

(3) The shear resistance may be increased by the use of stiffeners or supplementary web plates.

(4) Where transverse web stiffeners are used in both the compression zone and the tension zone, the plastic shear resistance of the column web panel $V_{wp,Rd}$ may be increased by $V_{wp,add,Rd}$ given by:

$$V_{wp,add,Rd} = \frac{4\, M_{p\ell,fc,Rd}}{d_s} \quad \text{but} \quad V_{wp,add,Rd} \leq \frac{2\, M_{p\ell,fc,Rd} + 2\, M_{p\ell,st,Rd}}{d_s} \qquad \text{... (6.8)}$$

where: d_s is the distance between the centrelines of the stiffeners;

$M_{p\ell,fc,Rd}$ is the plastic moment resistance of a column flange

$M_{p\ell,st,Rd}$ is the plastic moment resistance of a stiffener.

NOTE: In welded joints, the transverse stiffeners should be aligned with the corresponding beam flange.

(5) When diagonal web stiffeners are used the shear resistance of a column web should be determined according to EN 1993-1-1.

NOTE: In double-sided beam-to-column joint configurations without diagonal stiffeners on the column webs, the two beams are assumed to have similar depths.

(6) Where a column web is reinforced by adding a supplementary web plate, see figure 6.5, the shear area A_{vc} may be increased by $b_s\, t_{wc}$. If a further supplementary web plate is added on the other side of the web, no further increase of the shear area should be made.

(7) Supplementary web plates may also be used to increase the rotational stiffness of a joint by increasing the stiffness of the column web in shear, compression or tension, see 6.3.2(1).

(8) The steel grade of the supplementary web plate should be similar to that of the column.

(9) The width b_s should be such that the supplementary web plate extend at least to the toe of the root radius.

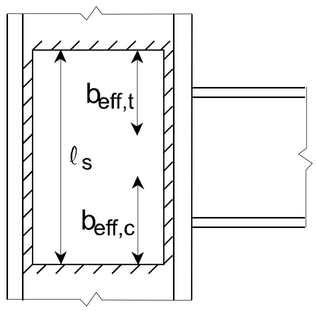
(10) The length ℓ_s should be such that the supplementary web plate extends throughout the effective width of the web in tension and compression, see figure 6.5.

(11) The thickness t_s of the supplementary web plate should be not less than the column web thickness t_{wc}.

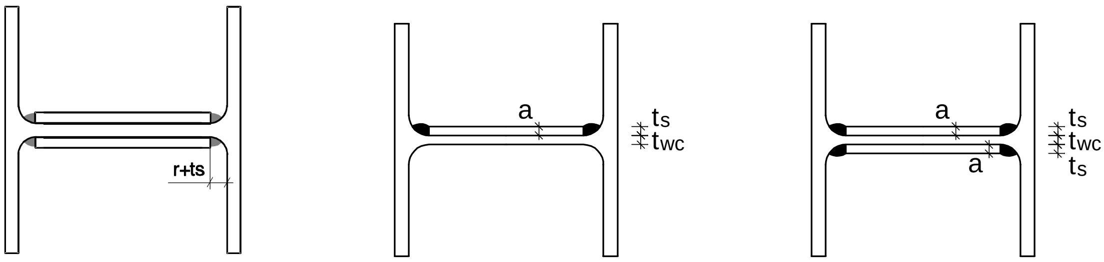
(12) The welds between the supplementary web plate and profile should be design to the acting design forces.

(13) The width b_s of a supplementary web plate should be less than $40\varepsilon\, t_s$.

(14) Discontinuous welds may be used in non corrosive environments.



a) Layout



NOTE: Weldability at the corner should be taken into account.

b) Examples of cross-section with longitudinal welds

Figure 6.5: Examples of supplementary web plates

6.2.4.2 Column web in transverse compression

(1) The resistance of an unstiffened column web subject to transverse compression should be determined from:

$$F_{c,wc,Rd} = \frac{\omega\, k_{wc}\, b_{eff,c,wc}\, t_{wc}\, f_{y,wc}}{\gamma_{M0}} \quad \text{but} \quad F_{c,wc,Rd} \leq \frac{\omega\, k_{wc}\, \rho\, b_{eff,c,wc}\, t_{wc}\, f_{y,wc}}{\gamma_{M1}} \qquad \text{... (6.9)}$$

where: ω is a reduction factor to allow for the possible effects of interaction with shear in the column web panel according to table 6.3 ;

$b_{eff,c,wc}$ is the effective width of column web in compression

- for a welded connection:

$$b_{eff,c,wc} = t_{fb} + 2\sqrt{2}\, a_b + 5(t_{fc} + s) \qquad \text{... (6.10)}$$

a_c and r_c are as indicated in figure 6.8 and a_b is as indicated in figure 6.6.

- for bolted end-plate connection:

$$b_{\mathrm{eff,c,wc}} = t_{\mathrm{fb}} + 2\sqrt{2}\,a_{\mathrm{p}} + 5(t_{\mathrm{fc}} + s) + s_{\mathrm{p}} \quad \text{... (6.11)}$$

s_{p} is the length obtained by dispersion at 45° through the end-plate (at least t_{p} and, provided that the length of end-plate below the flange is sufficient, up to $2t_{\mathrm{p}}$).

- for bolted connection with angle flange cleats:

$$b_{\mathrm{eff,c,wc}} = 2t_{\mathrm{a}} + 0{,}6r_{\mathrm{a}} + 5(t_{\mathrm{fc}} + s) \quad \text{... (6.12)}$$

  - for a rolled I or H section column: $s = r_{\mathrm{c}}$
  - for a welded I or H section column: $s = \sqrt{2}\,a_{\mathrm{c}}$

ρ is the reduction factor for plate buckling:

- if $\bar{\lambda}_{\mathrm{p}} \leq 0{,}72$: $\rho = 1{,}0$ ... (6.13a)
- if $\bar{\lambda}_{\mathrm{p}} > 0{,}72$: $\rho = (\bar{\lambda}_{\mathrm{p}} - 0{,}2)/\bar{\lambda}_{\mathrm{p}}^{\,2}$ ... (6.13b)

$\bar{\lambda}_{\mathrm{p}}$ is the plate slenderness:

$$\bar{\lambda}_{\mathrm{p}} = 0{,}932\sqrt{\frac{b_{\mathrm{eff,c,wc}}\,d_{\mathrm{wc}}\,f_{\mathrm{y,wc}}}{E\,t_{\mathrm{wc}}^{\,2}}} \quad \text{... (6.13c)}$$

- for a rolled I or H section column: $d_{\mathrm{wc}} = h_{\mathrm{c}} - 2(t_{\mathrm{fc}} + r_{\mathrm{c}})$
- for a welded I or H section column: $d_{\mathrm{wc}} = h_{\mathrm{c}} - 2(t_{\mathrm{fc}} + \sqrt{2}\,a_{\mathrm{c}})$

k_{wc} see (2).

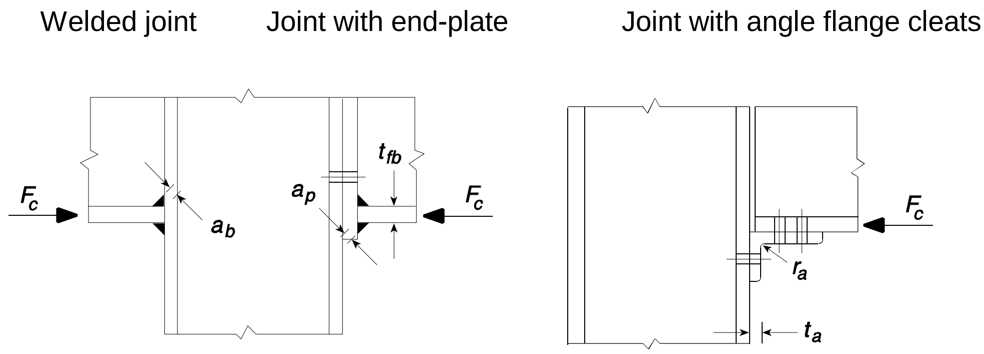
Table 6.3: Reduction factor ω for interaction with shear

Transformation parameter β	Reduction factor ω
$0 \leq \beta \leq 0{,}5$	$\omega = 1$
$0{,}5 < \beta < 1$	$\omega = \omega_1 + 2(1-\beta)(1-\omega_1)$
$\beta = 1$	$\omega = \omega_1$
$1 < \beta < 2$	$\omega = \omega_1 + (\beta - 1)(\omega_2 - \omega_1)$
$\beta = 2$	$\omega = \omega_2$
$\omega_1 = \dfrac{1}{\sqrt{1 + 1{,}3\left(b_{\mathrm{eff,c,wc}}\,t_{\mathrm{wc}}/A_{\mathrm{vc}}\right)^2}}$	$\omega_2 = \dfrac{1}{\sqrt{1 + 5{,}2\left(b_{\mathrm{eff,c,wc}}\,t_{\mathrm{wc}}/A_{\mathrm{vc}}\right)^2}}$
A_{vc} is the shear area of the column, see 6.2.4.1; β is the transformation parameter, see 5.3(7).	

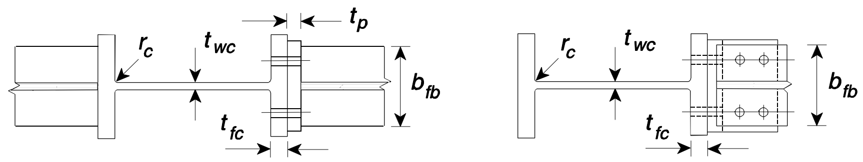
(2) Where the maximum longitudinal compressive stress $\sigma_{\mathrm{com,Ed}}$ due to axial force and bending moment in the column exceeds $0{,}7f_{\mathrm{y,wc}}$ in the web (adjacent to the root radius for a rolled section or the toe of the weld for a welded section), its effect on the resistance of the column web in compression should be allowed for by multiplying the value of $F_{\mathrm{c,wc,Rd}}$ given by expression (6.9) by a reduction factor k_{wc} as follows:

- when $\sigma_{\mathrm{com,Ed}} \leq 0{,}7f_{\mathrm{y,wc}}$: $k_{\mathrm{wc}} = 1$
- when $\sigma_{\mathrm{com,Ed}} > 0{,}7f_{\mathrm{y,wc}}$: $k_{\mathrm{wc}} = 1{,}7 - \sigma_{com,Ed}/f_{y,wc}$ ... (6.14)

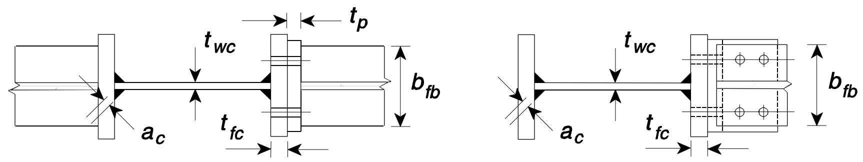
NOTE: Generally the reduction factor k_{wc} is 1,0 and no reduction is necessary. It can therefore be omitted in preliminary calculations when the longitudinal stress is unknown and checked later.



a) Elevation



b) Rolled column



c) Welded column

Figure 6.6: Transverse compression on an unstiffened column

(3) The ‘column-sway' buckling mode of an unstiffened column web in compression illustrated in figure 6.7 should normally be prevented by constructional restraints.

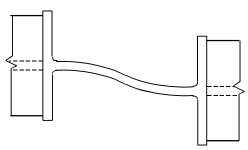


Figure 6.7: ‘Column-sway’ buckling mode of an unstiffened web

(4) The resistance may be increased by the use of stiffeners or supplementary web plates.

(5) Transverse stiffeners or appropriate arrangements of diagonal stiffeners may be used in association with, or as an alternative to, transverse stiffeners in order to increase the resistance of the column web in compression.

NOTE: In welded joints, the transverse stiffeners should be aligned with the corresponding beam flange. In bolted joints, the stiffener in the compression zone should be aligned with the centre of compression as defined figure 6.15.

(6) Where an unstiffened column web is reinforced by adding a supplementary web plate conforming with 6.2.4.1, the effective thickness of the web may be taken as $1{,}5\,t_{wc}$ if one supplementary web plate is added, or $2{,}0\,t_{wc}$ if supplementary web plates are added both sides of the web. In calculating the reduction factor ω for the possible effects of shear stress, the shear area A_{vc} of the web may be increased only to the extent permitted when determining its shear resistance, see 6.2.4.1(6).

6.2.4.3 Column web in transverse tension

(1) The resistance of an unstiffened column web subject to transverse tension should be determined from:

$$F_{t,wc,Rd} = \frac{\omega\ b_{eff,t,wc}\ t_{wc}\ f_{y,wc}}{\gamma_{M0}} \qquad \text{... (6.15)}$$

where: ω is a reduction factor to allow for the possible effects of interaction with shear in the column web panel.

(2) For a welded connection, the effective width $b_{eff,t,wc}$ of column web in tension should be obtained using:

$$b_{eff,t,wc} = t_{fb} + 2\sqrt{2}\,a_b + 5(t_{fc} + s) \qquad \text{... (6.16)}$$

where:

- for a rolled I or H section column: $s = r_c$
- for a welded I or H section column: $s = \sqrt{2}\,a_c$

where: a_c and r_c are as indicated in figure 6.8 and a_b is as indicated in figure 6.6.

(3) For a bolted connection, the effective width $b_{eff,t,wc}$ of column web in tension should be taken as equal to the effective length of equivalent T-stub representing the column flange, see 6.2.4.4.

(4) The reduction factor ω to allow for the possible effects of shear in the column web panel should be determined from table 6.3, using the value of $b_{eff,t,wc}$ given in (2) or (3) as appropriate.

(5) The shear resistance may be increased by the use of stiffeners or supplementary web plates.

(6) Transverse stiffeners or appropriate arrangements of diagonal stiffeners may be used in association with, or as an alternative to, transverse stiffeners in order to increase the resistance of the column web in tension.

NOTE: In welded joints, the transverse stiffeners should be aligned with the corresponding beam flange. In bolted joints, the stiffener in the compression zone should be aligned with the centre of compression as defined figure 6.15.

(7) The welds connecting diagonal stiffeners to the column flange should be fill-in welds with a sealing run proving a combination throat thickness equal to the thickness of the stiffeners.

(8) Where an unstiffened column web is reinforced by adding supplementary web plates conforming with 6.2.4.1, the tension resistance depends on the throat thickness of the longitudinal welds connecting the supplementary web plates. The effective thickness of the web $t_{w,eff}$ should be taken as follows:

- when the longitudinal welds are full penetration butt welds with a throat thickness $a \geq t_s$ then:
  - for one supplementary web plate: $t_{w,eff} = 1{,}5\,t_{wc}$ ... (6.17)
  - for supplementary web plates both sides: $t_{w,eff} = 2{,}0\,t_{wc}$ ... (6.18)
- when the longitudinal welds are fillet welds with a throat thickness $a \geq t_s/\sqrt{2}$ then for either one or two supplementary web plates:
  - for steel grades S 235, S 275 or S 355: $t_{w,eff} = 1{,}4\,t_{wc}$ ... (6.19a)
  - for steel grades S 420 or S 460: $t_{w,eff} = 1{,}3\,t_{wc}$ ... (6.19b)

(9) In calculating the reduction factor ω for the possible effects of shear stress, the shear area A_{vc} of a column web reinforced by adding supplementary web plates may be increased only to the extent permitted when determining its shear resistance, see 6.2.4.1(6).

6.2.4.4 Column flange in tranverse bending

6.2.4.4.1 Unstiffened column flange, bolted connection

(1) The resistance and failure mode of an unstiffened column flange in tranverse bending, together with the associated bolts in tension, should be taken as similar to those of an equivalent T-stub flange, see 6.2.2, for both:

- each individual bolt-row required to resist tension;
- each group of bolt-rows required to resist tension.

(2) The dimensions e_{min} and m for use in 6.2.2 should be determined from figure 6.8.

(3) The effective length of equivalent T-stub flange should be determined for the individual bolt-rows and the bolt-group in accordance with 6.2.2.2 from the values given for each bolt-row in table 6.4.

a) Welded end-plate narrower than column flange.

b) Welded end-plate wider than column flange.

c) Angle flange cleats.

Figure 6.8: Definitions of e, e_{min}, r_c and m

Table 6.4: Effective lengths for an unstiffened column flange

Bolt-row location	Bolt-row considered individually		Bolt-row considered as part of a group of bolt-rows	
	Circular patterns $\ell_{eff,cp}$	Non-circular patterns $\ell_{eff,nc}$	Circular patterns $\ell_{eff,cp}$	Non-circular patterns $\ell_{eff,nc}$
Inner bolt-row	$2\pi m$	$4m + 1{,}25e$	$2p$	p
End bolt-row	The smaller of: $2\pi m$ $\pi m + 2e_1$	The smaller of: $4m + 1{,}25e$ $2m + 0{,}625e + e_1$	The smaller of: $\pi m + p$ $2e_1 + p$	The smaller of: $2m + 0{,}625e + 0{,}5p$ $e_1 + 0{,}5p$
Mode 1:	$\ell_{eff,1} = \ell_{eff,nc}$ but $\ell_{eff,1} \leq \ell_{eff,cp}$		$\Sigma\ell_{eff,1} = \Sigma\ell_{eff,nc}$ but $\Sigma\ell_{eff,1} \leq \Sigma\ell_{eff,cp}$	
Mode 2:	$\ell_{eff,2} = \ell_{eff,nc}$		$\Sigma\ell_{eff,2} = \Sigma\ell_{eff,nc}$	

6.2.4.4.2 Stiffened column flange, joint with bolted end-plate or flange cleats

(1) Transverse stiffeners or appropriate arrangements of diagonal stiffeners may be used in association with, or as an alternative to, transverse stiffeners in order to increase the resistance of the column flange in bending.

(2) The resistance and failure mode of a stiffened column flange in transverse bending, together with the associated bolts in tension, should be taken as similar to those of an equivalent T-stub flange, see 6.2.2, for both:

- each individual bolt-row required to resist tension;
- each group of bolt-rows required to resist tension.

(3) The groups of bolt-rows each side of a stiffener should be modelled as separate equivalent T-stub flanges, see figure 6.9. The resistance and failure mode should be determined separately for each equivalent T-stub.

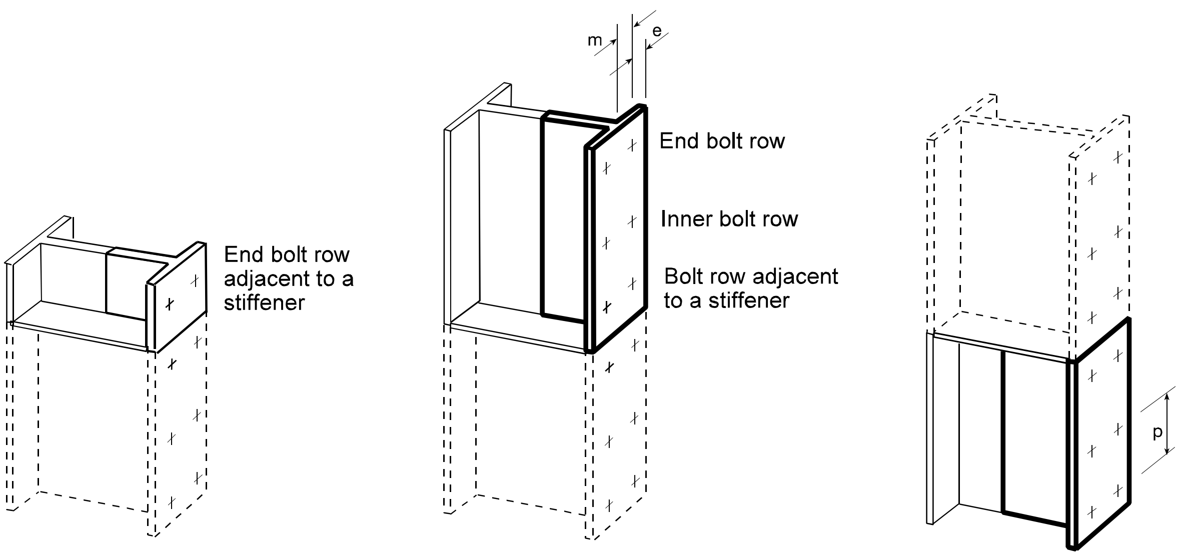


Figure 6.9: Modelling a stiffened column flange as separate T-stubs

(4) The dimensions e_{min} and m for use in 6.2.2 should be determined from figure 6.8.

(5) The effective lengths of equivalent T-stub flange ℓ_{eff} should be determined in accordance with 6.2.2.2 from the values given for each bolt-row in table 6.5. The value of α for use in table 6.5 should be obtained from figure 6.11.

(6) The stiffeners should meet the requirements specified in 6.2.4.1.

Table 6.5: Effective lengths for a stiffened column flange

Bolt-row location	Bolt-row considered individually		Bolt-row considered as part of a group of bolt-rows	
	Circular patterns $\ell_{eff,cp}$	Non-circular patterns $\ell_{eff,nc}$	Circular patterns $\ell_{eff,cp}$	Non-circular patterns $\ell_{eff,nc}$
Bolt-row adjacent to a stiffener	$2\pi m$	αm	$\pi m + p$	$0{,}5p + \alpha m - (2m + 0{,}625e)$
Other inner bolt-row	$2\pi m$	$4m + 1{,}25e$	$2p$	p
Other end bolt-row	The smaller of: $2\pi m$; $\pi m + 2e_1$	The smaller of: $4m + 1{,}25e$; $2m + 0{,}625e + e_1$	The smaller of: $\pi m + p$; $2e_1 + p$	The smaller of: $2m + 0{,}625e + 0{,}5p$; $e_1 + 0{,}5p$
End bolt-row adjacent to a stiffener	The smaller of: $2\pi m$; $\pi m + 2e_1$	$e_1 + \alpha m - (2m + 0{,}625e)$	not relevant	not relevant
For Mode 1:	$\ell_{eff,1} = \ell_{eff,nc}$ but $\ell_{eff,1} \leq \ell_{eff,cp}$		$\Sigma\ell_{eff,1} = \Sigma\ell_{eff,nc}$ but $\Sigma\ell_{eff,1} \leq \Sigma\ell_{eff,cp}$	
For Mode 2:	$\ell_{eff,2} = \ell_{eff,nc}$		$\Sigma\ell_{eff,2} = \Sigma\ell_{eff,nc}$	
α should be obtained from figure 6.11.				

6.2.4.4.3 Unstiffened column flange, welded connection

(1) In a welded joint, the resistance $F_{fc,Rd}$ of an unstiffened column flange in bending, due to tension or compression from a beam flange, should be obtained using:

$$F_{fc,Rd} = b_{eff,b,fc}\, t_{fb}\, f_{y,fb} / \gamma_{M0} \qquad \text{... (6.20)}$$

where: $b_{eff,b,fc}$ is determined as the effective breath b_{eff} defined in 4.10 where the beam flange is considered as a plate.

NOTE: The requirements specified in 4.10(4) and 4.10(6) should be satisfied.

