TITLE 5. JOINTS AND STRUCTURAL MEMBERS

CHAPTER XIV. JOINTS

Section 55. General

55.1. Bases

Joint connections in a structure should be designed to fulfil the intended level of safety, serviceability and durability, and the ability to withstand at least the stresses provided for them in the global analysis of the structure.

55.2. Manufacture and assembly

Joints should be designed for ease of use and safety. Special attention shall be paid to providing the space required to:

Assemble components safely.

Tighten the bolts.

Need for access for welders.

Accessibility for people in charge of performing protection and maintenance treatments and inspection work, and their teams.

The effect of the length and angle tolerances between the faces of a single part on the fit with contiguous parts should also be taken into account.

55.3. Transmission of forces

The arrangement of each joint shall be studied to ensure that, with a minimum of members, the existing forces are transmitted under the best possible conditions and so as to minimise secondary forces.

55.4. Nodes in triangular structures

In the case of triangular structures, compliance with the above condition shall be facilitated if the axes of the bars to be joined in a node come together in a point and when the angle formed by contiguous bars is between 30º and 150º.

If both conditions are met, it may be assumed that the bars are joined in the node. This assumption may not be made if the second condition is not met, or if any loads are applied to intermediate points of the bar other than its own weight or the direct action of the wind, or if the ratio of span to height of the structure is less than 6.

If the first condition is not met, the corresponding eccentricity shall be taken into account in the calculation, notwithstanding the provisions of Section 64 for certain joints between tubular sections.

55.5. Splices

Splices are extension joints of bars or profiles of the same or very similar section. Splices shall only be permitted if included in the design or workshop drawings duly approved by Project Management.

55.6. On-site joints

The number of joints made on site shall be reduced to a minimum. For this purpose, it is advisable for the Designer and Constructor to work together to resolve transport and assembly issues that such a reduction may cause.

It is advisable to follow best building practice of designing joints to be made on site bolted together.

55.7. Hybrid connections

Hybrid joints are joints in which two or more different joining means (welding or bolting) are used together to transmit a given force between two separate parts.

This does not include the transmission of a given force from one part to another using a given joining means, and from this second part to a third part using a different means.

Section 56. Determination of forces in joints and distribution between components

56.1. Forces in joints

The forces applied to joints are determined by means of a global analysis of the structure, carried out in accordance with the provisions of Chapter II (Project Bases) and Chapter V (Structural Analysis) of this Code.

This global analysis shall take specific account of secondary effects and structural imperfections, where relevant, and the flexibility of joints in all cases.

Joints shall be dimensioned to withstand at least the forces applied to them, calculated as indicated above. Under no circumstances shall the forces N_{Ed}, M_{Ed} and V_{Ed} be less than:

Half of the plastic axial force of the part section, $N_{Ed} = 1/2\ N_p = 0.5Af_y$ in parts subjected predominantly to axial forces, such as columns, ties and lattice parts.

Half of the elastic moment of the part section, $M_{Ed} = 1/2\ M_{el} = 0.5\ W_e f_y$ and one third of its plastic shear force, $V_{Ed} = 1/3\ V_p \approx 0.2A_w f_y$ in internal points of flexed parts. If the joint is located less than two heights away from the anticipated location of a plastic hinge, half of the elastic moment M_{el} shall be replaced with the full plastic moment, $M_{Ed} = M_{pl} = 2S_y f_y$, subject to detailed study.

One third of the plastic shear force of the part section $V_{Ed} = 1/3V_p \approx 0.2A_w f_y$ in articulated ends of flexed parts.

Unless said forces have been determined precisely and cannot be increased by introducing new members in the construction or by the presence of members not taken into consideration, it is advisable to dimension the joints for the maximum forces that the parts can transmit, depending on the working method to be used on them.

56.2. Stress distribution

The distribution of forces between the different elements of the joint, based on an elastic linear analysis of the joint, shall be permitted in all cases.