6.2.4.5 End-plate in bending

(1) The resistance and failure mode of an end-plate in bending, together with the associated bolts in tension, should be taken as similar to those of an equivalent T-stub flange, see 6.2.2 for both:

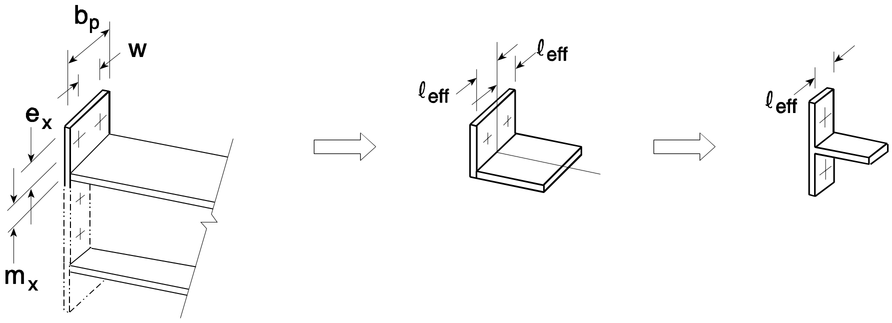
- each individual bolt-row required to resist tension;
- each group of bolt-rows required to resist tension.

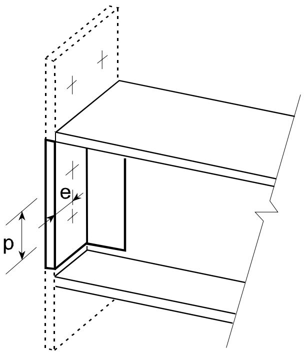
(2) The groups of bolt-rows at each side of any stiffener connected to the end-plate should be treated as separate equivalent T-stubs. In extended end-plates, the bolt-row in the extended part should also be treated as a separate equivalent T-stub, see figure 6.10. The resistance and failure mode should be determined separately for each equivalent T-stub.

(3) The dimension e_{min} required for use in 6.2.2 should be obtained from figure 6.8 for the part of the end-plate located between the beam flanges. For the end-plate extension e_{min} should be taken as equal to e_x, see figure 6.10.

(4) The effective lengths of equivalent T-stub flange ℓ_{eff} should be determined in accordance with 6.2.2.2 from the values given for each bolt-row in table 6.6.

(5) The values of m and m_x for use in table 6.6 should be obtained from figure 6.10.





The extension of the end-plate and the portion between the beam flanges are modelled as two separate equivalent T-stub flanges.

For the end-plate extension, use e_x and m_x in place of e and m when determining the resistance of the equivalent T-stub flange.

Figure 6.10: Modelling an extended end-plate as separate T-stubs

Table 6.6: Effective lengths for an end-plate

Bolt-row location	Bolt-row considered individually		Bolt-row considered as part of a group of bolt-rows	
	Circular patterns $\ell_{eff,cp}$	Non-circular patterns $\ell_{eff,nc}$	Circular patterns $\ell_{eff,cp}$	Non-circular patterns $\ell_{eff,nc}$
Bolt-row outside tension flange of beam	Smallest of: $2\pi m_x$ $\pi m_x + w$ $\pi m_x + 2e$	Smallest of: $4m_x + 1{,}25e_x$ $e + 2m_x + 0{,}625e_x$ $0{,}5b_p$ $0{,}5w + 2m_x + 0{,}625e_x$	—	—
First bolt-row below tension flange of beam	$2\pi m$	αm	$\pi m + \text{p}$	$0{,}5p + \alpha m - (2m + 0{,}625e)$
Other inner bolt-row	$2\pi m$	$4m + 1{,}25\,e$	$2p$	p
Other end bolt-row	$2\pi m$	$4m + 1{,}25\,e$	$\pi m + p$	$2m + 0{,}625e + 0{,}5p$
Mode 1:	$\ell_{eff,1} = \ell_{eff,nc}$ but $\ell_{eff,1} \leq \ell_{eff,cp}$		$\Sigma\ell_{eff,1} = \Sigma\ell_{eff,nc}$ but $\Sigma\ell_{eff,1} \leq \Sigma\ell_{eff,cp}$	
Mode 2:	$\ell_{eff,2} = \ell_{eff,nc}$		$\Sigma\ell_{eff,2} = \Sigma\ell_{eff,nc}$	
α should be obtained from figure 6.11.				

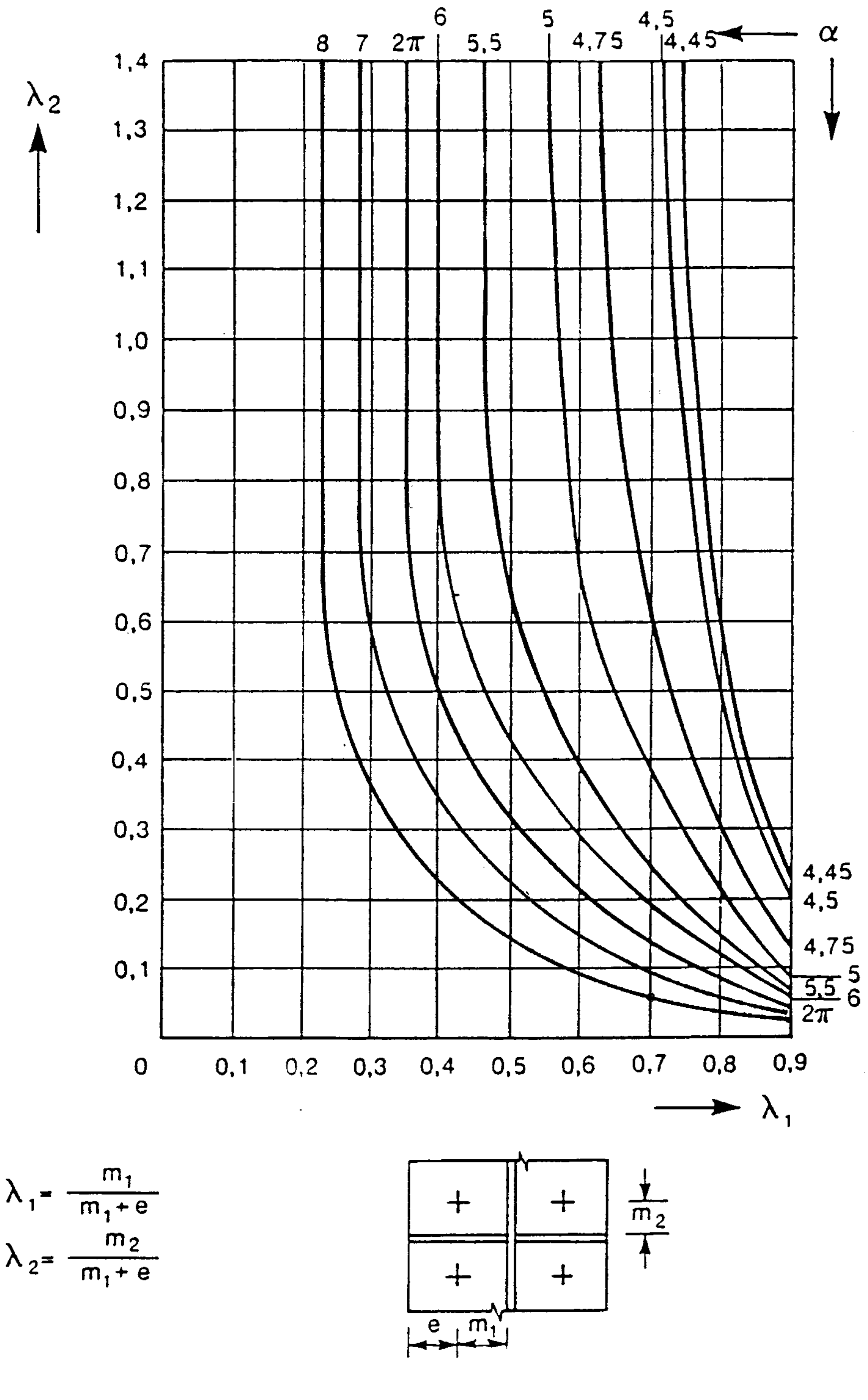


Figure 6.11: Values of *α* for stiffened column flanges and end-plates

6.2.4.6 Flange cleat in bending

(1) The resistance and failure mode of a bolted angle flange cleat in bending, together with the associated bolts in tension, should be taken as similar to those of an equivalent T-stub flange, see 6.2.2.

(2) The effective length ℓ_{eff} of the equivalent T-stub flange should be taken as $0{,}5b_a$ where b_a is the length of the angle cleat, see figure 6.12.

(3) The dimensions e_{min} and m for use in 6.2.2 should be determined from figure 6.13.

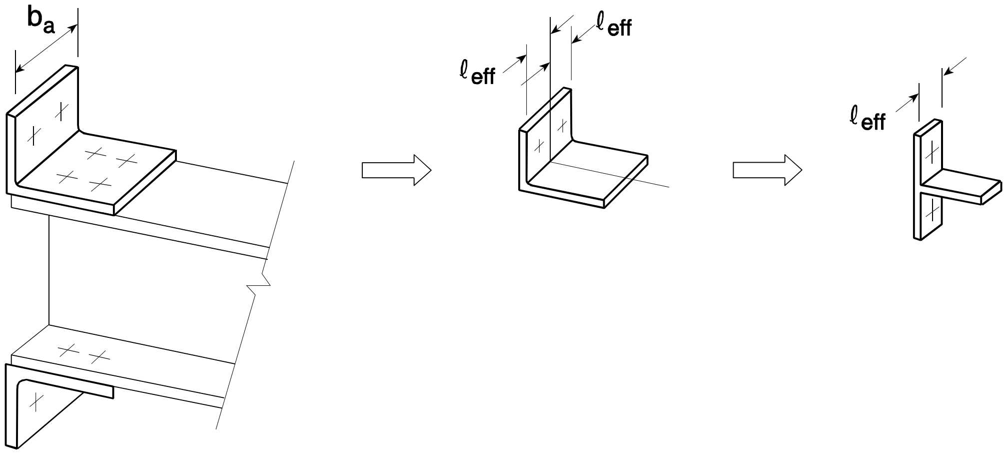
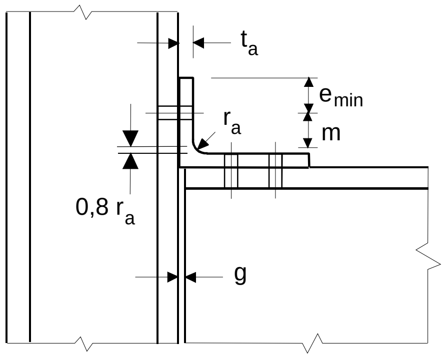
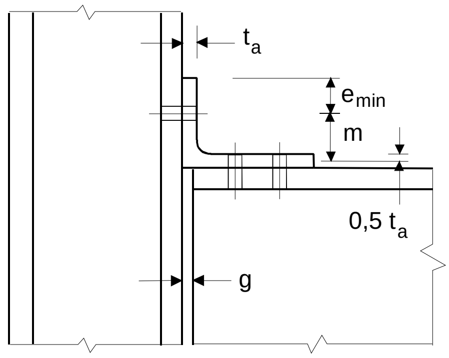


Figure 6.12: Effective length ℓ_{eff} of an angle flange cleat





a) Gap $g \leq 0{,}4\,t_a$

b) Gap $g > 0{,}4\,t_a$

Notes:

- The number of bolt-rows connecting the cleat to the column flange is limited to one;
- The number of bolt-rows connecting the cleat to the beam flange is not limited;
- The length b_a of the cleat may be different from both the width of the beam flange and the width of the column flange.

Figure 6.13: Dimensions e_{min} and m for a bolted angle cleat

6.2.4.7 Beam flange and web in compression

(1) The compression resistance of a beam flange and the adjacent compression zone of the beam web, may be assumed to act at the level of the centre of compression, see 6.2.5. It may be assumed to be given with sufficient accuracy by:

$$F_{c,fb,Rd} = M_{c,Rd}/(h - t_{fb}) \qquad \text{... (6.21)}$$

where: h is the depth of the connected beam;

$M_{c,Rd}$ is the moment resistance of the beam cross-section, reduced if necessary to allow for shear, see EN 1993-1-1. For a haunched beam $M_{c,Rd}$ may be calculated by neglecting intermediate flange.

t_{fb} is the flange thickness of the connected beam.

If the height of the beam including the haunch exceeds 600 mm the contribution of the beam web to the compression resistance should be limited to 20%.

(2) If a beam is reinforced with haunches they should be arranged with:

- the steel grade should match that of the member;
- the flange size and the web thickness of the haunch should not be less than that of the member;
- the angle of the haunch flange to the flange of the member should not be greater than 45°;
- the length of stiff bearing s_s should be taken as equal to the thickness of the haunch flange parralel to the beam.

(3) If a beam is reinforced with haunches, the resistcance of beam web in compression should be determined according to 6.2.4.2.

6.2.4.8 Beam web in tension

(1) In a bolted end-plate connection, the tension resistance of the beam web should be obtained using:

$$F_{t,wb,Rd} = b_{eff,t,wb}\, t_{wb}\, f_{y,wb} / \gamma_{M0} \qquad \text{... (6.22)}$$

(2) The effective width $b_{eff,t,wb}$ of the beam web in tension should be taken as equal to the effective length of the equivalent T-stub representing the end-plate in bending, obtained from 6.2.4.5 for an individual bolt-row or a bolt-group.

6.2.4.9 Concrete in compression including grout

(1) The design bearing strength of the joint between the base plate and the foundation should be determined taking account of the material properties and dimensions of both the grout and the concrete foundation. The punching resistance of the concrete slab shall be designed according to EN 1992.

(2) The resistance of concrete in compression, including grout, together with the associated base plate in bending $F_{c,pl,Rd}$, should be taken as similar to those of an equivalent T-stub, see 6.2.3.

6.2.4.10 Base plate in bending under compression

(1) The resistance of a base plate in bending under compression, together with concrete slab on which the column base is placed $F_{c,pl,Rd}$, should be taken as similar to those of an equivalent T-stub, see 6.2.3.

6.2.4.11 Base plate in bending under tension

(1) The resistance and failure mode of a base plate in bending under tension, together with the associated anchor bolts in tension $F_{t,pl,Rd}$, may be determined with the rules given in 6.2.4.5.

(2) In case of base plates prying forces which may develop should not be taken into consideration.

6.2.4.12 Anchor bolt in tension

(1) Anchor bolts should be designed to resist the effects of the design loads. They should provide resistance to tension due to uplift forces and bending moments where appropriate.

(2) When calculating the tension forces in the anchor bolts due to bending moments, the lever arm should not be taken as more than the distance between the centroid of the bearing area on the compression side and the centroid of the bolt group.

> **NOTE:** Tolerances on the positions of the anchor bolts should be taken into account if the influence of tolerances is significant.

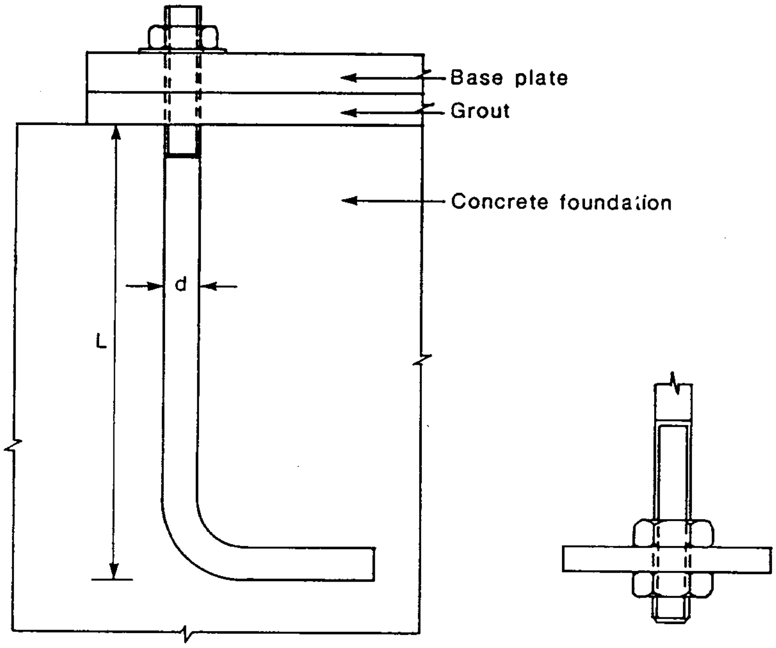
(3) The design resistance of the anchor bolts should be taken as the smaller of the tension resistance of the anchor bolt in tension, see 3.6, and the bound resistance in concrete of the anchor bolt according to EN 1992-1.

(4) Anchor bolts should either be anchored into the foundation by:

- a hook (figure 6.14(a)), or
- a washer plate (figure 6.14(b)), or
- some other appropriate load distributing member embedded in the concrete, or
- some other fixing which has been adequately tested and approved.

(5) When the bolts are provided with a hook, the anchorage length should be such as to prevent bond failure before yielding of the bolt. The anchorage length should be calculated in accordance EN 1992-1. This type of anchorage should not be used for bolts with a specified yield strength higher than 300 N/mm^2.

(6) When the anchor bolts are provided with a washer plate or other load distributing member, no account should be taken of the contribution of bond. The whole of the force should be transferred through the load distributing device.



(a) Hook (b) Washer plate

Figure 6.14: Anchorage of anchor bolts

6.2.5 Moment resistance of beam-to-column joints and splices

6.2.5.1 General

(1)P The applied design moment $M_{j,Ed}$ shall satisfy:

$$\frac{M_{j,Ed}}{M_{j,Rd}} \leq 1,0 \qquad \text{... (6.23)}$$

(2) The methods given in 6.2.5 for determining the moment resistance of a joint $M_{j,Rd}$ do not take account of any co-existing axial force N_{Ed} in the connected member. They should not be used if the axial force in the connected member exceeds 5% of the plastic resistance $N_{p\ell,Rd}$ of its cross-section.

(3) If the axial force N_{Ed} in the connected beam exceeds 5% of the resistance , $N_{pl,Rd}$, the following conservative method may be used:

$$\frac{M_{j,Ed}}{M_{j,Rd}} + \frac{N_{j,Ed}}{N_{j,Rd}} \leq 1,0$$

where: $M_{j.Rd}$ is the moment resistance of the joint, assuming no axial force;
$N_{j.Rd}$ is the axial resistance of the joint, assuming no applied moment.

(4) The moment resistance of a welded joint should be determined as indicated in figure 6.15(a).

(5) The moment resistance of a bolted joint with a flush end-plate that has only one bolt-row in tension (or in which only one bolt-row in tension is considered, see 6.2.1.3(6)) should be determined as indicated in figure 6.15(b).

(6) The moment resistance of a bolted joint with angle flange cleats should be determined as indicated in figure 6.15(c).

(7) The moment resistance of a bolted end-plate joint with more than one row of bolts in tension should generally be determined as specified in 6.2.5.2.

(8) As a conservative simplification, the moment resistance of an extended end-plate joint with only two rows of bolts in tension may be approximated as indicated in figure 6.16, provided that the total resistance F_{Rd} does not exceed $3,8B_{t,Rd}$, where $B_{t,Rd}$ is as in 6.2.2.1(?). In this case the whole tension region of the end-plate may be treated as a single basic component. Provided that the two bolt-rows are approximately equidistant either side of the beam flange, this part of the end-plate may be treated as a T-stub to determine the bolt-row force $F_{1,Rd}$. The value of $F_{2,Rd}$ may then be assumed to be equal to $F_{1,Rd}$, and so F_{Rd} may be taken as equal to $2F_{1,Rd}$.

(9) The centre of compression should be taken as the centre of the compression stress block of the compression forces. As a simplification the centre of compression may be taken as given in figure 6.15.

(10)P A splice in a member or part subject to tension shall be designed to transmit all the moments and forces to which the member or part is subjected at that point.

(11)P Splices shall be designed to hold the connected members in place. Friction forces between contact surfaces may not be relied upon to hold connected members in place in a bearing splice.

(12) Wherever practicable the members should be arranged so that the centroidal axis of any splice material coincides with the centroidal axis of the member. If eccentricity is present then the resulting forces should be taken into account.

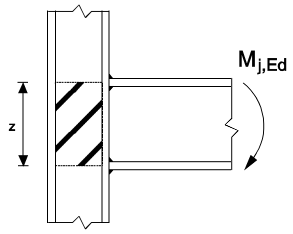
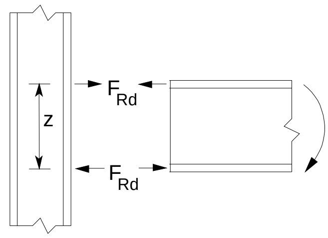
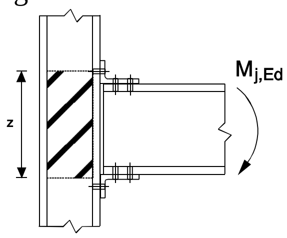
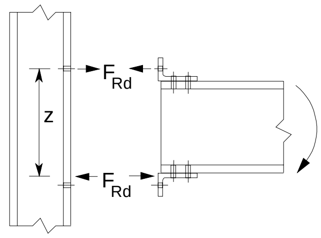
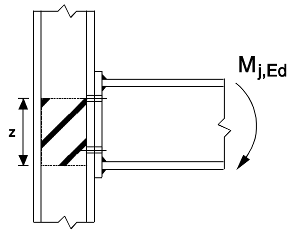
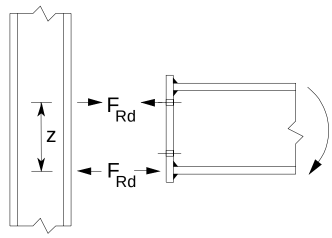
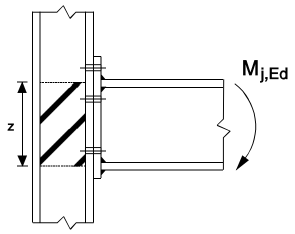
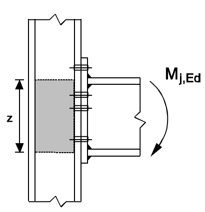
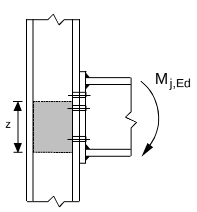
Type of connection	Centre of compression	Lever arm	Force distributions
a) Welded connection 	In line with the mid thickness of the compression flange	$z = h - t_{fb}$ h is the depth of the connected beam t_{fb} is the thickness of the beam	
b) Bolted connection with angle flange cleats 	In line with the mid-thickness of the leg of the angle cleat on the compression flange	Distance from the centre of compression to the bolt-row in tension	
c) Bolted end-plate connection with only one bolt-row active in tension 	In line with the mid-thickness of the compression flange	Distance from the centre of compression to the bolt-row in tension	
d) Bolted extended end-plate connection with only two bolt-rows active in tension 	In line with the mid-thickness of the compression flange	Conservatively z may be taken as the distance from the centre of compression to a point midway between these two bolt-rows	
e) Other bolted end-plate connections with two or more bolt-rows in tension  	In line with the mid-thickness of the compression flange	An approximate value may be obtained by taking the distance from the centre of compression to a point midway between the farthest two bolt-rows in tension	A more accurate value may be determined by taking the lever arm z as equal to z_{eq} obtained using the method given in 6.3.3.1.

Figure 6.15: Centre of compression, lever arm z and force distributions for deriving the moment resistance $M_{j,Rd}$

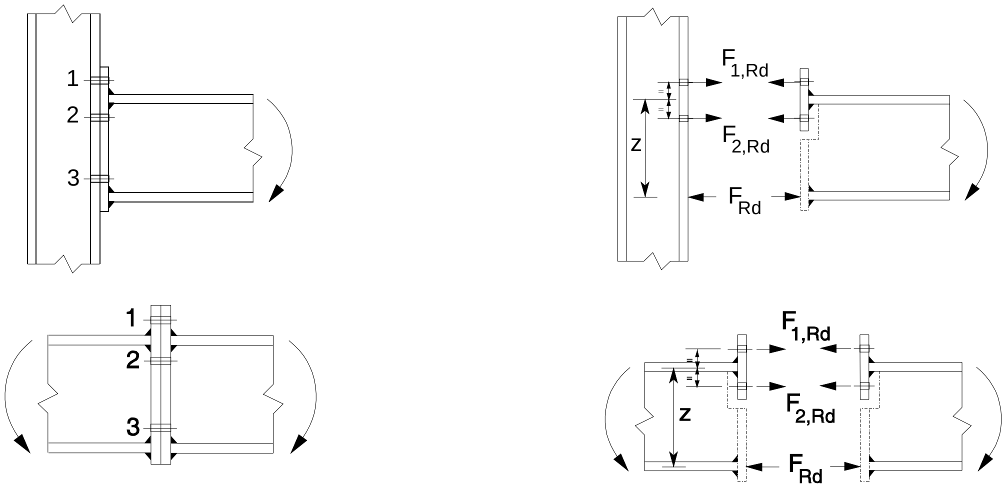


Figure 6.16: Simplified models for bolted joints with extended end-plates

(13) Where the members subjected to compression are not prepared for full contact in bearing, splice material should be provided to transmit the internal forces and moments in the member at the spliced section, including the moments due to applied eccentricity, initial imperfections and second-order deformations, see execution standards (see 2.8).

(14) Where the members subjected to compression are prepared for full contact in bearing, the splice should be designed as rigid about both axes, see 5.1.1, and to resist any tension where moments are present for any reason, including those given in (13).

(15) The alignment of the abutting ends of members subjected to compression should be maintained by cover plates or other means. The splice material and its fastenings should be proportioned to carry forces at the abutting ends, acting in any direction perpendicular to the axis of the member. In the design of splices also the second order effects should be taken into account.

(16) Splices in flexural members should comply with the following:

a) Compression flanges should be treated as compression members;
b) Tension flanges should be treated as tension members;
c) Parts subjected to shear should be designed to transmit the following effects acting together:
   - the shear force at the splice;
   - the moment resulting from the eccentricity, if any, of the centroids of the groups of fasteners on each side of the splice;
   - the proportion of moment, deformation or rotations carried by the web or part, irrespective of any shedding of stresses into adjoining parts assumed in the design of the member or part.

6.2.5.2 Beam-to-column joints with bolted end-plate connections

(1) The moment resistance $M_{j,Rd}$ of a beam-to-column joint with a bolted end-plate connection may be determined from:

$$M_{j,Rd} = \sum_r h_r F_{tr,Rd} \qquad \text{... (6.24)}$$

where: $F_{tr,Rd}$ is the effective tension resistance of bolt-row r ;

h_r is the distance from bolt-row r to the centre of compression;

r is the bolt-row number.

NOTE: In a bolted connection with more than one bolt-row in tension, the bolt-rows are numbered starting from the bolt-row farthest from the centre of compression.

(2) For bolted end-plate connections, the centre of compression should be assumed to be in line with the mid-thickness of the compression flange of the connected member.

(3) The effective tension resistance $F_{tr,Rd}$ for each bolt-row should be determined in sequence, starting from bolt-row 1, the bolt-row farthest from the centre of compression, then bolt-row 2, etc.

(4) When determining the value of $F_{tr,Rd}$ for bolt-row r all other bolt-rows closer to the centre of compression should be taken as omitted.

(5) The effective tension resistance $F_{tr,Rd}$ of bolt-row r should be taken as its tension resistance $F_{t,Rd}$ as an individual bolt-row determined from (6), reduced if necessary to satisfy the conditions specified in (7), (8) and (9).

(6) The tension resistance $F_{t,Rd}$ of bolt-row r as an individual bolt-row should be taken as the smallest value of the tension resistance for an individual bolt-row of the following basic components:

- the column web in tension $F_{t,wc,Rd}$ - see 6.2.4.3;
- the column flange in bending $F_{t,fc,Rd}$ - see 6.2.4.4;
- the end-plate in bending $F_{t,ep,Rd}$ - see 6.2.4.5;
- the beam web in tension $F_{t,wb,Rd}$ - see 6.2.4.8.

(7) The effective tension resistance $F_{tr,Rd}$ of bolt-row r should, if necessary, be reduced below the value of $F_{t,Rd}$ given by (6) in order to ensure that, when account is taken of all bolt-rows up to and including bolt-row r:

- the total resistance $\Sigma F_{t,Rd} \leq V_{wp,Rd}/\beta$ - with β from 5.3(7) - see 6.2.4.1;
- the total resistance $\Sigma F_{t,Rd}$ does not exceed the smaller of:
  - the resistance of the column web in compression $F_{c,wc,Rd}$ - see 6.2.4.2;
  - the resistance of the beam flange and web in compression $F_{c,fb,Rd}$ - see 6.2.4.7.

(8) The effective tension resistance $F_{tr,Rd}$ of bolt-row r should, if necessary, be reduced below the value of $F_{t,Rd}$ given by (6), in order to ensure that the sum of the resistances taken for the bolt-rows up to and including bolt-row r that form part of the same group of bolt-rows, does not exceed the resistance of that group as a whole. This should be checked for the following basic components:

- the column web in tension $F_{t,wc,Rd}$ - see 6.2.4.3;
- the column flange in bending $F_{t,fc,Rd}$ - see 6.2.4.4;
- the end-plate in bending $F_{t,ep,Rd}$ - see 6.2.4.5;
- the beam web in tension $F_{t,wb,Rd}$ - see 6.2.4.8.

(9) Where the effective tension resistance $F_{tx,Rd}$ of one of the previous bolt-rows x is greater than $1,9 B_{t,Rd}$, then the effective tension resistance $F_{tr,Rd}$ for bolt-row r should be reduced, if necessary, in order to ensure that:

$$F_{tr,Rd} \leq F_{tx,Rd}\, h_r / h_x \qquad \text{... (6.25)}$$

where: h_x is the distance from bolt-row x to the centre of compression;

x is the bolt-row farthest from the centre of compression that has a tension resistance greater than $1,9 B_{t,Rd}$.

NOTE: *The National Annex may give other situations where equation (6.25) is relevant*

(10) The same method may be applied to a bolted beam splice with welded end-pates, see figure 6.17, by omitting the items relating to the column.

Figure 6.17: Bolted beam splices with welded end-plates

6.2.6 Resistance of column bases

6.2.6.1 General

(1) For base plate connections loaded with normal force and bending moment, there may be:

- full compression under both column flanges in case of a dominant compressive normal force, see figure 6.18(a);
- full tension under both column flanges in case of a dominant tensile normal force, see figure 6.18(b);
- compression under one column flange and tension under the other column flange in case of a dominant moment, see figures 6.18(c) and (d);

(2) For column bases, a potential centre of compression should be assumed in line with the mid-thickness of the corresponding column flange, see figures 6.18(a), (b) and (c).

(3) For column bases with potentially one bolt row in tension, the corresponding centre of tension should be assumed in line with this bolt row in tension, see figure 6.18(c).

(4) As an approximation, for column bases with potentially two bolt rows in tension the corresponding centre of tension may be assumed in line with the mid thickness of the corresponding column flange. In this case, one bolt row should be located in the extended part of the base plate and one bolt row between the column flanges. The distance between both bolt rows and the centre of the column flange should be identical

(5) Methods to calculate the lever arm z, dependent on the loading, are given in 6.2.6.

(6) Columns should be provided with base plates capable of distributing the compressive forces in the compressed parts of the column over a bearing area, such that the bearing pressure on the foundation does not exceed the design bearing strength of the joint, see 6.2.3(7).

(7) When calculating the tension forces due to bending moments, the lever arm should not be taken as more than the distance between the centroid of the bearing area on the compression side and the centroid of the bolt group on the tension side, taking the tolerances on the positions of the anchor bolts into account. This clause relevant to the lever arm should apply only within a plastic distribution of the forces.

(8) If no special elements for resisting the shear force are provided, such as block or bar shear connectors, it should be demonstrated that sufficient resistance to transfer the shear force between the column and the foundation is provided by one of the following:

- the frictional resistance of the joint between the base plate and the foundation;
- the shear resistance of the anchor bolts;

- the shear resistance of the surrounding part of the foundation.

When the shear force between the columns and the foundation is taken up by shear in the anchor bolts, the risk of concrete rupture due to bearing should also be checked.

a) Column base connection in case of a dominant compressive normal force

b) Column base connection in case of a dominant tensile normal force

c) Column base connection in case of a dominant bending moment

d) Column base connection in case of a dominant bending moment

Figure 6.18: Determination of the lever arm z for column base connections

6.2.6.2 Column bases only subjected to axial forces

(1) The resistance of a symmetric column base $N_{j,Rd}$ with a compressive normal force only may be determined as the sum of the individual resistance $F_{C,Rd}$ of three T-stubs according to 6.2.3: Two T-stubs under the column flanges and one T-stub under the column web.

NOTE: If the compressive normal force causes any bending moment in the column base (e.g. due to eccentricity), the resistance should be determined according to 6.2.6.3.

(2) The three T-stubs referred to in (1) should be non-overlapping, see figure 6.19.

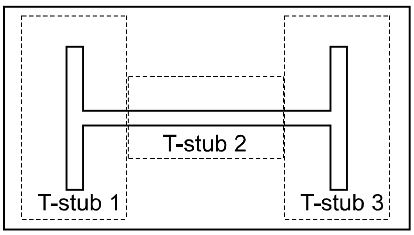


Figure 6.19: Non overlapping T-stubs

6.2.6.3 Column bases subjected to axial forces and bending moments

(1) The moment resistance $M_{j,Rd}$ of a column base for normal force in combination with bending moment may be determined by table 6.7 where the contribution of the concrete portion just under the column web (T-stub 2 of figure 6.19) to the compressive capacity is omitted.

(2) The tension resistance $F_{T,l,Rd}$ of the left side of the joint should be taken as the smallest values of the resistance of following basic components:

- the column web in tension under the left column flange $F_{t,wc,Rd}$ - see 6.2.4.3;
- the base plate in bending under the left column flange $F_{t,pl,Rd}$ - see 6.2.4.11.

(3) The tension resistance $F_{T,r,Rd}$ of the right side of the joint should be taken as the smallest values of the resistance of following basic components:

- the column web in tension under the right column flange $F_{t,wc,Rd}$ - see 6.2.4.3;
- the base plate in bending under the right column flange $F_{t,pl,Rd}$ - see 6.2.4.11.

(4) The compressive resistance $F_{C,l,Rd}$ of the left side of the joint should be taken as the smallest values of the resistance of following basic components:

- the concrete in compression under the left column flange $F_{c,pl,Rd}$ - see 6.2.4.9;
- the left column flange and web in compression $F_{c,fc,Rd}$ - see 6.2.4.7.

(5) The compressive resistance $F_{C,r,Rd}$ of the right side of the joint should be taken as the smallest values of the resistance of following basic components:

- the concrete in compression under the right column flange $F_{c,pl,Rd}$ - see 6.2.4.9;
- the right column flange and web in compression $F_{c,fc,Rd}$ - see 6.2.4.7.

(6) For the calculation of $z_{T,l}$, $z_{C,l}$, $z_{T,r}$, $z_{C,r}$ see 6.2.6.1.

Table 6.7: Moment resistance $M_{j,Rd}$ of column bases

Loading	Lever arm z	Moment resistance $M_{j,Rd}$	
Left side in tension Right side in compression	$z = z_{T,l} + z_{C,r}$	$N_{Ed} > 0$ and $e > z_{T,l}$	$N_{Ed} \leq 0$ and $e \leq -z_{C,r}$
		The smaller of $\frac{F_{T,l,Rd}\ z}{z_{C,r}/e\ +\ 1}$ and $\frac{-F_{C,r,Rd}\ z}{z_{T,l}/e\ -\ 1}$	
Left side in tension Right side in tension	$z = z_{T,l} + z_{T,r}$	$N_{Ed} > 0$ and $0 < e < z_{T,l}$	$N_{Ed} > 0$ and $-z_{T,r} < e \leq 0$
		The smaller of $\frac{F_{T,l,Rd}\ z}{z_{T,r}/e\ +\ 1}$ and $\frac{F_{T,r,Rd}\ z}{z_{T,l}/e\ -\ 1}$	The smaller of $\frac{F_{T,l,Rd}\ z}{z_{T,r}/e\ +\ 1}$ and $\frac{F_{T,r,Rd}\ z}{z_{T,l}/e\ -\ 1}$
Left side in compression Right side in tension	$z = z_{C,l} + z_{T,r}$	$N_{Ed} > 0$ and $e \leq -z_{T,r}$	$N_{Ed} \leq 0$ and $e > z_{C,l}$
		The smaller of $\frac{-F_{C,l,Rd}\ z}{z_{T,r}/e\ +\ 1}$ and $\frac{F_{T,r,Rd}\ z}{z_{C,l}/e\ -\ 1}$	
Left side in compression Right side in compression	$z = z_{C,l} + z_{C,r}$	$N_{Ed} \leq 0$ and $0 < e < z_{C,l}$	$N_{Ed} \leq 0$ and $-z_{C,r} < e \leq 0$
		The smaller of $\frac{-F_{C,l,Rd}\ z}{z_{C,r}/e\ +\ 1}$ and $\frac{-F_{C,r,Rd}\ z}{z_{C,l}/e\ -\ 1}$	The smaller of $\frac{-F_{C,l,Rd}\ z}{z_{C,r}/e\ +\ 1}$ and $\frac{-F_{C,r,Rd}\ z}{z_{C,l}/e\ -\ 1}$
$M_{Ed} > 0$ is clockwise, $N_{Ed} > 0$ is tension $e = \frac{M_{Ed}}{N_{Ed}} = \frac{M_{Rd}}{N_{Rd}}$			

6.3 Rotational stiffness

6.3.1 Basic model

(1) The rotational stiffness of a joint should be determined from the flexibilities of its basic components, each represented by its elastic stiffness coefficient k_i obtained from 6.3.2.

NOTE: These elastic stiffness coefficients are of general application.

(2) For bolted end-plate connections with more than one row of bolts in tension, the stiffness coefficients k_i for the related basic components should be combined. For beam-to-column joints and beam splices a method is given in 6.3.3 and for column bases a method is given in 6.3.4.

(3) In a bolted connection with more than one bolt-row in tension, as a simplification the contribution of any bolt-row may be neglected, provided that the contributions of all other bolt-rows closer to the centre of compression are also neglected. The number of bolt-rows retained need not necessarily be the same as for the determination of the moment resistance.

(4) Provided that the axial force N_{Ed} in the connected member does not exceed 5% of the resistance $N_{p\ell,Rd}$ of its cross-section, the rotational stiffness S_j of a beam-to-column joint or beam splice, for a moment $M_{j,Ed}$ less than the moment resistance $M_{j,Rd}$ of the joint, may be obtained with sufficient accuracy from:

$$S_j = \frac{E z^2}{\mu \sum_i \frac{1}{k_i}} \quad \text{... (6.26)}$$

where: k_i is the stiffness coefficient for basic joint component i;

z is the lever arm, see 6.2.5;

μ is the stiffness ratio $S_{j,ini} / S_j$, see (6);

NOTE: The initial rotational stiffness $S_{j,ini}$ of the joint is given by expression (6.26) with $\mu = 1{,}0$.

(5) The rotational stiffness S_j of a column base, for a moment $M_{j,Ed}$ less than the moment resistance $M_{j,Rd}$ of the joint, may be obtained with sufficient accuracy from 6.3.4.

(6) The stiffness ratio μ should be determined from the following:

- if $M_{j,Ed} \leq 2/3\, M_{j,Rd}$:

$$\mu = 1 \quad \text{... (6.27a)}$$

- if $2/3\, M_{j,Rd} < M_{j,Ed} \leq M_{j,Rd}$:

$$\mu = \left(1{,}5\, M_{j,Ed} / M_{j,Rd}\right)^{\psi} \quad \text{... (6.27b)}$$

in which the coefficient ψ is obtained from table 6.8.

Table 6.8: Value of the coefficient ψ

Type of connection:	ψ
Welded	2,7
Bolted end-plate	2,7
Bolted angle flange cleats	3,1
Base plate connections	2,7

(7) In stiffness calculations, the stiffness coefficients k_i for basic components taken into account should be as listed in table 6.9 for joint configurations with welded connections or bolted angle flange cleat connections, in table 6.10 for bolted end-plate connections and for base plate connections.

Table 6.9: Joints with welded connections or bolted angle flange cleat connections

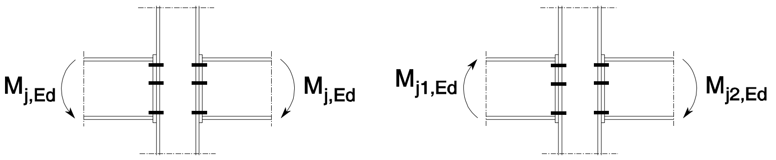
Beam-to-column joint with welded connections	Stiffness coefficients k_i to be taken into account
Single-sided	k_1; k_2; k_3
Double-sided - Moments equal and opposite	k_2; k_3
Double-sided - Moments unequal	k_1; k_2; k_3
Beam-to-column joint with bolted angle flange cleat connections	**Stiffness coefficients k_i to be taken into account**
Single-sided	k_1; k_2; k_3; k_4; k_6; k_{10}; k_{11} *); k_{12} **)
Double-sided - Moments equal and opposite	k_2; k_3; k_4; k_6; k_{10}; k_{11} *); k_{12} **)
Double-sided - Moments unequal	k_1; k_2; k_3; k_4; k_6; k_{10}; k_{11} *); k_{12} **)
 Moments equal and opposite Moments unequal	*) Two k_{11} coefficients, one for each flange; **) Four k_{12} coefficients, one for each flange and one for each cleat.

Table 6.10: Joints with bolted end-plate connections and base plate connections

Beam-to-column joint with bolted end-plate connections	Number of bolt-rows in tension	Stiffness coefficients k_i to be taken into account
Single-sided	One	k_1; k_2; k_3; k_4; k_5; k_{10}
	Two or more	k_1; k_2; k_{eq}
Double sided - Moments equal and opposite	One	k_2; k_3; k_4; k_5; k_{10}
	Two or more	k_2; k_{eq}
Double sided - Moments unequal	One	k_1; k_2; k_3; k_4; k_5; k_{10}
	Two or more	k_1; k_2; k_{eq}
Beam splice with bolted end-plates	**Number of bolt-rows in tension**	**Stiffness coefficients k_i to be taken into account**
Double sided - Moments equal and opposite	One	k_5[left]; k_5[right]; k_{10}
	Two or more	k_{eq}
Base plate connections	**Number of bolt-rows in tension**	**Stiffness coefficients k_i to be taken into account**
Base plate connections	One	k_{13}; k_{15}; k_{16}
	Two or more	k_{13}; k_{15} and k_{16} for each bolt row

6.3.2 Stiffness coefficients for basic joint components

(1) The stiffness coefficients for basic joint component should be determined from table 6.11.

Table 6.11: Stiffness coefficients for basic joint components

Component	Stiffness coefficient k_i	
Column web panel in shear	unstiffened, single-sided joint, or a double-sided joint in which the beam depths are similar	stiffened
	$k_1 = \dfrac{0{,}38\,A_{vc}}{\beta\,z}$	$k_1 = \infty$
	z is the lever arm from figure 6.15; β is the transformation parameter from 5.3(7).	
Column web in compression	unstiffened	stiffened
	$k_2 = \dfrac{0{,}7\,b_{eff,c,wc}\,t_{wc}}{d_c}$	$k_2 = \infty$
	$b_{eff,c,wc}$ is the effective width from 6.2.4.2	
Column web in tension	stiffened or unstiffened bolted connection with a single bolt-row in tension or unstiffened welded connection	stiffened welded connection
	$k_3 = \dfrac{0{,}7\,b_{eff,t,wc}\,t_{wc}}{d_c}$	$k_3 = \infty$
	$b_{eff,t,wc}$ is the effective width of the column web in tension from 6.2.4.3. For a joint with a single bolt-row in tension, $b_{eff,t,wc}$ should be taken as equal to the smallest of the effective lengths ℓ_{eff} (individually or as part of a group of bolt-rows) given for this bolt-row in table 6.4 (for an unstiffened column flange) or table 6.5 (for a stiffened column flange).	
Column flange in bending (for a single bolt-row in tension)	$k_4 = \dfrac{0{,}9\,\ell_{eff}\,t_{fc}^{\,3}}{m^3}$ ℓ_{eff} is the smallest of the effective lengths (individually or as part of a bolt group) for this bolt-row given in table 6.4 for an unstiffened column flange or table 6.5 for a stiffened column flange; m is as defined in figure 6.8;	
End-plate in bending (for a single bolt-row in tension)	$k_5 = \dfrac{0{,}9\,\ell_{eff}\,t_p^{\,3}}{m^3}$ ℓ_{eff} is the smallest of the effective lengths (individually or as part of a group of bolt-rows) given for this bolt-row in table 6.6; m is generally as defined in figure 6.11, but for a bolt-row located in the extended part of an extended end-plate $m = m_x$, where m_x is as defined in figure 6.10.	
Flange cleat in bending	$k_6 = \dfrac{0{,}9\,\ell_{eff}\,t_a^{\,3}}{m^3}$ ℓ_{eff} is the effective length of the flange cleat from figure 6.12; m is as defined in figure 6.13.	