Alternatively, where expressly authorised in this Code and on the basis of the principle of minimum plasticity, distributions based on non-linear analyses shall be permitted. Any distribution of forces between the different members of the joint that meets the following conditions shall be accepted as correct:

The sum of the forces and moments assumed for each of the different elements of the joint are in equilibrium with the external forces applied to it.

Each element of the joint is able to withstand the forces allocated to it in the distribution.

Each element of the joint has sufficient deformation capacity to make the proposed distribution physically possible.

The distribution of forces should be realistic with regard to relative stiffnesses with the joints elements involved, preferably transmitted through the joint via the most rigid areas.

If the fracture line method is used to justify a particular distribution, it must be checked by testing. The exceptions expressly given in this Code shall apply.

Stress distribution may not be made using plastic methods in the cases indicated in section 58.10.

Section 57. Classification of joints subjected to bending moment

57.1. General

This section looks at joints between two parts (such as beam-column joints or beam splices) that are essentially intended to transmit bending moments.

57.2. Moment-rotation characteristic

A joint intended to transmit moments is defined once the relation between the moment applied to it and the relative rotation it permits between the parts to be joined is known. The graphical representation of this relation is known as a moment-rotation characteristic.

In general, this diagram has a linear branch, Figure 57.2.a (1), a plastification start zone, Figure 57.2.a (2), and a major deformation branch, Figure 57.2.a (3).

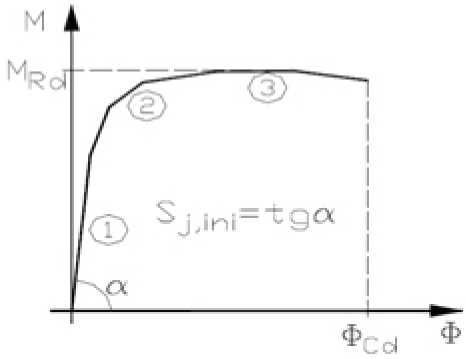
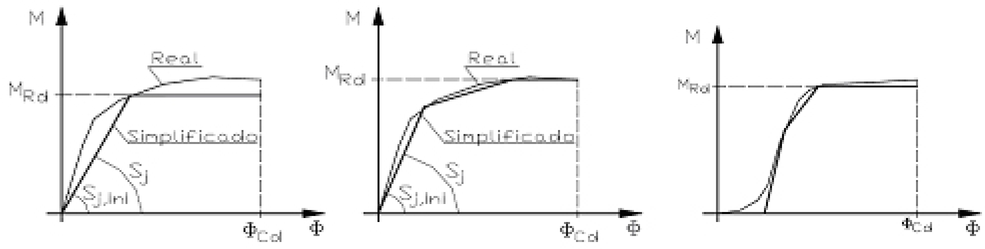


Figure 57.2.a Design Moment-rotation characteristic

These characteristics can be obtained by testing or using numerical methods that consider the non-linearity of the different materials, structural steel, weld metal, bolts, etc., used in the joint. For calculation purposes, they may be replaced by simplified, two- or three-line diagrams obtained from the real diagram, with the sole condition that all of its points be below it, Figure 57.2.b.



Real	Actual
Simplificado	Simplified

Figure 57.2.b Simplified moment-rotation characteristic

A simplified diagram is defined by three parameters, Figure 57.2.b:

Moment resistance M_{Rd}, determined by the highest y-axis value of the diagram.

Rotation capacity, ΦC_d, determined by the highest x-axis value of the diagram.

Rotational stiffness, S_j, depending on the linear branch passing the origin.

57.3. Classification by strength

Joints are classified as follows, depending on their strength relative to the strength of the parts to be joined:

Nominally pinned joints are joints that should be incapable of transmitting significant moments (greater than 25 % of the plastic moment of the parts to be joined), which might adversely affect the performance of individual parts of the structure. They should be capable of accepting the rotations provided for in the global analysis.

Full strength joints, whose ultimate moment is equal to or greater than that of the parts to be joined, $M_{Rd} \geq M_{pl.Rd}$.