<table>
<tr><td>Component</td><td colspan="3">Stiffness coefficient k_i</td></tr>
<tr><td>Bolts in tension
(for a single bolt-row)</td><td colspan="3">$k_{10} = 1,6 A_s / L_b$ preloaded or non-preloaded
L_b is the bolt elongation length, taken as equal to the grip length (total thickness of material and washers), plus half the sum of the height of the bolt head and the height of the nut.</td></tr>
<tr><td rowspan="3">Bolts in shear</td><td colspan="2">non-preloaded</td><td>preloaded *)</td></tr>
<tr><td colspan="2">k_{11} (or k_{17}) $= \dfrac{16\, n_b\, d^2 f_{ub}}{E\, d_{M16}}$</td><td>$k_{11} = \infty$</td></tr>
<tr><td colspan="3">d_{M16} is the nominal diameter of an M16 bolt;
n_b is the number of bolt-rows in shear.</td></tr>
<tr><td rowspan="3">Bolts in bearing
(for each component j on which the bolts bear)</td><td colspan="2">non-preloaded</td><td>preloaded *)</td></tr>
<tr><td colspan="2">k_{12} (or k_{18}) $= \dfrac{24\, n_b\, k_b\, k_t\, d\, f_u}{E}$</td><td>$k_{12} = \infty$</td></tr>
<tr><td>$k_b = k_{b1}$ but $k_b \leq k_{b2}$
$k_{b1} = 0,25\, e_b / d + 0,5$ but $k_{b1} \leq 1,25$
$k_{b2} = 0,25\, p_b / d + 0,375$ but $k_{b2} \leq 1,25$
$k_t = 1,5\, t_j / d_{M16}$ but $k_t \leq 2,5$</td><td colspan="2">e_b is the distance from the bolt-row to the free edge of the plate in the direction of load transfer;
f_u is the ultimate tensile strength of the steel on which the bolt bears;
p_b is the spacing of the bolt-rows in the direction of load transfer;
t_j is the thickness of that component.</td></tr>
<tr><td>Concrete in compression
(including grout)</td><td colspan="3">$k_{13} = \dfrac{E_c \sqrt{b_{eff}\, l_{eff}}}{1,275\, E}$
b_{eff} is the effective width of the T-stub flange, see 6.2.3(3);
l_{eff} is the effective length of the T-stub flange, see 6.2.3(3).</td></tr>
<tr><td>Plate in bending under compression</td><td colspan="3">$k_{14} = \infty$
This coefficient is already taken into consideration in the calculation of the stiffness coefficient k_{13}</td></tr>
<tr><td rowspan="3">Base plate in bending under tension
(for a single bolt row in tension)</td><td>with prying forces **)</td><td colspan="2">without prying forces **)</td></tr>
<tr><td>$k_{15} = \dfrac{0,85\, \ell_{eff}\, t_p^{\,3}}{m^3}$</td><td colspan="2">$k_{15} = \dfrac{0,425\, \ell_{eff}\, t_p^{\,3}}{m^3}$</td></tr>
<tr><td colspan="3">l_{eff} is the effective length of the T-stub flange, see 6.2.3(3);
t_p is the thickness of the base plate;
m is the distance according to figure 6.8.</td></tr>
<tr><td rowspan="2">Anchor bolts in tension</td><td>with prying forces **)</td><td colspan="2">without prying forces **)</td></tr>
<tr><td>$k_{16} = 1,6 A_s / L_b$</td><td colspan="2">$k_{16} = 2,0 A_s / L_b$</td></tr>
<tr><td colspan="4">*) provided that the bolts have been designed not to slip into bearing at the load level concerned
**) prying forces may develop, if $L_b \leq \dfrac{8,8\, m^3 A_s}{l_{eff}\, t^3}$
L_b is the anchor bolt elongation length, taken as equal to the sum of 8 times the nominal bolt diameter, the grout layer, the plate thickness, the washer and half of the height of the nut;
A_s is the tensile stress area of the anchor bolt</td></tr>
</table>

Component	Stiffness coefficient k_i
NOTE: When calculating b_{eff} and l_{eff} the distance c should be taken as 1,25 times the base plate thickness. **NOTE:** Backing plates should be assumed not to affect the rotational stiffness S_j of the joint. **NOTE:** For *welds* (k_{19}) the stiffness coefficient should be taken as equal to infinity. This component need not be taken into account when calculating the rotational stiffness S_j. **NOTE:** For *beam flange and web in compression* (k_7), *beam web in tension* (k_8), *plate in tension or compression* (k_9), *haunched beams* (k_{20}), the stiffness coefficients should be taken as equal to infinity. These components need not be taken into account when calculating the rotational stiffness S_j. **NOTE:** Where a *supplementary web plate* is used, the stiffness coefficients for the relevant basic joint components k_1 to k_3 should be increased as follows: - k_1 for the column web panel in shear should be based on the increased shear area A_{vc} from 6.2.4.1(6); - k_2 for the column web in compression should be based on the effective thickness of the web from 6.2.4.2(6); - k_3 for the column web in tension, should be based on the effective thickness of the web from 6.2.4.3(8).	

6.3.3 End-plate connections with two or more bolt-rows in tension

6.3.3.1 General method

(1) For end-plate connections with two or more bolt-rows in tension, the basic components related to all of these bolt-rows should be represented by a single equivalent stiffness coefficient k_{eq} determined from:

$$k_{eq} = \frac{\sum_r k_{eff,r}\, h_r}{z_{eq}} \qquad \text{... (6.28)}$$

where: h_r is the distance between bolt-row r and the centre of compression;

$k_{eff,r}$ is the effective stiffness coefficient for bolt-row r taking into account the stiffness coefficients k_i for the basic components listed in (4) or (5) as appropriate;

z_{eq} is the equivalent lever arm, see (3).

(2) The effective stiffness coefficient $k_{eff,r}$ for bolt-row r should be determined from:

$$k_{eff,r} = \frac{1}{\sum_i \frac{1}{k_{i,r}}} \qquad \text{... (6.29)}$$

where: $k_{i,r}$ is the stiffness coefficient representing component i relative to bolt-row r.

(3) The equivalent lever arm z_{eq} should be determined from:

$$z_{eq} = \frac{\sum_r k_{eff,r}\, h_r^2}{\sum_r k_{eff,r}\, h_r} \qquad \text{... (6.30)}$$

(4) In the case of a beam-to-column joint with an end-plate connection, k_{eq} should be based upon (and replace) the stiffness coefficients k_i for:

- the column web in tension (k_3);
- the column flange in bending (k_4);
- the end-plate in bending (k_5);
- the bolts in tension (k_{10}).

(5) In the case of a beam splice with bolted end-plates, k_{eq} should be based upon (and replace) the stiffness coefficients k_i for:

- the end-plates in bending (k_5);
- the bolts in tension (k_{10}).

(6) In calculating the rotational stiffness of joint configurations with bolted end-plate connections, the stiffness coefficients k_i for the basic components listed in table 6.10 should be taken into account.

6.3.3.2 Simplified method for extended end-plates with two bolt-rows in tension

(1) For extended end-plate connections with two bolt-rows in tension, (one in the extended part of the end-plate and one between the flanges of the beam, see figure 6.20), a set of modified values may be used for the stiffness coefficients of the related basic components to allow for the combined contribution of both bolt-rows. Each of these modified values should be taken as double the corresponding value for a single bolt-row in the extended part of the end-plate.

NOTE: This approximation leads to a slightly lower estimate of the rotational stiffness.

(2) When using this simplified method, the lever arm z should be taken as equal to the distance from the centre of compression to a point midway between the two bolt-rows in tension, see figure 6.20.

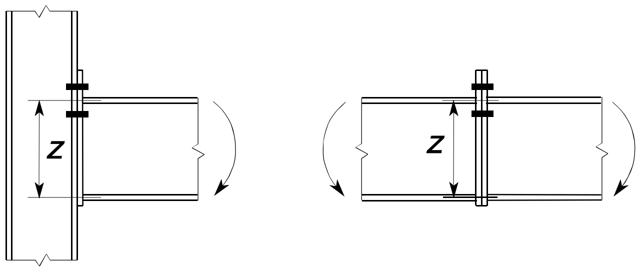


Figure 6.20: Lever arm z for simplified method

6.3.4 Column bases

(1) The rotational stiffness S_j of a column base for normal force in combination with bending moment may be determined with table 6.12 where the contribution of the concrete portion just under the column web (T-stub 2 of figure 6.19) to the rotational stiffness is omitted.

(2) The tension stiffness coefficient $k_{T,l}$ of the left side of the joint should be taken as the sum of the stiffness coefficients k_{15} and k_{16} acting on the left side of the joint, see table 6.11.

(3) The tension stiffness coefficient $k_{T,r}$ of the right side of the joint should be taken as the sum of the stiffness coefficients k_{15} and k_{16} acting on the right side of the joint, see table 6.11.

(4) The compressive stiffness coefficient $k_{C,l}$ of the left side of the joint should be taken as the stiffness coefficient k_{13} acting on the left side of the joint, see table 6.11.

(5) The compressive stiffness coefficient $k_{C,r}$ of the right side of the joint should be taken as the stiffness coefficient k_{13} acting on the right side of the joint, see table 6.11.

(6) For the calculation of $z_{T,l}$, $z_{C,l}$, $z_{T,r}$, $z_{C,r}$ see 6.2.6.1.

Table 6.12: Rotational stiffness S_j of column bases

Loading	Lever arm z	Rotational stiffness $S_{j,ini}$	
Left side in tension Right side in compression	$z = z_{T,l} + z_{C,r}$	$N_{Ed} > 0$ and $e > z_{T,l}$	$N_{Ed} \leq 0$ and $e \leq -z_{C,r}$
		$\dfrac{E\ z^2}{\mu\ (1/k_{T,l} + 1/k_{C,r})}\ \dfrac{e}{e+e_k}$	where $e_k = \dfrac{z_{C,r}\,k_{C,r} - z_{T,l}\,k_{T,l}}{k_{T,l} + k_{C,r}}$
Left side in tension Right side in tension	$z = z_{T,l} + z_{T,r}$	$N_{Ed} > 0$ and $0 < e < z_{T,l}$	$N_{Ed} > 0$ and $-z_{T,r} < e \leq 0$
		$\dfrac{E\ z^2}{\mu\ (1/k_{T,l} + 1/k_{T,r})}\ \dfrac{e}{e+e_k}$	where $e_k = \dfrac{z_{T,r}\,k_{T,r} - z_{T,l}\,k_{T,l}}{k_{T,l} + k_{T,r}}$
Left side in compression Right side in tension	$z = z_{C,l} + z_{T,r}$	$N_{Ed} > 0$ and $e \leq -z_{T,r}$	$N_{Ed} \leq 0$ and $e > z_{C,l}$
		$\dfrac{E\ z^2}{\mu\ (1/k_{C,l} + 1/k_{T,r})}\ \dfrac{e}{e+e_k}$	where $e_k = \dfrac{z_{T,r}\,k_{T,r} - z_{C,l}\,k_{C,l}}{k_{C,l} + k_{T,r}}$
Left side in compression Right side in compression	$z = z_{C,l} + z_{C,r}$	$N_{Ed} \leq 0$ and $0 < e < z_{C,l}$	$N_{Ed} \leq 0$ and $-z_{C,r} < e \leq 0$
		$\dfrac{E\ z^2}{\mu\ (1/k_{C,l} + 1/k_{C,r})}\ \dfrac{e}{e+e_k}$	where $e_k = \dfrac{z_{C,r}\,k_{C,r} - z_{C,l}\,k_{C,l}}{k_{C,l} + k_{C,r}}$
$M_{Ed} > 0$ is clockwise, $N_{Ed} > 0$ is tension, μ see 6.3.1(6). $e = \dfrac{M_{Ed}}{N_{Ed}} = \dfrac{M_{Rd}}{N_{Rd}}$			

6.4 Rotation capacity

(1) When rigid-plastic global analysis is used, see 5.1.3(4), and design resistance $M_{j,Rd}$ of the joint is at least 1,2 times the design plastic resistance $M_{p\ell,Rd}$ of the member, the rotation capacity need not be checked.

(2) A joint with a bolted connection in which the moment resistance $M_{j,Rd}$ is governed by the resistance of bolts in shear, should not be assumed to have sufficient rotation capacity for plastic global analysis.

(3) In the case of members of steel grades S 235, S 275 and S 355, the provisions given in (4) to (7) may be used for joints in which the axial force N_{Ed} in the connected member does not exceed 5% of the resistance $N_{p\ell,Rd}$ of its cross-section. However these provisions should not be applied in the case of members of steel grades S 420 and S 460.

(4) A beam-to-column joint in which the moment resistance of the joint $M_{j,Rd}$ is governed by the resistance of the column web panel in shear, may be assumed to have adequate rotation capacity for plastic global analysis, provided that $d/t_w \leq 69\varepsilon$.

(5) In a welded beam-to-column joint in which the column web is stiffened in compression but unstiffened in tension, provided that the moment resistance is not governed by the shear resistance of the column web panel, see (4), the rotation capacity ϕ_{Cd} may be assumed to be not less than the value given by:

$$\phi_{Cd} = 0{,}025\, h_c / h_b \qquad \text{... (6.31)}$$

where: h_b is the depth of the beam;

h_c is the depth of the column.

(6) An unstiffened welded beam-to-column joint designed in conformity with the provisions of this section, may be assumed to have a rotation capacity ϕ_{Cd} of at least 0,015 radians.

(7) A joint with a bolted connection with end-plates or angle flange cleats may be assumed to have sufficient rotation capacity for plastic analysis, provided that both of the following conditions are satisfied:

a) the moment resistance of the joint is governed by the resistance of either:

- the column flange in bending;
- the beam end-plate or tension flange cleat in bending.

b) the thickness t of either the column flange or the beam end-plate or tension flange cleat (not necessarily the same basic component as in (a)) satisfies:

$$t \leq 0{,}36\, d \sqrt{f_{ub} / f_y} \qquad \text{... (6.32)}$$

where: f_y is the yield strength of the relevant basic component.

(8) In cases not covered by (3) to (7), the rotation capacity may be determined by testing in accordance with EN 1990. Alternatively, appropriate calculation models may be used, provided that they are based on the results of tests in accordance with EN 1990.

7 Hollow section joints

7.1 General

7.1.1 Scope

(1) This section gives detailed application rules to determine the static resistances of uniplanar and multiplanar joints in lattice structures composed of circular, square or rectangular hollow sections, and of uniplanar joints in lattice structures composed of combinations of hollow sections with open sections.

(2) The static resistances of the joints are expressed in terms of maximum design axial and/or moment resistances for the brace members.

(3) These application rules are valid both for hot finished hollow sections to EN 10210 and for cold formed hollow sections to EN 10219, if the dimensions of the structural hollow sections fulfil the requirements of this section.

(4) The nominal yield strength of hot finished hollow sections and the nominal yield strength of the basic material of cold formed hollow sections should not exceed 460 N/mm^2. For grades S 420 and S 460 the static resistances given in this section should be reduced by a factor 0,9.

NOTE: According to EN 10210 and EN 10219 the requirements for material is determined based on the end product, not on the base material.

(5) The nominal wall thickness of hollow sections should be limited to a minimum of 2,5 mm.

(6) The nominal wall thickness of a hollow section chord should not be greater than 25 mm unless special measures have been taken to ensure that the through thickness properties of the material will be adequate.

(7) For fatigue assessment see EN 1993-1-9.

(8) The types of joints covered are indicated in figure 7.1.

7.1.2 Field of application

(1) The application rules given in this section may be used only where all of the following conditions are satisfied.

(2) The compression elements of the members should satisfy the requirements for Class 1 or Class 2 given in EN 1993-1-1 for the condition of pure bending.

(3) The angles θ_i between the chords and the brace members, and between adjacent brace members, should satisfy:

$$\theta_i \geq 30^\circ$$

(4) The ends of members that meet at a joint should be prepared in such a way that their cross-sectional shape is not modified. Flattened end connections and cropped end connections are not covered in this section.

(5) In gap type joints, in order to ensure that the clearance is adequate for forming satisfactory welds, the gap between the brace members should not be less than $(t_1 + t_2)$.

(6) In overlap type joints, the overlap should be large enough to ensure that the interconnection of the brace members is sufficient for adequate shear transfer from one brace to the other. In any case the overlap should be at least 25%.

(7) Where overlapping brace member have different thicknesses and/or different strength grades, the member with lowest $t_i\, f_{yi}$ - value should overlap the other member.

(8) Where overlapping brace members are of different widths, the narrower member should overlap the wider one.

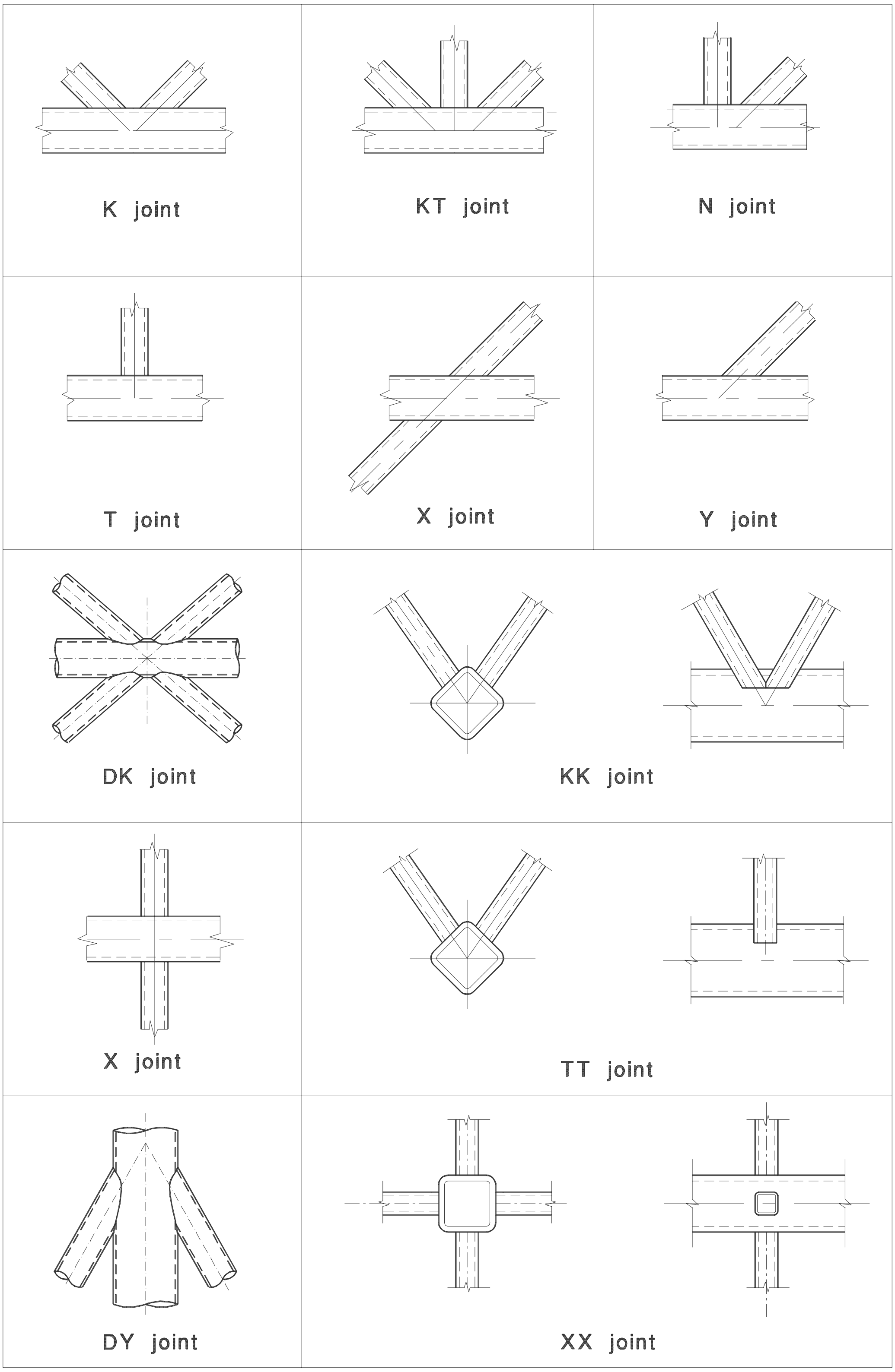


Figure 7.1: Types of joints in hollow section lattice girders

7.2 Design

7.2.1 General

(1) The design values of the internal axial forces both in the brace members and in the chords at the ultimate limit state should not exceed the design resistances of the members determined from EN 1993-1-1.

(2) The design values of the internal axial forces in the brace members at the ultimate limit state should also not exceed the design resistances of the joints given in 7.4, 7.5 or 7.6 as appropriate.

(3) The stresses $\sigma_{0,Ed}$ or $\sigma_{p,Ed}$ in the chord at a joint should be determined from:

$$\sigma_{0,Ed} = \frac{N_{0,Ed}}{A_0} + \frac{M_{0,Ed}}{W_{e\ell,0}} \quad \text{... (7.1)}$$

$$\sigma_{p,Ed} = \frac{N_{p,Ed}}{A_0} + \frac{M_{0,Ed}}{W_{e\ell,0}} \quad \text{... (7.2)}$$

where: $N_{p,Ed} = N_{0,Ed} - \sum_{i>0} N_{i,Ed} \cos \theta_i$

7.2.2 Failure modes for hollow section connections

(1) The design joint resistances of connections between hollow sections and of connections of hollow sections to open sections, should be based on the following failure modes as applicable:

a) **Chord face failure** (plastic failure of the chord face) or chord plastification (plastic failure of the chord cross-section);

b) **Chord side wall failure** (or **chord web failure**) by yielding, crushing or instability (crippling or buckling of the chord side wall or chord web) under the compression brace member;

c) **Chord shear failure**;

d) **Punching shear** failure of a hollow section chord wall (crack initiation leading to rupture of the brace members from the chord member);

e) **Brace failure** with reduced effective width (cracking in the welds or in the brace members);

f) **Local buckling** failure of a brace member or of a hollow section chord member at the joint location.

NOTE: The phrases printed in boldface type in this list are used to describe the various failure modes in the tables of design resistances given in 7.4 to 7.6.

(2) Figure 7.2 illustrates failure modes (a) to (f) for joints between CHS brace and chord members.

(3) Figure 7.3 illustrates failure modes (a) to (f) for joints between RHS brace and chord members.

(4) Figure 7.4 illustrates failure modes (a) to (f) for joints between CHS or RHS brace members and I or H section chord members.

(5) Although the resistance of a joint with properly formed welds is generally higher under tension than under compression, the design resistance of the joint is generally based on the resistance of the brace in compression to avoid the possible excessive local deformation or reduced rotation capacity or deformation capacity which might otherwise occur.

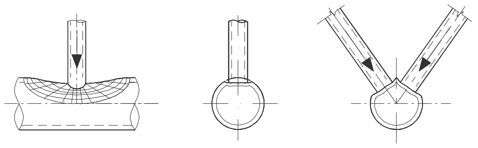
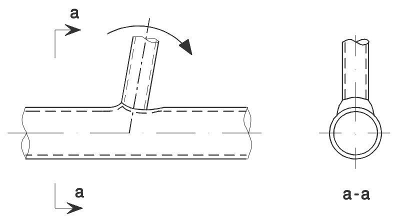
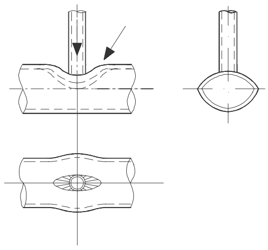
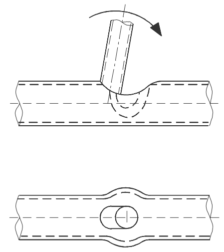
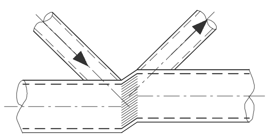
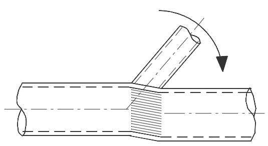
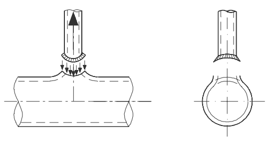
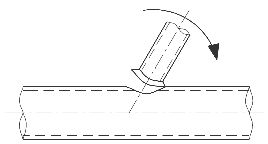
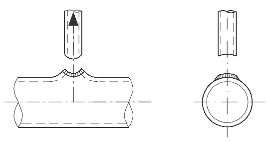
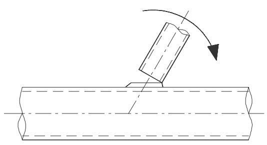
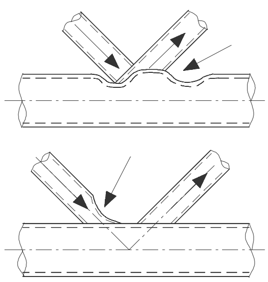
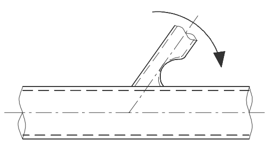
Mode	Axial loading	Bending moment
a		
b		
c		
d		
e		
f		

Figure 7.2: Failure modes for joints between CHS members

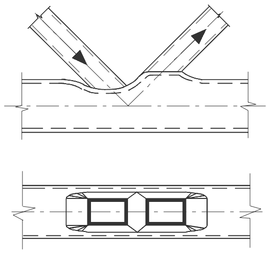
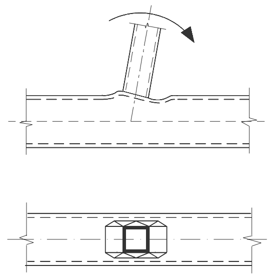
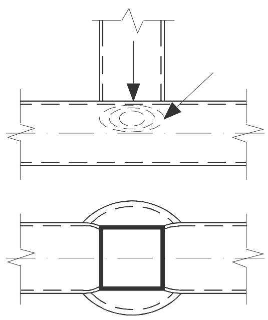
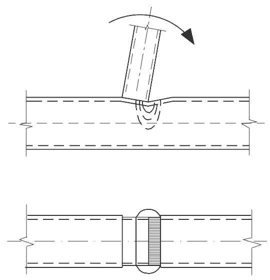
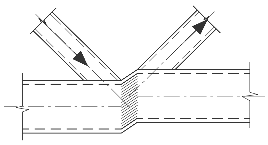
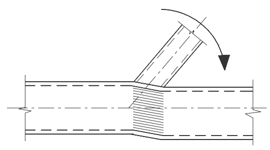
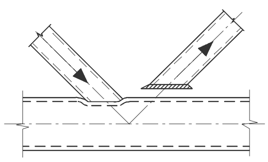
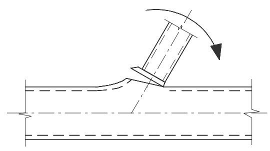
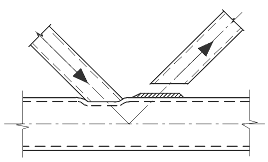
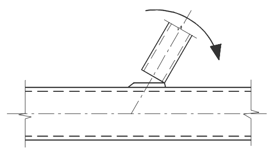
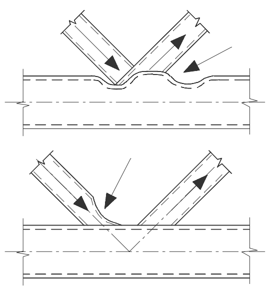
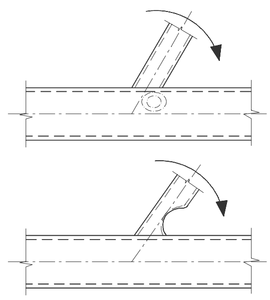
Mode	Axial loading	Bending moment
a		
b		
c		
d		
e		
f		

Figure 7.3: Failure modes for joints between RHS brace members and RHS chord members

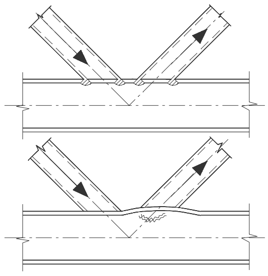
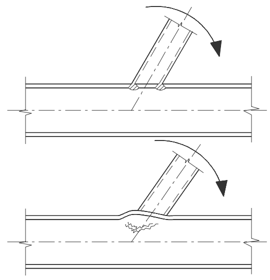
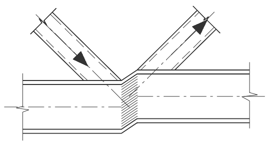
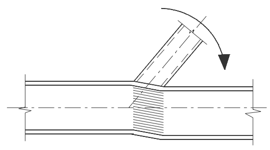
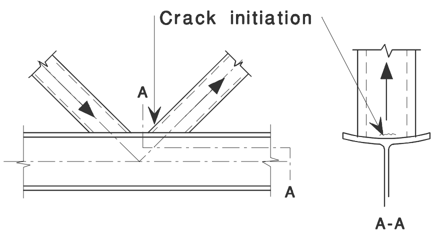
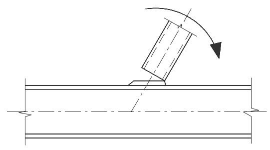
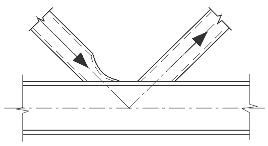
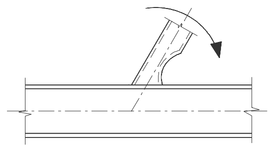
Mode	Axial loading	Bending moment
a	—	—
b		
c		
d	—	—
e		
f		

Figure 7.4: Failure modes for joints between CHS or RHS brace members and I or H section chord members

7.3 Welds

7.3.1 Design resistance

(1) The welds connecting the brace members to the chords should be designed to have sufficient resistance to allow for non-uniform stress-distributions and sufficient deformation capacity to allow for redistribution of bending moments.

(2) In welded joints, the connection should normally be established around the entire perimeter of the hollow section by means of a butt weld, a fillet weld, or combinations of the two. However in partially overlapping joints the hidden part of the connection need not be welded, provided that the axial forces in the brace members are such that their components perpendicular to the axis of the chord do not differ by more than 20%.

(3) Typical weld details are indicated in execution standards (see 2.8).

(4) The design resistance of the weld, per unit length of perimeter of a brace member, should not normally be less than the design resistance of the cross-section of that member per unit length of perimeter.

(5) The required throat thickness should be determined from section 4.

(6) The criterion given in (4) may be waived where a smaller weld size can be justified both with regard to resistance and with regard to deformation capacity and rotation capacity, taking account of the possibility that only part of its length is effective.

(7) For rectangular structural hollow sections the design throat thickness of flare groove welds is defined in figure 7.5.

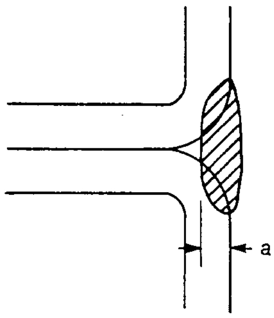


Figure 7.5: Effective throat of flare groove welds in rectangular structural hollow section

(8) Welding in cold-formed zones, see 4.14.

7.4 Welded joints between CHS members

7.4.1 General

(1) Provided that the geometry of the joints is within the range of validity given in table 7.1, the design resistances of welded joints between circular hollow section members should be determined using 7.4.2 and 7.4.3.

(2) For joints within the range of validity given in table 7.1, only chord face failure and punching shear need be considered. The design resistance of a connection should be taken as the minimum value for these two criteria.

(3) For joints outside the range of validity given in table 7.1, all the criteria given in 7.2.2 should be considered. In addition, the secondary moments in the joints caused by their rotational stiffness should be taken into account.

Table 7.1: Range of validity for welded joints between CHS brace members and CHS chords

$0{,}2 \leq d_i / d_0 \leq 1{,}0$
Class 2 and $10 \leq d_0 / t_0 \leq 50$ generally but $10 \leq d_0 / t_0 \leq 40$ for X joints
Class 2 and $10 \leq d_i / t_i \leq 50$
$\lambda_{ov} \geq 25\%$
$g \geq t_1 + t_2$

7.4.2 Uniplanar joints

(1) In brace member connections subject only to axial forces, the design internal axial force $N_{i,Ed}$ should not exceed the design axial resistance of the welded joint $N_{i,Rd}$ obtained from tables 7.2, 7.3 or 7.4 as appropriate.

(2) Brace member connections subject to combined bending and axial force should satisfy:

$$\frac{N_{i,Ed}}{N_{i,Rd}} + \left[\frac{M_{ip,i,Ed}}{M_{ip,i,Rd}}\right]^2 + \frac{M_{op,i,Ed}}{M_{op,i,Rd}} \leq 1{,}0 \qquad \text{... (7.3)}$$

where: $M_{ip,i,Rd}$ is the design in-plane moment resistance;

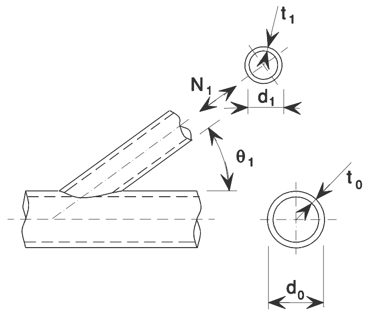
$M_{ip,i,Ed}$ is the design in-plane internal moment;

$M_{op,i,Rd}$ is the design out-of-plane moment resistance;

$M_{op,i,Ed}$ is the design out-of-plane internal moment.

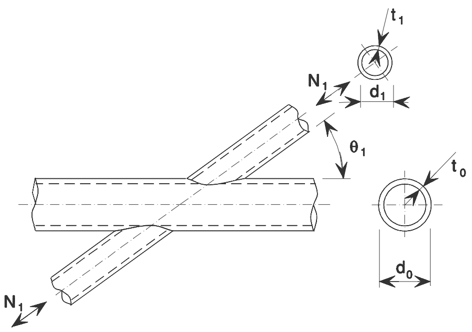
Table 7.2: Design axial resistances of welded joints between CHS brace members and CHS chords

Chord face failure - T and Y joints



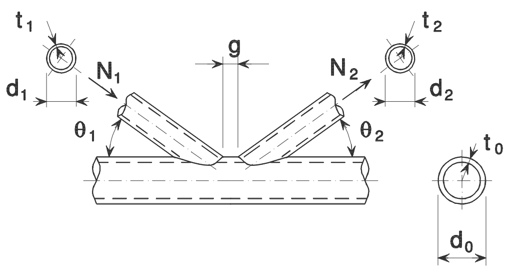
$$N_{1,Rd} = \frac{\gamma^{0,2} k_p f_{y0} t_0^2}{\sin\theta_1}\left(2,8 + 14,2\beta^2\right) \Big/ \gamma_{M5}$$

Chord face failure - X joints



$$N_{1,Rd} = \frac{k_p f_{y0} t_0^2}{\sin\theta_1} \frac{5,2}{(1 - 0,81\beta)} \Big/ \gamma_{M5}$$

Chord face failure - K and N gap or overlap joints



$$N_{1,Rd} = \frac{k_g k_p f_{y0} t_0^2}{\sin\theta_1}\left(1,8 + 10,2\,\frac{d_1}{d_0}\right) \Big/ \gamma_{M5}$$

$$N_{2,Rd} = \frac{\sin\theta_1}{\sin\theta_2} N_{1,Rd}$$

Punching shear failure - K, N and KT gap joints and all T, Y and X joints [i = 1, 2 or 3]

When $d_i \le d_0 - 2t_0$: $N_{i,Rd} = \dfrac{f_{y0}}{\sqrt{3}}\, t_0 \pi d_i \, \dfrac{1 + \sin\theta_i}{2\sin^2\theta_i} \Big/ \gamma_{M5}$

Factors k_g and k_p

$k_g = \gamma^{0,2}\left(1 + \dfrac{0,024\,\gamma^{1,2}}{1 + \exp\left(0,5g/t_0 - 1,33\right)}\right)$ (see figure 7.6)

For $n_p > 0$ (compression): $k_p = 1 - 0,3\,n_p(1 + n_p)$ but $k_p \le 1,0$
For $n_p \le 0$ (tension): $k_p = 1,0$

Table 7.3: Design resistances of welded joints connecting gusset plates to CHS members

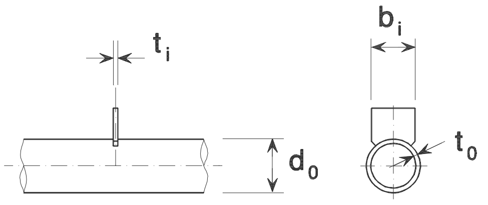
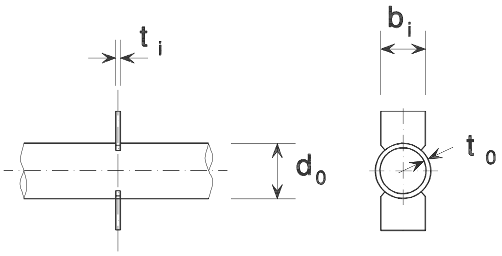
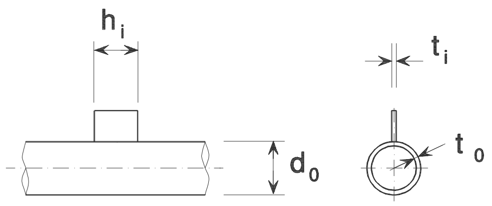
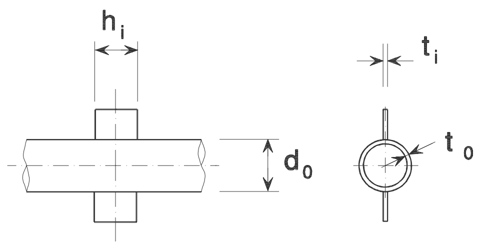
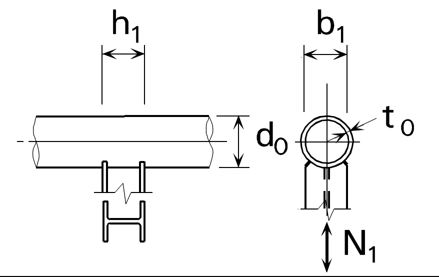
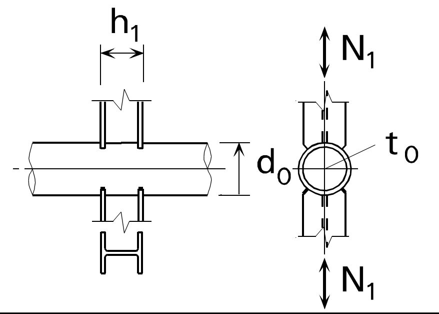
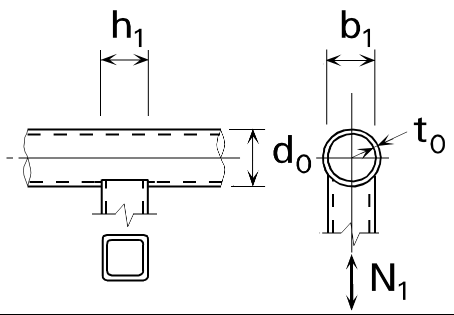
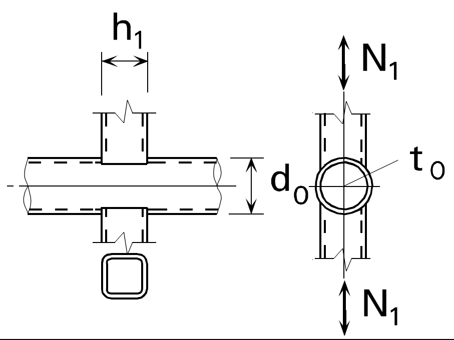
Chord face failure	
	$N_{i,Rd} = k_p f_{y0} t_0^2 \left(4 + 20\beta^2\right) / \gamma_{M5}$ $M_{ip,i,Rd} = 0$ $M_{op,i,Rd} = 0,5\, b_i\, N_{i,Rd}$
	$N_{i,Rd} = \dfrac{5 k_p f_{y0} t_0^2}{1 - 0,81\beta} / \gamma_{M5}$ $M_{ip,i,Rd} = 0$ $M_{op,i,Rd} = 0,5\, b_i\, N_{i,Rd}$
	$N_{i,Rd} = 5 k_p f_{y0} t_0^2 \left(1 + 0,25\eta\right) / \gamma_{M5}$ $M_{ip,i,Rd} = h_i\, N_{i,Rd}$ $M_{op,i,Rd} = 0$
	$N_{i,Rd} = 5 k_p f_{y0} t_0^2 \left(1 + 0,25\eta\right) / \gamma_{M5}$ $M_{ip,i,Rd} = h_i\, N_{i,Rd}$ $M_{op,i,Rd} = 0$
Punching shear failure	
$\sigma_{max}\, t_i = \left(N_{Ed}/A + M_{Ed}/W_{e\ell}\right) t_i \leq 2 t_0 \left(f_{y0}/\sqrt{3}\right) / \gamma_{M5}$	
Range of validity	Factor k_p
In addition to the limits given in table 7.1: $\beta \geq 0,4$ and $\eta \leq 4$ where $\beta = b_i/d_0$ and $\eta = h_i/d_0$	For $n_p > 0$ (compression): $k_p = 1 - 0,3\, n_p (1 + n_p)$ but $k_p \leq 1,0$ For $n_p \leq 0$ (tension): $k_p = 1,0$

Table 7.4: Design resistances of welded joints connecting I, H or RHS sections to CHS members

Chord face failure	
	$N_{1,Rd} = k_p f_{y0} t_0^2 (4 + 20\beta^2)(1 + 0,25\eta) \;/\; \gamma_{M5}$ $M_{ip,1,Rd} = h_1 N_{1,Rd}/(1 + 0,25\eta)$ $M_{op,1,Rd} = 0,5\, b_1 N_{1,Rd}$
	$N_{1,Rd} = \dfrac{5 k_p f_{y0} t_0^2}{1 - 0,81\beta} (1 + 0,25\eta) \;/\; \gamma_{M5}$ $M_{ip,1,Rd} = h_1 N_{1,Rd}/(1 + 0,25\eta)$ $M_{op,1,Rd} = 0,5\, b_1 N_{1,Rd}$
	$N_{1,Rd} = k_p f_{y0} t_0^2 (4 + 20\beta^2)(1 + 0,25\eta) \;/\; \gamma_{M5}$ $M_{ip,1,Rd} = h_1 N_{1,Rd}$ $M_{op,1,Rd} = 0,5\, b_1 N_{1,Rd}$
	$N_{1,Rd} = \dfrac{5 k_p f_{y0} t_0^2}{1 - 0,81\beta} (1 + 0,25\eta) \;/\; \gamma_{M5}$ $M_{ip,1,Rd} = h_1 N_{1,Rd}$ $M_{op,1,Rd} = 0,5\, b_1 N_{1,Rd}$

Punching shear failure	
I or H sections:	$\sigma_{max}\, t_1 = (N_{Ed}/A + M_{Ed}/W_{e\ell})\, t_1 \le 2 t_0 (f_{y0}/\sqrt{3}) \;/\; \gamma_{M5}$
RHS sections:	$\sigma_{max}\, t_1 = (N_{Ed}/A + M_{Ed}/W_{e\ell})\, t_1 \le t_0 (f_{y0}/\sqrt{3}) \;/\; \gamma_{M5}$

Range of validity	Factor k_p
In addition to the limits given in table 7.1:	For $n_p > 0$ (compression):
$\beta \ge 0,4$ and $\eta \le 4$	$k_p = 1 - 0,3\, n_p (1 + n_p)$ but $k_p \le 1,0$
where $\beta = b_1 / d_0$ and $\eta = h_1 / d_0$	For $n_p \le 0$ (tension): $k_p = 1,0$

(3) The design internal moment $M_{i,Ed}$ may be taken as the value at the point where the centreline of the brace member meets the face of the chord member.

(4) The design in-plane moment resistance and the design out-of-plane moment resistance $M_{i,Rd}$ should be obtained from tables 7.3, 7.4 or 7.5 as appropriate.

(5) The special types of welded joints indicated in table 7.6 should satisfy the appropriate design criteria specified for each type in that table.

(6) Values of the factor k_g which is used in table 7.2 for K, N and KT joints are given in figure 7.6. The factor k_g is used to cover both gap type and overlap type joints by adopting g for both the gap and the overlap and using negative values of g to represent the overlap q as defined in figure 1.3(b).

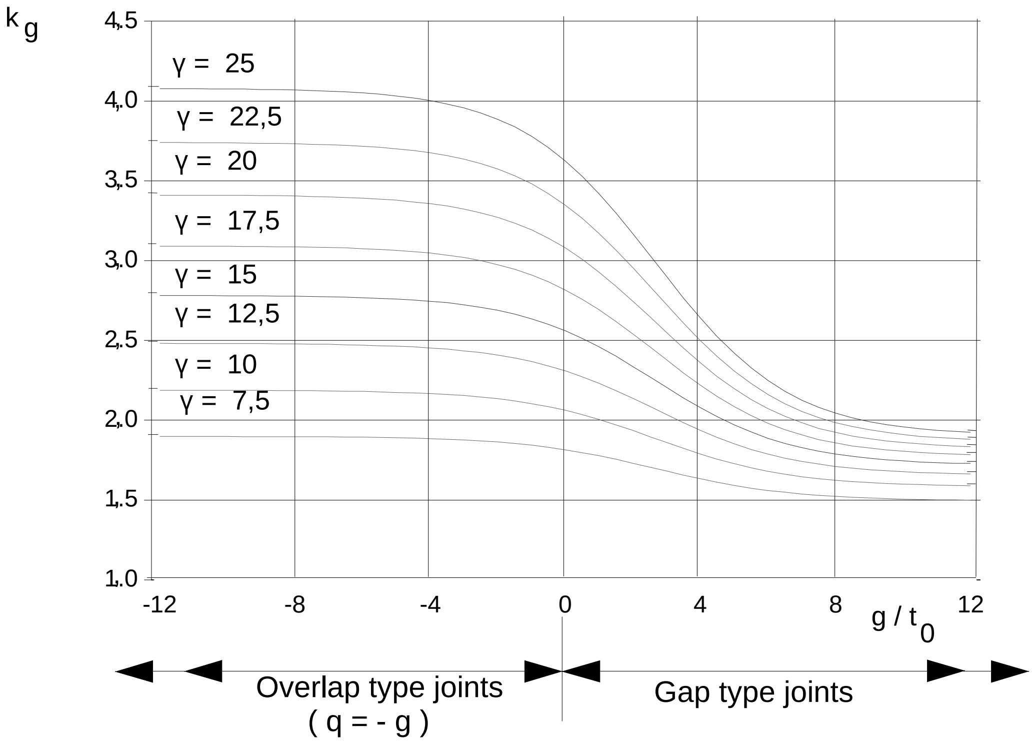
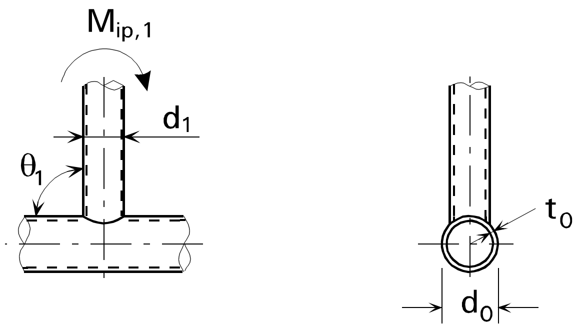
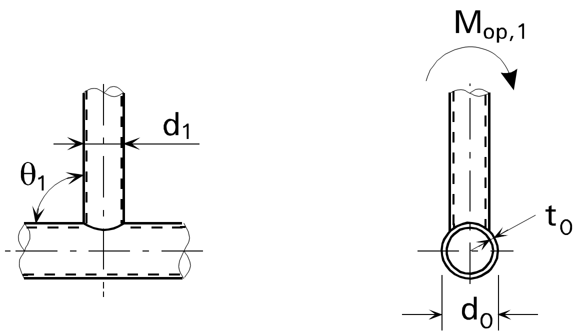