Partial-strength joints are joints that do not meet the criteria for nominally pinned or full-strength. Their ultimate moment may not be less than that determined in the analysis, $M_{Rd} \geq M_{Ed}$.

In any case, the rotation capacity of the joint shall be such as not to limit the formation of the plastic hinges provided for in the analyses.

The rotation capacity of a joint must be demonstrated experimentally or by numerical means that take account of the non-linearity of the performance of the materials and elements involved, except where this Code provides simplified methods for calculating it, as in Section 62.

Specifically, if the ultimate moment of a full-strength connection is at least 20 % greater than the plastic moment of the largest part being joined, $M_{Rd} \geq 1.2M_{pl.Rd}$ the rotation capacity need not be checked, this being deemed sufficient.

57.4. Classification by stiffness

Joints are classified as follows, depending on their rotational stiffness Nominally pinned joints which are joints whose stiffness meets the following condition:

$$S_{j,ini} \leq \frac{E_b}{2L_b}$$

where I_b and L_b are the second moment of area and the length of the connected beam.

Rigid joints or abutments are joints whose deformation has no significant influence on the laws of global forces of the structure or its general deformability. Joints whose initial stiffness $S_{j,ini}$ in their moment-rotation characteristic complies with the condition below shall be placed in this category:

$$S_{j,ini} \leq k \frac{EI_b}{L_b}$$

where k shall be 8 if the beam is part of a non-translational portal frame, or 25 if it is part of a translational portal frame.

Semi-rigid joints are joints that do not meet the criteria for nominally pinned connections or rigid joints. Joints that are not nominally pinned connections and that are part of portal frame floors for which the following is true shall also be classified as semi-rigid:

$$\frac{K_b}{K_c} < 0{,}1$$

where K_b is the average value of I_b/L_b for all of the beams at the top of the storey and K_c is the average value of I_c/L_c of the columns or pillars of said storey.

57.5. Modelling joints in the global analysis

In general, for the global analysis, joints shall be modelled using their moment-rotation characteristic.

Joints may be modelled using nominally pinned connections that do not transmit bending moment. Full-strength joints may be modelled as a continuous (or rigid) node. In these cases, the moment-rotation characteristic for the joint need not be included.

In the case of semi-rigid or partial-strength joints, the stiffness of the joint Sj, which is the moment $M_{j,Ed}$ (defined by its moment-rotation diagram), must be used in the global analysis to determine the forces in the structure.

As a simplification, in the case of global elastic and plastic analysis, a two-line diagram may be used (such as the one shown in Figure 57.2.b) in which the stiffness S_j is:

$$S_{j,ini} / \eta.$$

As a simplification in the case of global elastic analysis, the rotation stiffness for all moment values M_{Ed} may be as follows:

$$S_{j,ini} \text{ if } M_{Ed} \leq 2/3\ M_{Rd}$$

$$S_{j,ini} / \eta \text{ if } M_{Ed} \leq M_{Rd}$$

The parameter η shall be 2 for beam-column joints, 3.5 for beam-beam joints, splices and bases with angle irons bolted to the flanges, and 3 for any other type of joint.

Section 58. Bolted joints

58.1. Bolt types

The bolts used in structural steel joints shall preferably be of one of the following classes: 4.6, 5.6, 6.8, 8.8 or 10.9, covered by one of the standards provided for in section 29.2 of this Code.

Bolts whose class is below 4.6 or above 10.9 shall not be used without experimental demonstration that they are suitable for the joint for which they are intended.

The nominal yield strength f_{yb} and ultimate tensile strength f_{ub} values of the steel used in the bolts of permissible classes are shown in Table 58.1.

Table 58.1. Nominal values of yield strength f_{yb} and ultimate tensile strength f_{ub} for bolts

BOLT CLASS	4.6	5.6	6.8	8.8	10.9
f_{yb}, N/mm2	240	300	480	640	900
f_{ub}, N/mm2	400	500	600	800	1 000

Grade 8.8 and 10.9 bolts are classified as high-strength.