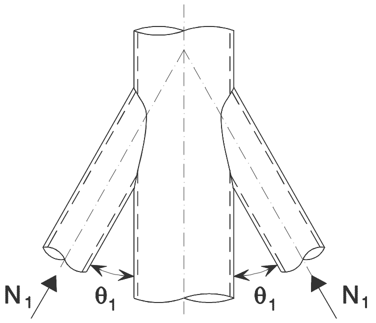
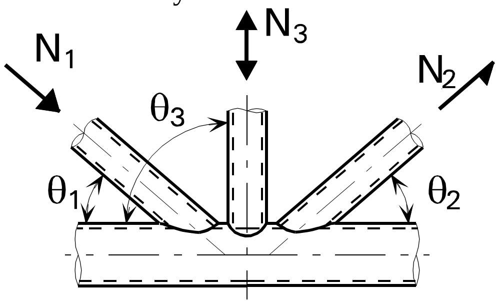
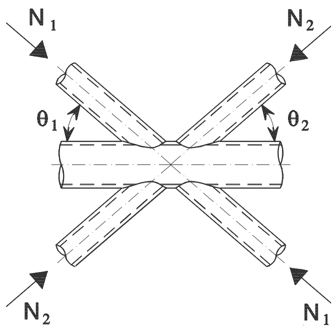
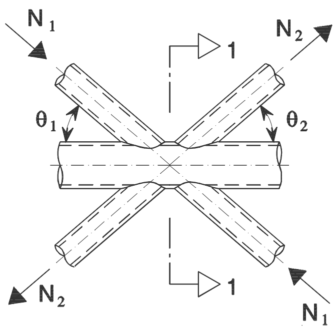
Figure 7.6: Values of the factor k_g for use in table 7.2

Table 7.5: Design resistance moments of welded joints between CHS brace members and CHS chords

<table>
<tr><td colspan="2">Chord face failure - T, X, and Y joints</td></tr>
<tr><td>
</td><td>$M_{ip,1,Rd} = 4{,}85 \dfrac{f_{y0}\, t_0^{\,2} d_1}{\sin\theta_1} \sqrt{\gamma}\,\beta\, k_p \;\Big/ \gamma_{M5}$</td></tr>
<tr><td colspan="2">Chord face failure - K, N, T, X and Y joints</td></tr>
<tr><td>
</td><td>$M_{op,1,Rd} = \dfrac{f_{y0}\, t_0^{\,2} d_1}{\sin\theta_1} \dfrac{2{,}7}{1 - 0{,}81\beta} k_p \;\Big/ \gamma_{M5}$</td></tr>
<tr><td colspan="2">Punching shear failure - K and N gap joints and all T, X and Y joints</td></tr>
<tr><td colspan="2">When $d_1 \le d_0 - 2t_0$: $M_{ip,1,Rd} = \dfrac{f_{y0}\, t_0 d_1^{\,2}}{\sqrt{3}} \dfrac{1 + 3\sin\theta_1}{4\sin^2\theta_1} \;\Big/ \gamma_{M5}$

$M_{op,1,Rd} = \dfrac{f_{y0}\, t_0 d_1^{\,2}}{\sqrt{3}} \dfrac{3 + \sin\theta_1}{4\sin^2\theta_1} \;\Big/ \gamma_{M5}$</td></tr>
<tr><td colspan="2">Factor k_p</td></tr>
<tr><td colspan="2">For $n_p > 0$ (compression): $k_p = 1 - 0{,}3 n_p (1 + n_p)$ but $k_p \le 1{,}0$
For $n_p \le 0$ (tension): $k_p = 1{,}0$</td></tr>
</table>

Table 7.6: Design criteria for special types of welded joints between CHS brace members and CHS chords

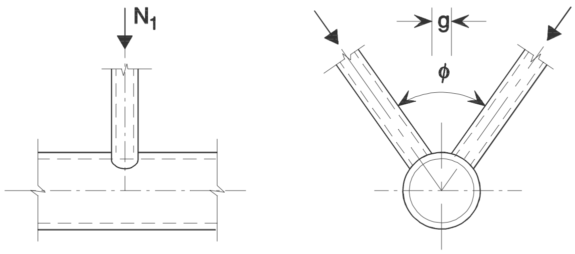
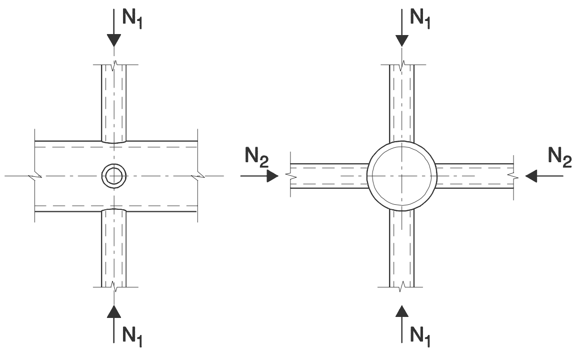
Type of joint	Design criteria
 The forces may be either tension or compression but shall act in the same direction for both members.	$N_{1,Ed} \leq N_{1,Rd}$ in which $N_{1,Rd}$ is the value of $N_{1,Rd}$ for an X joint from table 7.2.
Member 1 is always in compression and member 2 is always in tension. 	$N_{1,Ed} \sin\theta_1 + N_{3,Ed} \sin\theta_3 \leq N_{1,Rd} \sin\theta_1$ $N_{2,Ed} \sin\theta_2 \leq N_{1,Rd} \sin\theta_1$ in which $N_{1,Rd}$ is the value of $N_{1,Rd}$ for a K joint from table 7.2 but with $\dfrac{d_1}{d_0}$ replaced by: $\dfrac{d_1 + d_2 + d_3}{3 d_0}$
All bracing members shall always be in either compression or tension. 	$N_{1,Ed} \sin\theta_1 + N_{2,Ed} \sin\theta_2 \leq N_{x,Rd} \sin\theta_x$ in which $N_{x,Rd}$ is the value of $N_{x,Rd}$ for an X joint from table 7.2, where $N_{x,Rd} \sin\theta_x$ is the larger of: $\lvert N_{1,Rd} \sin\theta_1 \rvert$ and $\lvert N_{2,Rd} \sin\theta_2 \rvert$
Member 1 is always in compression and member 2 is always in tension. 	$N_{i,Ed} \leq N_{i,Rd}$ in which $N_{i,Rd}$ is the value of $N_{i,Rd}$ for a K joint from table 7.2, provided that, in a gap-type joint, at section 1-1 the chord satisfies: $\left[\dfrac{N_{0,Ed}}{N_{0,p\ell,Rd}}\right]^2 + \left[\dfrac{V_{0,Ed}}{V_{0,p\ell,Rd}}\right]^2 \leq 1{,}0$

7.4.3 Multiplanar joints

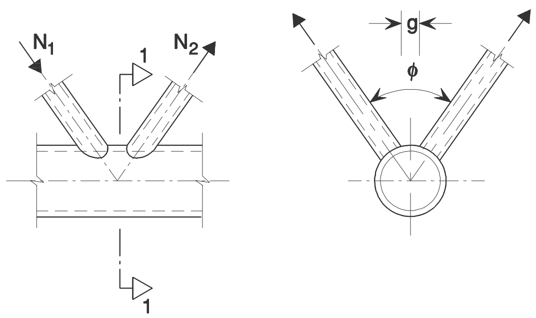
(1) In each relevant plane of a multiplanar joint, the design criteria given in 7.4.2 should be satisfied using the reduced design resistances obtained from (2).

(2) The design resistances for each relevant plane of a multiplanar joint should be determined by applying the appropriate reduction factor μ given in table 7.7 to the resistance of the corresponding uniplanar joint calculated according to 7.4.2 by using the appropriate chord force for k_p .

Table 7.7: Reduction factors for multiplanar joints.

<table>
<tr><th>Type of joint</th><th>Reduction factor μ</th></tr>
<tr><td colspan="2">TT joint $60° \leq \varphi \leq 90°$</td></tr>
<tr><td>Member 1 may be either tension or compression.

</td><td>$\mu = 1,0$</td></tr>
<tr><td colspan="2">XX joint</td></tr>
<tr><td>Members 1 and 2 can be either in compression or tension. $N_{2,\mathrm{Ed}}/N_{1,\mathrm{Ed}}$ is negative if one member is in tension and one in compression.

</td><td>$\mu = 1 + 0,33\ N_{2,Ed}/N_{1,Ed}$

taking account of the sign of $N_{1,\mathrm{Ed}}$ and $N_{2,\mathrm{Ed}}$

where $|N_{2,\mathrm{Ed}}| \leq |N_{1,\mathrm{Ed}}|$</td></tr>
<tr><td colspan="2">KK joint $60° \leq \varphi \leq 90°$</td></tr>
<tr><td>Member 1 is always in compression and member 2 is always in tension.

</td><td>$\mu = 0,9$

provided that, in a gap-type joint, at section 1-1 the chord satisfies:

$$\left[\frac{N_{0,\mathrm{Ed}}}{N_{\mathrm{p}\ell,0,\mathrm{Rd}}}\right]^2 + \left[\frac{V_{0,\mathrm{Ed}}}{V_{\mathrm{p}\ell,0,\mathrm{Rd}}}\right]^2 \leq 1,0$$</td></tr>
</table>

7.5 Welded joints between CHS or RHS brace members and RHS chord members

7.5.1 General

(1) Provided that the geometry of the joints is within the range of validity given in table 7.8, the design resistances of welded joints between hollow section brace members and rectangular or square hollow section chord members may be determined using 7.5.2 and 7.5.3.

(2) For joints within the range of validity given in table 7.8, only the design criteria covered in the appropriate table need be considered. The design resistance of a connection should be taken as the minimum value for all applicable criteria.

(3) For joints outside the range of validity given in table 7.8, all the criteria given in 7.2.2 should be considered. In addition, the secondary moments in the joints caused by their rotational stiffness should be taken into account.

Table 7.8: Range of validity for welded joints between CHS or RHS brace members and RHS chord members.

<table>
<tr><th rowspan="3">Type of joint</th><th colspan="6">Joint parameters [i = 1 or 2, j = overlapped brace]</th></tr>
<tr><th rowspan="2">b_i/b_0
or
d_i/b_0</th><th colspan="2">b_i/t_i and h_i/t_i or d_i/t_i</th><th rowspan="2">h_0/b_0
and
h_i/b_i</th><th rowspan="2">b_0/t_0
and
h_0/t_0</th><th rowspan="2">Gap or overlap

b_i/b_j</th></tr>
<tr><th>Compression</th><th>Tension</th></tr>
<tr><td>T, Y or X</td><td>$b_i/b_0 \geq 0,25$</td><td rowspan="2">$b_i/t_i \leq 35$
and
$h_i/t_i \leq 35$
and
Class 2</td><td rowspan="3">b_i/t_i
≤ 35
and
h_i/t_i
≤ 35</td><td rowspan="3">$\geq 0,5$
but
$\leq 2,0$</td><td>≤ 35
and
Class 2</td><td>–</td></tr>
<tr><td>K gap
N gap</td><td>$b_i/b_0 \geq 0,35$
and
$\geq 0,1 + 0,01\, b_0/t_0$</td><td>$\leq 35$
and
Class 2</td><td>$g/b_0 \geq 0,5(1-\beta)$
but $\leq 1,5(1-\beta)$ 1)

and as minimum
$g \geq t_1 + t_2$</td></tr>
<tr><td>K overlap
N overlap</td><td>$b_i/b_0 \geq 0,25$</td><td>Class 1</td><td>Class 2</td><td>$\lambda_{ov} \geq 25\%$
but $\lambda_{ov} \leq 100\%$ 2)

and $b_i/b_j \geq 0,75$</td></tr>
<tr><td>Circular brace member</td><td>$d_i/b_0 \geq 0,4$

but $\leq 0,8$</td><td>Class 1</td><td>d_i/t_i
≤ 50</td><td colspan="3">As above but with d_i replacing b_i and d_j replacing b_j.</td></tr>
<tr><td colspan="7">1) If $g/b_0 > 1,5(1-\beta)$ and $g/b_0 > t_1 + t_2$ treat the joint as two separate T or Y joints.

2) The overlap may be increased to enable the toe of the overlapped brace to be welded to the chord.</td></tr>
</table>

7.5.2 Uniplanar joints

7.5.2.1 Unreinforced joints

(1) In brace member connections subject only to axial forces, the design internal axial force $N_{i,Ed}$ should not exceed the design axial resistance of the welded joint $N_{i,Rd}$, determined from (2) or (4) as appropriate.

(2) For welded joints between square or circular hollow section brace members and square hollow section chord members only, where the geometry of the joints is within the range of validity given in table 7.8 and also satisfies the additional conditions given in table 7.9, the design axial resistances may be determined from the expressions given in table 7.10.

(3) For joints within the range of validity of table 7.9, the only design criteria that need be considered are chord face failure and brace failure with reduced effective width. The design axial resistance should be taken as the minimum value for these two criteria.

> **NOTE:** The design axial resistances for joints of hollow section brace members to square hollow section chords given in table 7.10 have been simplified by omitting design criteria that are never decisive within the range of validity of table 7.9.

(4) The design axial resistances of any unreinforced welded joint between CHS or RHS brace members and RHS chords, within the range of validity of table 7.8, may be determined using the expressions given in tables 7.11, 7.12 or 7.13 as appropriate. For reinforced joints see 7.5.2.2.

Table 7.9: Additional conditions for the use of table 7.10

Type of brace	Type of joint	Joint parameters	
Square hollow section	T, Y or X	$b_i/b_0 \leq 0,85$	$b_0/t_0 \geq 10$
	K gap or N gap	$0,6 \leq \dfrac{b_1 + b_2}{2\ b_1} \leq 1,3$	$b_0/t_0 \geq 15$
Circular hollow section	T, Y or X	—	$b_0/t_0 \geq 10$
	K gap or N gap	$0,6 \leq \dfrac{d_1 + d_2}{2\ d_1} \leq 1,3$	$b_0/t_0 \geq 15$

Table 7.10: Design axial resistances of welded joints between square or circular hollow section braces and square hollow section chords

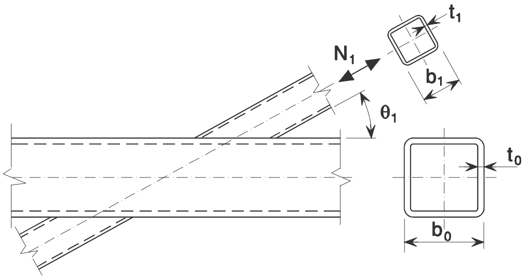
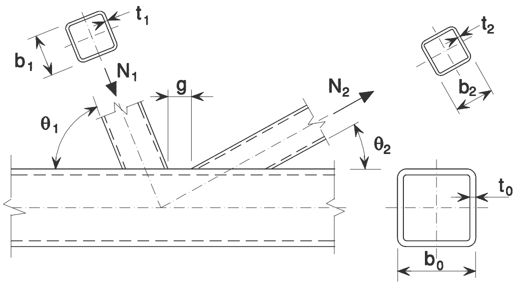
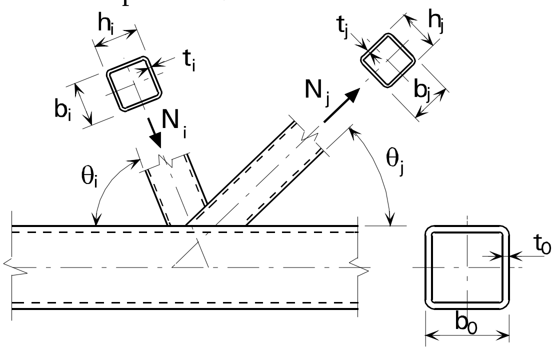
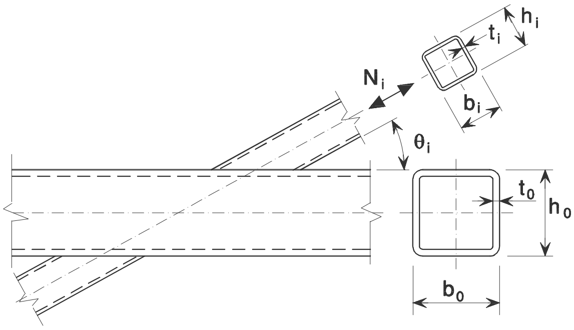
Type of joint	Design resistance [i = 1 or 2, j = overlapped brace]
T, Y and X joints	Chord face failure $\beta \leq 0,85$
	$N_{1,Rd} = \frac{k_n f_{y0} t_0^2}{(1-\beta)\sin\theta_1}\left(\frac{2\beta}{\sin\theta_1} + 4\sqrt{1-\beta}\right) / \gamma_{M5}$
K and N gap joints	Chord face failure $\beta \leq 1,0$
	$N_{i,Rd} = \frac{8,9\gamma^{0,5} k_n f_{y0} t_0^2}{\sin\theta_i}\left(\frac{b_1 + b_2}{2b_0}\right) / \gamma_{M5}$
K and N overlap joints *)	Brace failure $25\% \leq \lambda_{ov} < 50\%$
Member i or member j may be either tension or compression but one shall be tension and the other compression. 	$N_{i,Rd} = f_{yi} t_i \left(b_{eff} + b_{e,ov} + \frac{\lambda_{ov}}{50}(2h_i - 4t_i)\right) / \gamma_{M5}$
	Brace failure $50\% \leq \lambda_{ov} < 80\%$
	$N_{i,Rd} = f_{yi} t_i \left[b_{eff} + b_{e,ov} + 2h_i - 4t_i\right] / \gamma_{M5}$
	Brace failure $\lambda_{ov} \geq 80\%$
	$N_{i,Rd} = f_{yi} t_i \left[b_i + b_{e,ov} + 2h_i - 4t_i\right] / \gamma_{M5}$
Parameters b_{eff}, $b_{e,ov}$ and k_n	
$b_{eff} = \frac{10}{b_0/t_0}\frac{f_{y0} t_0}{f_{yi} t_i} b_i$ but $b_{eff} \leq b_i$	For $n > 0$ (compression): $k_n = 1,3 - \frac{0,4n}{\beta}$ but $k_n \leq 1,0$
$b_{e,ov} = \frac{10}{b_j/t_j}\frac{f_{yj} t_j}{f_{yi} t_i} b_i$ but $b_{e,ov} \leq b_i$	For $n \leq 0$ (tension): $k_n = 1,0$
For circular braces, multiply the above resistances by $\pi/4$, replace b_1 and h_1 by d_1 and replace b_2 and h_2 by d_2.	
*) Only the overlapping brace member need be checked. The brace member efficiency (i.e. the design resistance of the joint divided by the design plastic resistance of the brace member) of the overlapped brace member should be taken as equal to that of the overlapping brace member.	

Table 7.11: Design axial resistances of welded T, X and Y joints between RHS or CHS braces and RHS chords

Type of joint	Design resistance [i = 1]
	Chord face failure $\beta \leq 0{,}85$
	$N_{i,Rd} = \frac{k_n f_{y0} t_0^2}{(1-\beta)\sin\theta_i}\left(\frac{2\eta}{\sin\theta_i} + 4\sqrt{1-\beta}\right) / \gamma_{M5}$
	Chord side wall buckling [1] $\beta = 1{,}0$ [2]
	$N_{i,Rd} = \frac{f_b t_0}{\sin\theta_i}\left(\frac{2h_i}{\sin\theta_i} + 10 t_0\right) / \gamma_{M5}$
	Brace failure $\beta \geq 0{,}85$
	$N_{i,Rd} = f_{yi} t_i \left(2h_i - 4t_i + 2b_{eff}\right) / \gamma_{M5}$
	Punching shear $0{,}85 \leq \beta \leq (1 - 1/\gamma)$
	$N_{i,Rd} = \frac{f_{y0} t_0}{\sqrt{3}\sin\theta_i}\left(\frac{2h_i}{\sin\theta_i} + 2b_{e,p}\right) / \gamma_{M5}$

[1] For X joints with $\theta < 90°$ use the smaller of this value and the shear resistance of the chord side walls given for K and N gap joints in table 7.12.
[2] For $0{,}85 \leq \beta \leq 1{,}0$ use linear interpolation between the value for chord face failure at $\beta = 0{,}85$ and the governing value for chord side wall failure at $\beta = 1{,}0$ (side wall buckling or chord shear).

For circular braces, multiply the above resistances by $\pi/4$, replace b_1 and h_1 by d_1 and replace b_2 and h_2 by d_2.

For tension: $f_b = f_{y0}$ For compression: $f_b = \chi f_{y0}$ (T and Y joints) $f_b = 0{,}8 \chi f_{y0} \sin\theta_i$ (X joints) where χ is the reduction factor for flexural buckling obtained from EN 1993-1-1 using the relevant buckling curve and a normalized slenderness $\bar{\lambda}$ determined from: $\bar{\lambda} = 3{,}46 \frac{\left(\frac{h_0}{t_0} - 2\right)\sqrt{\frac{1}{\sin\theta_i}}}{\pi\sqrt{\frac{E}{f_{y0}}}}$	$b_{eff} = \frac{10}{b_0/t_0} \frac{f_{y0}\, t_0}{f_{yi}\, t_i} b_i$ but $b_{eff} \leq b_i$ $b_{e,p} = \frac{10}{b_0/t_0} b_i$ but $b_{e,p} \leq b_i$ For $n > 0$ (compression): $k_n = 1{,}3 - \frac{0{,}4n}{\beta}$ but $k_n \leq 1{,}0$ For $n \leq 0$ (tension): $k_n = 1{,}0$

Table 7.12: Design axial resistances of welded K and N joints between RHS or CHS braces and RHS chords

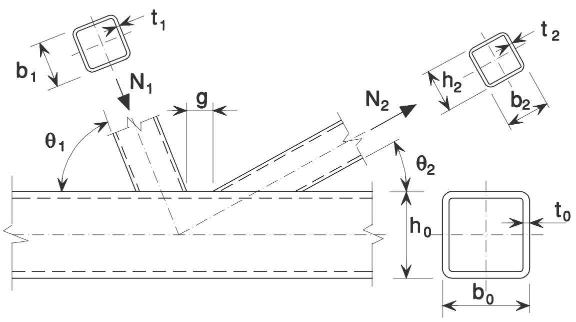
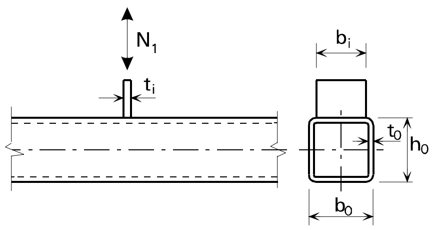
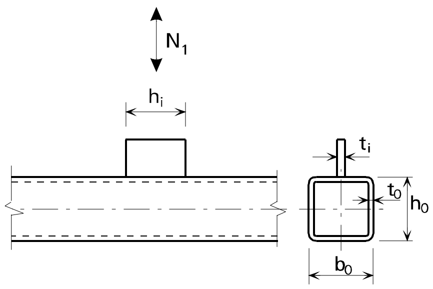
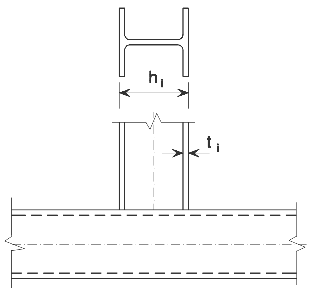
Type of joint	Design resistance [i = 1 or 2]
K and N gap joints	Chord face failure
	$N_{i,Rd} = \dfrac{8{,}9 k_n f_{y0} t_0^2 \sqrt{\gamma}}{\sin\theta_i}\left(\dfrac{b_1 + b_2 + h_1 + h_2}{4 b_0}\right) / \gamma_{M5}$
	Chord shear
	$N_{i,Rd} = \dfrac{f_{y0} A_v}{\sqrt{3}\sin\theta_i} / \gamma_{M5}$ $N_{0,Rd} = \left[(A_0 - A_v) f_{y0} + A_v f_{y0}\sqrt{1-(V_{Sd}/V_{pl,Rd})^2}\right] / \gamma_{M5}$
	Brace failure
	$N_{i,Rd} = f_{yi} t_i (2h_i - 4t_i + b_i + b_{eff}) / \gamma_{M5}$
	Punching shear $\beta \le (1 - 1/\gamma)$
	$N_{i,Rd} = \dfrac{f_{y0} t_0}{\sqrt{3}\sin\theta_i}\left(\dfrac{2h_i}{\sin\theta_i} + b_i + b_{e,p}\right) / \gamma_{M5}$
K and N overlap joints	As in table 7.10.
For circular braces, multiply the above resistances by $\pi/4$, replace b_1 and h_1 by d_1 and replace b_2 and h_2 by d_2.	
$A_v = (2h_0 + \alpha b_0)t_0$ For a square or rectangular brace member: $\alpha = \sqrt{\dfrac{1}{1 + \dfrac{4g^2}{3t_0^2}}}$ where g is the gap, see figure 1.3(a). For a circular brace member: $\alpha = 0$	$b_{eff} = \dfrac{10}{b_0/t_0}\dfrac{f_{y0}\,t_0}{f_{yi}\,t_i} b_i$ but $b_{eff} \le b_i$ $b_{e,p} = \dfrac{10}{b_0/t_0} b_i$ but $b_{e,p} \le b_i$ For $n > 0$ (compression): $k_n = 1{,}3 - \dfrac{0{,}4n}{\beta}$ but $k_n \le 1{,}0$ For $n \le 0$ (tension): $k_n = 1{,}0$

Table 7.13: Design resistances of welded joints connecting gusset plates or I or H sections to RHS members

Transverse plate	Brace failure $[i = 1]$
	$N_{1,Rd} = f_{y1} t_1 b_{eff} \,/\, \gamma_{M5}$ *) Chord side wall crushing when $b_1 \geq b_0 - 2t_0$ $N_{1,Rd} = f_{y0} t_0 \left(2t_1 + 10t_0\right) \,/\, \gamma_{M5}$ Punching shear when $b_1 \leq b_0 - 2t_0$ $N_{1,Rd} = \frac{f_{y0} t_0}{\sqrt{3}} \left(2t_1 + 2b_{e,p}\right) \,/\, \gamma_{M5}$
Longitudinal plate	Chord face failure $\beta \leq 0{,}85$
 $t_1/b_0 \leq 0{,}2$	$N_{1,Rd} = \frac{k_m f_{y0} t_0^2}{1 - t_1/b_0} \left(2h_1/b_0 + 4\sqrt{1 - t_1/b_0}\right) \,/\, \gamma_{M5}$
I or H section	
	Conservatively, base $N_{1,Rd}$ for an I or H section upon the design resistance of two transverse plates similar to its flanges, determined as specified above. $M_{ip,1,Rd} = N_{1,Rd}\ (h_1 - t_1)$
Range of validity	
In addition to the limits given in table 7.8: $0{,}5 \leq \beta \leq 1{,}0$ $b_0/t_0 \leq 30$	
Parameters b_{eff}, $b_{e,p}$ and k_m	
$b_{eff} = \frac{10}{b_0/t_0} \frac{f_{y0} t_0}{f_{y1} t_1} b_1$ but $b_{eff} \leq b_1$	For $n > 0$ (compression): $k_m = 1{,}3(1 - n)$ but $k_m \leq 1{,}0$
$b_{e,p} = \frac{10}{b_0/t_0} b_1$ but $b_{e,p} \leq b_1$	For $n \leq 0$ (tension): $k_m = 1{,}0$
*) Fillet welded connections should be designed in accordance with 4.10.	

(5) Brace member connections subjected to combined bending and axial force should satisfy:

$$\frac{N_{i,Ed}}{N_{i,Rd}} + \frac{M_{ip,i,Ed}}{M_{ip,i,Rd}} + \frac{M_{op,i,Ed}}{M_{op,i,Rd}} \leq 1,0 \qquad \text{... (7.4)}$$

where: $M_{ip,i,Rd}$ is the design in-plane moment resistance

$M_{ip,i,Ed}$ is the design in-plane internal moment

$M_{op,i,Rd}$ is the design out-of-plane moment resistance

$M_{op,i,Ed}$ is the design out-of-plane internal moment

(6) The design internal moment $M_{i,Ed}$ may be taken as the value at the point where the centreline of the brace member meets the face of the chord member.

(7) For unreinforced joints, the design in-plane moment resistance and out-of-plane moment resistance $M_{i,Rd}$ should be obtained from tables 7.13 or 7.14 as appropriate. For reinforced joints see 7.5.2.2.

(8) The special types of welded joints indicated in tables 7.15 and 7.16 should satisfy the appropriate design criteria specified for each type in that table.

7.5.2.2 Reinforced joints

(1) Various types of joint reinforcement may be used. The appropriate type depends upon the failure mode that, in the absence of reinforcement, governs the design resistance of the joint.

(2) Flange reinforcing plates may be used to increase the resistance of the joint to chord face failure, punching shear failure or brace failure with reduced effective width.

(3) A pair of side plates may be used to reinforce a joint against chord side wall failure or chord shear failure.

(4) In order to avoid partial overlapping of brace members in a K or N joint, the brace members may be welded to a vertical stiffener.

(5) Any combinations of these types of joint reinforcement may also be used.

(6) The grade of steel used for the reinforcement should not be lower than that of the chord member.

(7) The design resistances of reinforced joints should be determined using tables 7.17 and 7.18.

Table 7.14: Design resistance moments of welded joints between RHS brace members and RHS chords

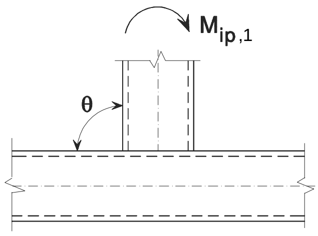
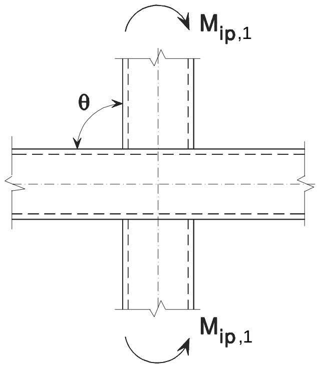
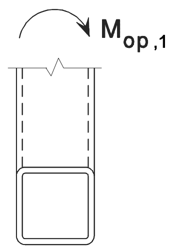
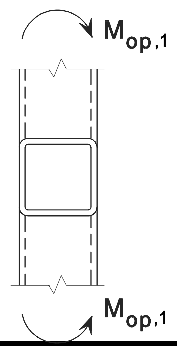
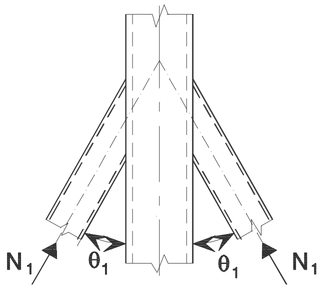
T and X joints	Design resistance
In-plane moments ($\theta \approx 90°$)	Chord face failure $\beta \leq 0,85$
	$M_{ip,1,Rd} = k_n f_{y0} t_0^{\,2} h_1 \left(\frac{1}{2\eta} + \frac{2}{\sqrt{1-\beta}} + \frac{\eta}{1-\beta} \right) / \gamma_{M5}$
	Chord side wall crushing $0,85 \leq \beta \leq 1,0$
	$M_{ip,1,Rd} = 0,5\, f_{yk}\, t_0 \left(h_1 + 5\, t_0\right)^2 / \gamma_{M5}$ $f_{yk} = f_{y0}$ for T joints $f_{yk} = 0,8\, f_{y0}$ for X joints
	Brace failure $0,85 \leq \beta \leq 1,0$
	$M_{ip,1,Rd} = f_{y1} \left(W_{p\ell,1} - (1 - b_{eff}/b_1)\, b_1\, h_1\, t_1\right) / \gamma_{M5}$
Out-of-plane moments ($\theta \approx 90°$)	Chord face failure $\beta \leq 0,85$
	$M_{op,1,Rd} = k_n\, f_{y0}\, t_0^{\,2} \left(\frac{h_1 (1+\beta)}{2(1-\beta)} + \sqrt{\frac{2\, b_0\, b_1 (1+\beta)}{1-\beta}} \right) / \gamma_{M5}$
	Chord side wall crushing $0,85 \leq \beta \leq 1,0$
	$M_{op,1,Rd} = f_{yk}\, t_0 (b_0 - t_0)(h_1 + 5t_0) / \gamma_{M5}$ $f_{yk} = f_{y0}$ for T joints $f_{yk} = 0,8\, f_{y0}$ for X joints
	Chord distortional failure (T joints only) *)
	$M_{op,1,Rd} = 2\, f_{y0}\, t_0 \left(h_1 t_0 + \sqrt{b_0 h_0 t_0 \left(b_0 + h_0\right)}\right) / \gamma_{M5}$
	Brace failure $0,85 \leq \beta \leq 1,0$
	$M_{op,1,Rd} = f_{y1} \left(W_{p\ell,1} - 0,5\,(1 - b_{eff}/b_1)^2\, b_1^{\,2}\, t_1\right) / \gamma_{M5}$
Parameters b_{eff} and k_n	
$b_{eff} = \frac{10}{b_0/t_0} \frac{f_{y0}\, t_0}{f_{y1}\, t_1} b_1$ but $b_{eff} \leq b_1$	For $n > 0$ (compression): $k_n = 1,3 - \frac{0,4\, n}{\beta}$ but $k_n \leq 1,0$ For $n \leq 0$ (tension): $k_n = 1,0$
*) This criterion does not apply where chord distortional failure is prevented by other means.	

Table 7.15: Design criteria for special types of welded joints between RHS brace members and RHS chords

<table>
<tr><th>Type of joint</th><th>Design criteria</th></tr>
<tr><td>

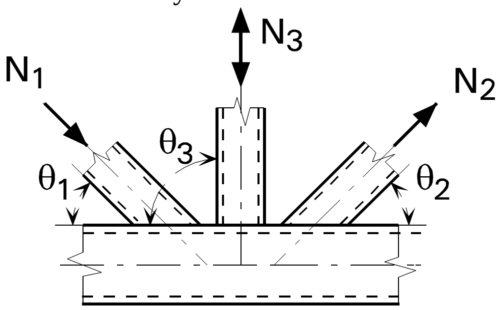
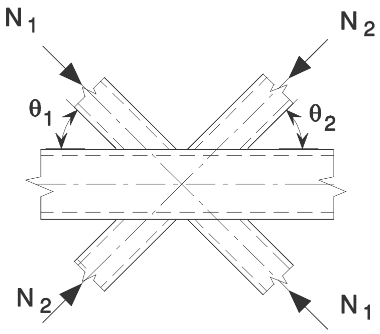
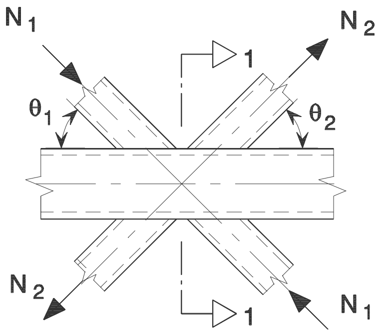
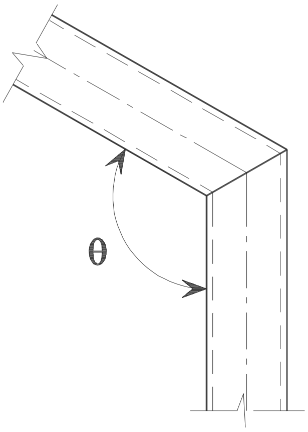
The members may be in either tension or compression and shall act as in the same direction for both members.</td><td>$$N_{1,\mathrm{Ed}} \leq N_{1,\mathrm{Rd}}$$
in which $N_{1,\mathrm{Rd}}$ is the value of $N_{1,\mathrm{Rd}}$ for an X joint from table 7.11.</td></tr>
<tr><td>The member 1 is always in compression and member 2 is always in tension.

</td><td>$$N_{1,\mathrm{Ed}} \sin\theta_1 + N_{3,\mathrm{Ed}} \sin\theta_3 \leq N_{1,\mathrm{Rd}} \sin\theta_1$$
$$N_{2,\mathrm{Ed}} \sin\theta_2 \leq N_{1,\mathrm{Rd}} \sin\theta_1$$
in which $N_{1,\mathrm{Rd}}$ is the value of $N_{1,\mathrm{Rd}}$ for a K joint from table 7.12, but with $\dfrac{b_1 + b_2 + h_1 + h_2}{4\, b_0}$
replaced by:
$$\frac{b_1 + b_2 + b_3 + h_1 + h_2 + h_3}{6\, b_0}$$</td></tr>
<tr><td>All bracing members shall be either compression or tension.

</td><td>$$N_{1,\mathrm{Ed}} \sin\theta_1 + N_{2,\mathrm{Ed}} \sin\theta_2 \leq N_{x,\mathrm{Rd}} \sin\theta_x$$
in which $N_{x,\mathrm{Rd}}$ is the value of $N_{x,\mathrm{Rd}}$ for an X joint from table 7.11, where $N_{x,\mathrm{Rd}} \sin\theta_x$ is the larger of:
$$\left| N_{1,\mathrm{Rd}} \sin\theta_1 \right| \quad \text{and} \quad \left| N_{2,\mathrm{Rd}} \sin\theta_2 \right|$$</td></tr>
<tr><td>Member 1 is always in compression and member 2 is always in tension.

</td><td>$$N_{1,\mathrm{Ed}} \leq N_{1,\mathrm{Rd}}$$
in which $N_{i,\mathrm{Rd}}$ is the value of $N_{i,\mathrm{Rd}}$ for a K joint from table 7.12, provided that, in a gap-type joint, at section 1-1 the chord satisfies:
$$\left[\frac{N_{0,\mathrm{Ed}}}{N_{0,\mathrm{p}\ell,\mathrm{Rd}}}\right]^2 + \left[\frac{V_{0,\mathrm{Ed}}}{V_{0,\mathrm{p}\ell,\mathrm{Rd}}}\right]^2 \leq 1{,}0$$</td></tr>
</table>

Table 7.16: Design criteria for welded knee joints and cranked-chord joints in RHS members

<table>
<tr><th>Type of joint</th><th>Criteria</th></tr>
<tr><td colspan="2">Welded knee joints</td></tr>
<tr><td>


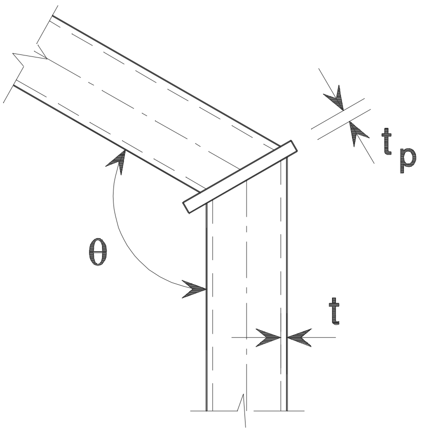
</td><td>
The cross-section should be Class 1 for pure bending, see EN 1993-1-1.

$$N_{Ed} \le 0,2 N_{p\ell,Rd}$$

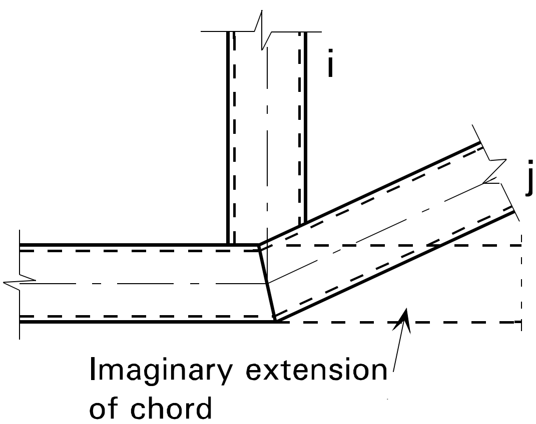
and $$\frac{N_{Ed}}{N_{p\ell,Rd}} + \frac{M_{Ed}}{M_{p\ell,Rd}} \le \kappa$$

If $\theta \le 90^\circ$: $$\kappa = \frac{3\sqrt{b_0/h_0}}{\left[b_0/t_0\right]^{0,8}} + \frac{1}{1 + 2b_0/h_0}$$

If $90^\circ < \theta \le 180^\circ$: $\kappa = 1 - \left(\sqrt{2}\cos(\theta/2)\right)\left(1 - \kappa_{90}\right)$

where κ_{90} is the value of κ for $\theta = 90^\circ$.
</td></tr>
<tr><td>


</td><td>
$$t_p \ge 1,5t \quad \text{and} \quad \ge 10\,\text{mm}$$

$$\frac{N_{Ed}}{N_{p\ell,Rd}} + \frac{M_{Ed}}{M_{p\ell,Rd}} \le 1,0$$
</td></tr>
<tr><td colspan="2">Cranked-chord</td></tr>
<tr><td>


</td><td>
$$N_{i,Ed} \le N_{i,Rd}$$

where $N_{i,Rd}$ is the value of $N_{i,Rd}$ for a K or N overlap joint from table 7.12.
</td></tr>
</table>

Table 7.17: Design resistances of reinforced welded T, Y and X joints between RHS or CHS brace members and RHS chords

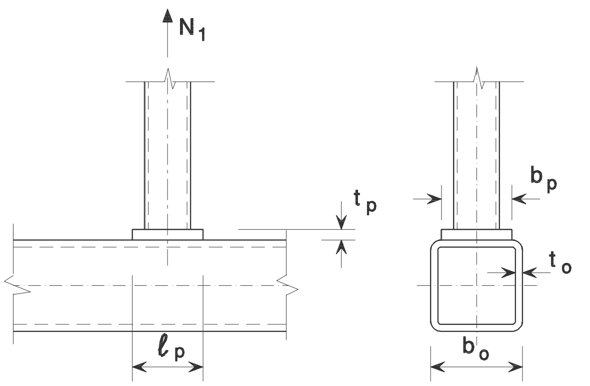
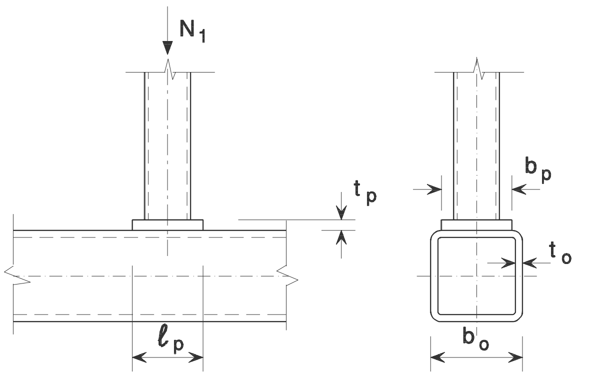
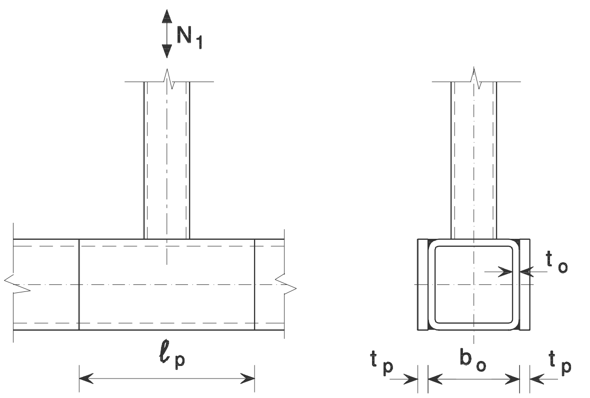
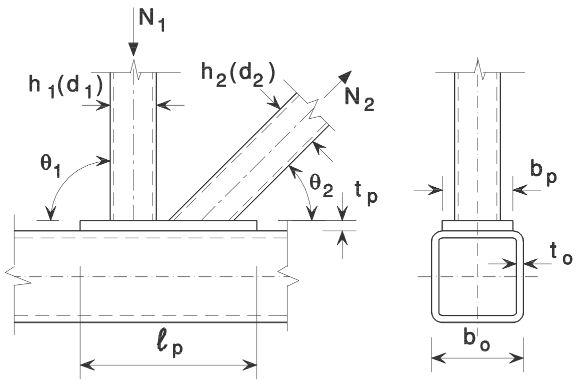
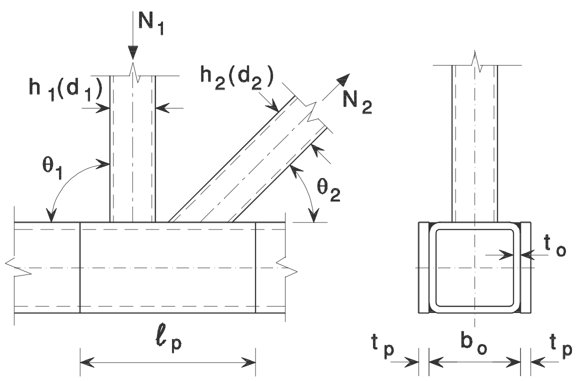
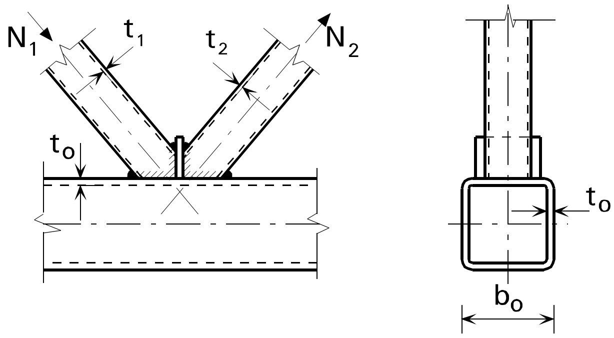
Type of joint	Design resistance [i = 1]
Reinforced with flange plates to avoid chord face failure, brace failure or punching shear.	
Tension loading	$\beta_p \leq 0{,}85$
 	$\ell_p \geq \dfrac{h_i}{\sin\theta_i} + \sqrt{b_p(b_p - b_i)}$ and $\geq 1{,}5 h_i/\sin\theta_i$ $b_p \geq b_0 - 2t_0$ $N_{i,Rd} = \dfrac{f_{yp} t_p^{\,2}}{(1 - b_i/b_p)\sin\theta_i} \times$ $\times \left(\dfrac{2 h_i/b_p}{\sin\theta_i} + 4\sqrt{1 - b_i/b_p} \right) / \gamma_{M5}$
Compression loading	$\beta_p \leq 0{,}85$
 	$\ell_p \geq \dfrac{h_i}{\sin\theta_i} + \sqrt{b_p(b_p - b_i)}$ and $\geq 1{,}5 h_i/\sin\theta_i$ $b_p \geq b_0 - 2t_0$ Take $N_{i,Rd}$ as the value of $N_{i,Rd}$ for a T, X or Y joint from table 7.11, but with $k_n = 1{,}0$ and t_0 replaced by t_p for chord face failure, brace failure and punching shear only.
Reinforced with side plates to avoid chord side wall buckling or chord side wall shear.	
 	$\ell_p \geq 1{,}5 h_i/\sin\theta_i$ Take $N_{i,Rd}$ as the value of $N_{i,Rd}$ for a T, X or Y joint from table 7.11, but with t_0 replaced by $(t_0 + t_p)$ for chord side wall buckling failure and chord side wall shear failure only.

Table 7.18: Design resistances of reinforced welded K and N joints between RHS or CHS brace members and RHS chords

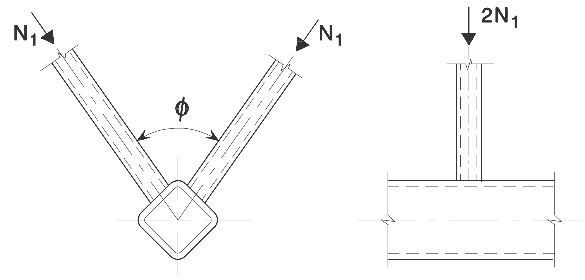
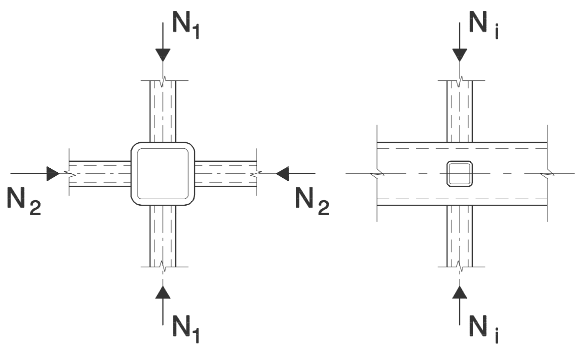
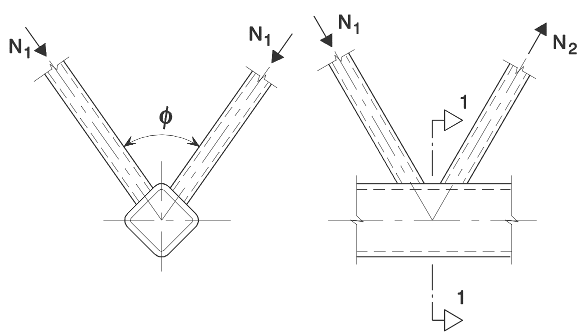
Type of joint	Design resistance [i = 1 or 2]
Reinforced with flange plates to avoid chord face failure, brace failure or punching shear.	
	$\ell_p \geq 1,5\left(\frac{h_1}{\sin\theta_1} + g + \frac{h_2}{\sin\theta_2}\right)$ $b_p \geq b_0 - 2t_0$ $t_p \geq 2t_1$ and $2t_2$ Take $N_{i,Rd}$ as the value of $N_{i,Rd}$ for a K or N joint from table 7.12, but with t_0 replaced by t_p for chord face failure, brace failure and punching shear only.
Reinforced with a pair of side plates to avoid chord shear failure.	
	$\ell_p \geq 1,5\left(\frac{h_1}{\sin\theta_1} + g + \frac{h_2}{\sin\theta_2}\right)$ Take $N_{i,Rd}$ as the value of $N_{i,Rd}$ for a K or N joint from table 7.12, but with t_0 replaced by $(t_0 + t_p)$ for chord shear failure only.
Reinforced by a division plate between the brace members because of insufficient overlap.	
	$t_p \geq 2t_1$ and $2t_2$ Take $N_{i,Rd}$ as the value of $N_{i,Rd}$ for a K or N overlap joint from table 7.12 with $\lambda_{ov} < 80\%$, but with b_j, t_j and f_{yj} replaced by b_p, t_p and f_{yp} in the expression for $b_{e,ov}$ given in table 7.10.

7.5.3 Multiplanar joints

(1) In each relevant plane of a multiplanar joint, the design criteria given in 7.5.2 should be satisfied using the reduced design resistances obtained from (2).

(2) The design resistances for each relevant plane of a multiplanar joint should be determined by applying the appropriate reduction factor μ given in table 7.19 to the resistance of the corresponding uniplanar joint calculated according to 7.5.2 with the appropriate chord load in the multiplanr situation.

Table 7.19: Reduction factors for multiplanar joints

Type of joint	Reduction factor μ
TT joint	$60^\circ \leq \varphi \leq 90^\circ$
Member 1 may be either tension or compression.  	$\mu = 0{,}9$
XX joint	
Members 1 and 2 can be either in compression or tension. $N_{2,\mathrm{Ed}}/N_{1,\mathrm{Ed}}$ is negative if one member is in tension and one in compression.  	$\mu = 0{,}9\left(1 + 0{,}33 N_{2,\mathrm{Ed}}/N_{1,\mathrm{Ed}}\right)$ taking account of the sign of $N_{1,\mathrm{Ed}}$ and $N_{2,\mathrm{Ed}}$ where $\lvert N_{2,\mathrm{Ed}}\rvert \leq \lvert N_{1,\mathrm{Ed}}\rvert$
KK joint	$60^\circ \leq \varphi \leq 90^\circ$
 	$\mu = 0{,}9$ provided that, in a gap-type joint, at section 1-1 the chord satisfies: $\left[\frac{N_{0,\mathrm{Ed}}}{N_{p\ell,0,\mathrm{Rd}}}\right]^2 + \left[\frac{V_{0,\mathrm{Ed}}}{V_{p\ell,0,\mathrm{Rd}}}\right]^2 \leq 1{,}0$

7.6 Welded joints between CHS or RHS brace members and I or H section chords

(1) Provided that the geometry of the joints is within the range of validity given in table 7.20, the design resistances of the joints should be determined using the expressions given in table 7.21 or table 7.22 as appropriate.

Table 7.20: Range of validity for welded joints between CHS or RHS brace members and I or H section chord members

<table>
<tr><th rowspan="3">Type of joint</th><th colspan="6">Joint parameter [i = 1 or 2, j = overlapped brace]</th></tr>
<tr><th rowspan="2">d_w/t_w</th><th colspan="2">b_i/t_i and h_i/t_i or d_i/t_i</th><th rowspan="2">h_i/b_i</th><th rowspan="2">b_0/t_f</th><th rowspan="2">b_i/b_j</th></tr>
<tr><th>Compression</th><th>Tension</th></tr>
<tr><td>X</td><td>Class 1
and
$d_w \leq 400$ mm</td><td rowspan="4">Class 1
and
$\frac{h_i}{t_i} \leq 35$
$\frac{b_i}{t_i} \leq 35$
$\frac{d_i}{t_i} \leq 50$</td><td rowspan="4">$\frac{h_i}{t_i} \leq 35$
$\frac{b_i}{t_i} \leq 35$
$\frac{d_i}{t_i} \leq 50$</td><td>$\geq 0,5$
but
$\leq 2,0$</td><td rowspan="4">Class 2</td><td>–</td></tr>
<tr><td>T or Y</td><td rowspan="3">Class 2
and
$d_w \leq 400$ mm</td><td rowspan="2">1,0</td><td rowspan="2">–</td></tr>
<tr><td>K gap
N gap</td></tr>
<tr><td>K overlap
N overlap</td><td>$\geq 0,5$
but
$\leq 2,0$</td><td>$\geq 0,75$</td></tr>
</table>

(2) For joints within the range of validity given in table 7.20, only the design criteria covered in the appropriate table need be considered. The design resistance of a connection should be taken as the minimum value for all applicable criteria.

(3) For joints outside the range of validity given in table 7.20, all the criteria given in 7.2.2 should be considered. In addition, the secondary moments in the joints caused by their rotational stiffness should be taken into account.

(4) In brace member connections subjected only to axial forces, the design axial force $N_{i,Ed}$ should not exceed the design axial resistance of the welded joint $N_{i,Rd}$, determined from table 7.21.

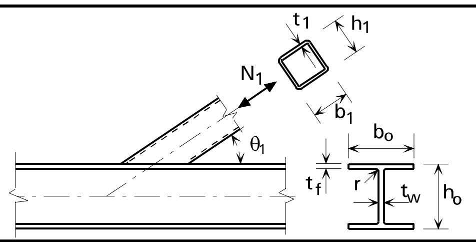
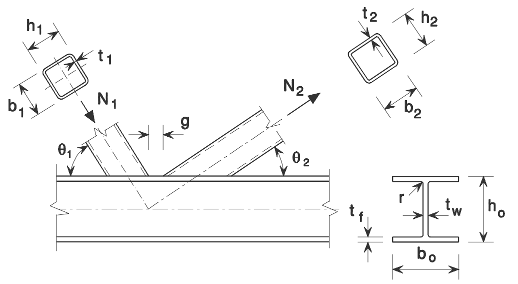
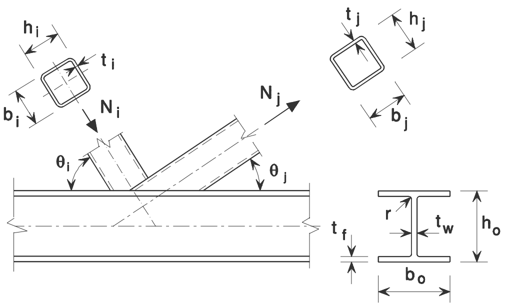
(5) Brace member connections subject to combined bending and axial force should satisfy:

$$\frac{N_{i,Ed}}{N_{i,Rd}} + \frac{M_{ip,i,Ed}}{M_{ip,i,Rd}} \leq 1,0 \qquad \text{... (7.5)}$$

where: $M_{ip,i,Rd}$ is the design in-plane moment resistance;

$M_{ip,i,Ed}$ is the design in-plane internal moment.

Table 7.21: Design resistances of welded joints between RHS or CHS brace members and I or H section chords

Type of joint	Design resistance [i = 1 or 2, j = overlapped brace]	
T, Y and X joints	Chord web yielding	
	$N_{1,Rd} = \dfrac{f_{y0}\, t_w\, b_w}{\sin\theta_1} \Big/ \gamma_{M5}$	
	Brace failure	
	$N_{1,Rd} = 2 f_{y1}\, t_1\, p_{eff} \Big/ \gamma_{M5}$	
K and N gap joints [i = 1 or 2]	Chord web stability	Brace failure need not be checked if: $g/t_f \leq 20 - 28\beta$ $\beta \leq 1{,}0 - 0{,}03\gamma$ where $\gamma = b_0/2t_f$ and for CHS: $0{,}75 \leq d_1/d_2 \leq 1{,}33$ or for RHS: $0{,}75 \leq b_1/b_2 \leq 1{,}33$
	$N_{i,Rd} = \dfrac{f_{y0}\, t_w\, b_w}{\sin\theta_1} \Big/ \gamma_{M5}$	
	Brace failure	
	$N_{i,Rd} = 2 f_{yi}\, t_i\, p_{eff} \Big/ \gamma_{M5}$	
	Chord shear	
	$N_{i,Rd} = \dfrac{f_{y0}\, A_v}{\sqrt{3}\,\sin\theta_i} \Big/ \gamma_{M5}$ $N_{0,Rd} = \left[(A_0 - A_v)\, f_{y0} + A_v\, f_{y0}\sqrt{1-(V_{Ed}/V_{pl,Rd})^2}\right] \Big/ \gamma_{M5}$	
K and N overlap joints *) [i = 1 or 2]	Brace failure $25\% \leq \lambda_{ov} < 50\%$	
Members i and j may be in either tension or compression.	$N_{i,Rd} = f_{yi}\, t_i \left(p_{eff} + b_{e,ov} + (h_i - 2t_i)\, \lambda_{ov}/50\right) \Big/ \gamma_{M5}$	
	Brace failure $50\% \leq \lambda_{ov} < 80\%$	
	$N_{i,Rd} = f_{yi}\, t_i \left(p_{eff} + b_{e,ov} + h_i - 2t_i\right) \Big/ \gamma_{M5}$	
	Brace failure $\lambda_{ov} \geq 80\%$	
	$N_{i,Rd} = f_{yi}\, t_i \left(b_i + b_{e,ov} + 2h_i - 4t_i\right) \Big/ \gamma_{M5}$	
$A_v = A_0 - (2 - \alpha)\, b_0\, t_f + (t_w + 2r)\, t_f$ For RHS brace: $\alpha = \sqrt{\dfrac{1}{(1 + 4g^2/3t_f^2)}}$ For CHS brace: $\alpha = 0$	$p_{eff} = t_w + 2r + 7 t_f f_{y0}/f_{yi}$ but $p_{eff} \leq b_i + h_i - 2t_i$ for T, Y, X joints and K and N gap joints and $b_{eff} \leq b_i + h_i - 2t_i$ for K and N overlap joints.	$b_w = \dfrac{h_i}{\sin\theta_i} + 5(t_f + r)$ but
	$b_{e,ov} = \dfrac{10}{b_j/t_j}\, \dfrac{f_{yj}\, t_j}{f_{yi}\, t_i}\, b_i$ but $b_{e,ov} \leq b_i$	$b_w \leq 2t_i + 10\,(t_f + r)$

For CHS braces multiply the above resistances for brace failure by $\pi/4$ and replace both b_1 and h_1 by d_1 and both b_2 and h_2 by d_2.

*) Only the overlapping brace member i need be checked. The brace member efficiency (i.e. the design resistance of the joint divided by the design plastic resistance of the brace member) of the overlapped brace member j should be taken as equal to that of the overlapping brace member.

(6) The design internal moment $M_{i,Ed}$ may be taken as the value at the point where the centreline of the brace member meets the face of the chord member.

(7) The design in-plane moment resistance $M_{ip,1,Rd}$ should be obtained from table 7.22.

(8) If stiffeners in the chord (see Figure 7.7) are used, then the bracing failure resistance $N_{i,Rd}$ for T-, X-, Y-, K-gap and N-gap joints (Table 7.22) is determined as follows:

$$N_{i,Rd} = 2\, f_{yi}\, t_i\, (b_{eff} + b_{eff,s}) / \gamma_{M5} \qquad \text{... (7.6)}$$

where: $b_{eff} = t_w + 2r + 7\, t_f\, f_{y0} / f_{yi}$ but $\leq b_i + h_i - 2t_i$

$b_{eff,s} = t_s + 2a + 7\, t_f\, f_{y0} / f_{yi}$ but $\leq b_i + h_i - 2t_i$

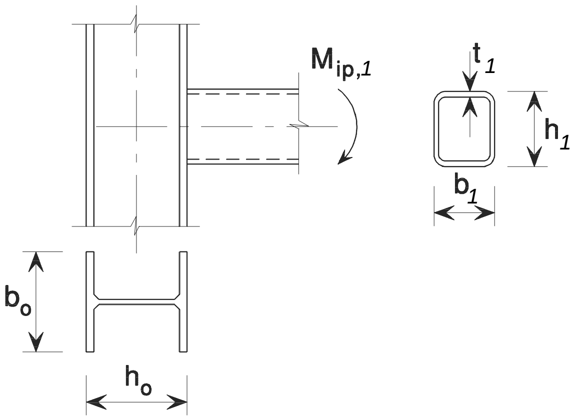
$b_{eff} + b_{eff,s} \leq b_i + h_i - 2t_i$

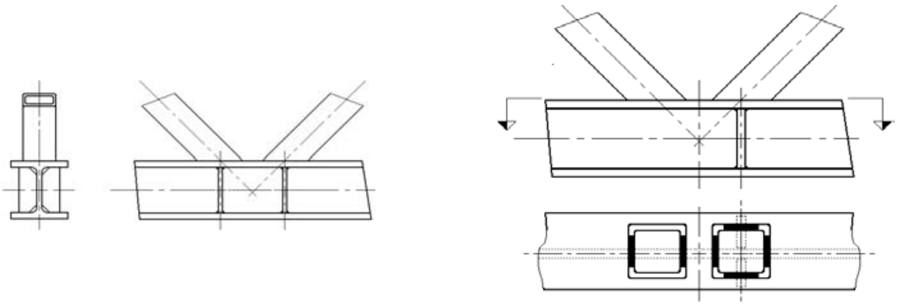
where: a is stiffener weld throat thickness, '2a' becomes 'a' if single sided fillet welds are used;

s refers to the stiffener.

(9) The stiffeners should be at least as thick as the I-section web.

Table 7.22: Design moment resistances of welded joints between rectangular hollow section brace members and I or H section chords

Type of joint	Design resistance [i = 1 or 2, j = overlapped brace]
T and Y joints	Chord web yielding
	$M_{ip,1,Rd} = 0{,}5 f_{y0}\, t_w\, b_w\, h_1 \,/\, \gamma_{M5}$
	Brace failure
	$M_{ip,1,Rd} = f_{y1}\, t_1\, b_{eff} (h_1 - t_1) \,/\, \gamma_{M5}$
Parameters b_{eff} and b_w	
$b_{eff} = t_w + 2r + 7\, t_f\, f_{y0}/f_{y1}$ but $b_{eff} \leq b_1$	$b_w = \dfrac{h_1}{\sin\,\theta_1} + 5(t_f + r)$ but $b_w \leq 2t_1 + 10(t_f + r)$



Bracing effective perimeter, without (left) and with (right) stiffeners

Figure 7.7: Stiffeners for I-section chords

7.7 Welded joints between CHS or RHS brace members and channel section chord members

(1) Provided that the geometry of the joints is within the range of validity given in table 7.23, the design resistances of welded joints between hollow section brace members and channel section chord members may be determined using table 7.24.

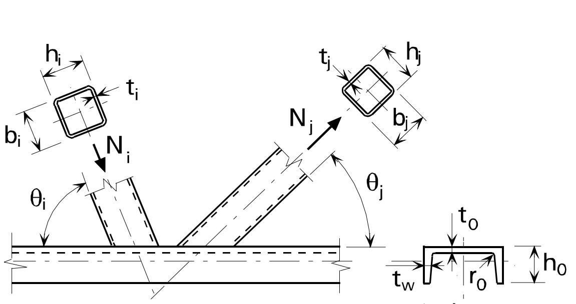
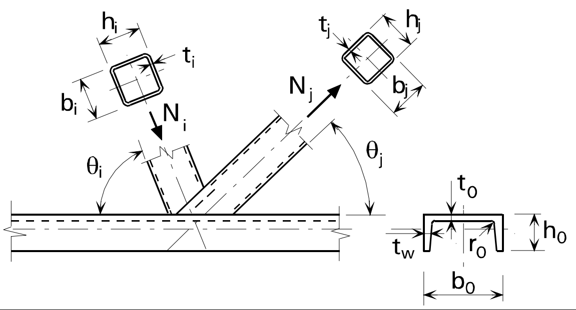
(2) The secondary moments in the joints caused by their bending stiffess should be taken into account.

(3) In a gap type joint, the design axial resistance of the chord cross-section $N_{0,\mathrm{Rd}}$ should be determined allowing for the shear force transferred between the brace members by the chord, neglecting the associated secondary moment. Verification should be made according to EN 1993-1-1.

Table 7.23: Range of validity for welded joints between CHS or RHS brace members and channel section chord

<table>
<tr><td rowspan="3">Type of joint</td><td colspan="6">Joint parameter [i = 1 or 2, j = overlapped brace]</td></tr>
<tr><td rowspan="2">b_i/b_0</td><td colspan="2">b_i/t_i and h_i/t_i or d_i/t_i</td><td rowspan="2">h_i/b_i</td><td rowspan="2">b_0/t_0</td><td rowspan="2">Gap or overlap
b_i/b_j</td></tr>
<tr><td>Compression</td><td>Tension</td></tr>
<tr><td>K gap
N gap</td><td>$\geq 0{,}4$
and
$b_0 \leq 400$ mm</td><td rowspan="2">Class 1
and
$\frac{h_i}{t_i} \leq 35$
$\frac{b_i}{t_i} \leq 35$
$\frac{d_i}{t_i} \leq 50$</td><td rowspan="2">$\frac{h_i}{t_i} \leq 35$
$\frac{b_i}{t_i} \leq 35$
$\frac{d_i}{t_i} \leq 50$</td><td rowspan="2">$\geq 0{,}5$
but
$\leq 2{,}0$</td><td rowspan="2">Class 2</td><td>$0{,}5(1-\beta^*) \leq g/b_0^* \leq 1{,}5(1-\beta^*)$ 1)
and
$g \geq t_1 + t_2$</td></tr>
<tr><td>K overlap
N overlap</td><td>$\geq 0{,}25$
and
$b_0 \leq 400$ mm</td><td>$25\% \leq \lambda_{ov} < 100\%$
$b_i/b_j \geq 0{,}75$</td></tr>
<tr><td colspan="7">$\beta^* = b_1/b_0^*$
$b_0^* = b_0 - 2\,(t_w + r_0)$
1) This condition only apply when $\beta \leq 0{,}85$.</td></tr>
</table>

Table 7.24: Design resistance of welded joints between RHS or CHS brace members and channel section chords

<table>
<tr><th>Type of joint</th><th>Design resistance [i = 1 or 2, j = overlapped brace]</th></tr>
<tr><td>K and N gap joints</td><td>Brace failure</td></tr>
<tr><td rowspan="3">
</td><td>$N_{i,Rd} = f_{yi}\, t_i \left(b_i + b_{eff} + 2h_i - 4t_i\right) / \gamma_{M5}$</td></tr>
<tr><td>Chord failure</td></tr>
<tr><td>$N_{i,Rd} = \dfrac{f_{y0} A_v}{\sqrt{3}\sin\theta_i} / \gamma_{M5}$
$N_{0,Rd} = \left[(A_0 - A_v)\, f_{y0} + A_v\, f_{y0}\sqrt{1-(V_{Ed}/V_{pl,Rd})^2}\right] / \gamma_{M5}$</td></tr>
<tr><td>K and N overlap joints *)</td><td>Brace failure 25% ≤ λ_{ov} < 50%</td></tr>
<tr><td rowspan="5">
</td><td>$N_{i,Rd} = f_{yi}\, t_i \left(b_{eff} + b_{e,ov} + (2h_i - 4t_i)\, \lambda_{ov}/50\right) / \gamma_{M5}$</td></tr>
<tr><td>Brace failure 50% ≤ λ_{ov} < 80%</td></tr>
<tr><td>$N_{i,Rd} = f_{yi}\, t_i \left(b_{eff} + b_{e,ov} + 2h_i - 4t_i\right) / \gamma_{M5}$</td></tr>
<tr><td>Brace failure λ_{ov} ≥ 80%</td></tr>
<tr><td>$N_{i,Rd} = f_{yi}\, t_i \left(b_i + b_{e,ov} + 2h_i - 4t_i\right) / \gamma_{M5}$</td></tr>
<tr><td>$A_v = A_0 - (1 - \alpha)\, b_0^*\, t_0$
$b_0^* = b_0 - 2\,(t_w + r_0)$

For RHS: $\alpha = \sqrt{\dfrac{1}{(1 + 4g^2/3t_f^2)}}$

For CHS: $\alpha = 0$

$V_{pl,Rd} = \dfrac{f_{y0} A_v}{\sqrt{3}} / \gamma_{M5}$
$V_{Ed} = (N_{i,Ed} \sin\theta_i)_{max}$</td><td>$b_{eff} = \dfrac{10}{b_0^*/t_0}\, \dfrac{f_{y0}\, t_0}{f_{yi}\, t_i}\, b_i$ but $b_{eff} \le b_i$

$b_{e,ov} = \dfrac{10}{b_j/t_j}\, \dfrac{f_{yj}\, t_j}{f_{yi}\, t_i}\, b_i$ but $b_{e,ov} \le b_i$</td></tr>
<tr><td colspan="2">For CHS braces exept the chord failure, multiply the above resistances by π/4 and replace both b_1 and h_1 by d_1 as well as b_2 and h_2 by d_2.</td></tr>
<tr><td colspan="2">*) Only the overlapping brace member i needs to be checked. The brace member efficiency (i.e. the design resistance of the joint divided by the design plastic resistance of the brace member) of the overlapped brace member j should be taken as equal to that of the overlapping brace member.</td></tr>
</table>