58.2. Categories of bolted joints

Bolted joints are classified into five categories depending on how the bolts are worked. Three categories apply to joints in which the bolts are stressed perpendicular to their axis (categories A, B and C) and the other two (categories D and E) apply to joints in which the bolts are stressed in the direction of their axis (i.e. tension).

Category A: Joints in which the ordinary or high-strength bolts work in shear and compression. If the bolts are high-strength (classes 8.8 or 10.9) they need not be pre-loaded and the contact surfaces need not undergo any special preparation. This should be calculated in accordance with the provisions of section 58.6. For obvious reasons of economy, joints of this category shall be used if the bolts will be loaded perpendicular to their axis. If the part is subjected to fatigue, impacts or alternating forces, the use of high-strength bolts pre-loaded to N_0 (as indicated in section 58.8) is recommended, although the bolts may continue to be calculated in shear and compression.

Category B: Joints made using pre-loaded, high-strength bolts with prepared contact surfaces that are not intended to slip in serviceability limit state. In the ultimate limit state, the joint may slip and the bolts may work in shear and compression. The force $F_{s.Ed}$ to be transmitted, calculated in serviceability limit state, must comply with the following:

$$F_{s.Ed} \leq F_{s.Rd}$$

where $F_{s.Rd}$ is the value given in section 58.8.In ultimate limit state, the joint shall be checked in accordance with the provisions of section 58.6.

Category C: Joints made using pre-loaded high-strength bolts with prepared contact surfaces that are not intended to slip in the ultimate limit state. The force $F_{s.Ed}$ to be transmitted, calculated in ultimate limit state, should comply with the following:

$$F_{s.Ed} \leq F_{s.Rd}$$

where $F_{s.Rd}$ is the value given in section 58.8.

It shall also be checked that, in ultimate limit state:

- joint resists shearing and compression in accordance with the provisions of section 58.6.This condition may be deemed met if the thickness t_{min} of the thinnest part to be joined is greater than d/2.4 if the parts are made of S 235 or S 275 steel, or *d/3.1* if the parts are made of S 355 steel;

- The force to be transmitted, $F_{s.Ed}$, is less than the plastic resistance of the net area of the part:

$$F_{s.Ed} \leq A_{net} f_y / \gamma_{M0}$$

This category of joints is used if, to simplify assembly, drill holes that are oversize or slotted in the direction of the force to be transmitted are used, or if any slipping of the joint would significantly reduce the strength or stiffness of the structure, or if the bolts are used jointly with weld seams in hybrid joints.

Category D: Joints made using ordinary or high-strength bolts working in tension. If high-strength bolts are used, they need not be pre-loaded and the contact surfaces need not be prepared. The use of joints in this category is not recommended if they are to be subjected to frequent variations in the tensile force to be transmitted, although they may be used if said tensile forces are attributable exclusively to wind loads.

Category E: Joints made using pre-loaded, high-strength bolts working intension. Pre-loading improves the stiffness of the joint in serviceability limit state and fatigue resistance, although this limit depends to a great extent on constructional details and adjustment tolerances between parts. Surfaces need only be prepared if the joint is subjected to forces perpendicular to the axis of the bolts as well as tensile forces (joints in categories E+B or E+C).

58.3. Bolt holes

Bolt holes should preferably be drilled. They may be punched if the diameter of the hole is greater than the thickness of the part, provided that said thickness is not greater than 15 mm and the parts to be joined are not subject to fatigue stressing.

The standard diameter of holes shall be the shank of the bolt plus:

1 mm for bolts with a diameter of 12 and 14 mm,

1 or 2 mm for bolts with a diameter of 16 to 24 mm,

2 or 3 mm for bolts with a diameter of 27 mm or more.

Holes for 12 and 14 mm bolts may have 2 mm play provided that the compression strength of the group of bolts is less than the shear strength.

In high-strength slip-resistant bolted connections, oversized or short or long slotted holes may be used to facilitate assembly of the parts.

For oversize holes, the drill diameter shall be the shank of the bolt plus:

- 3 mm for 12 mm bolts,
- 4 mm for 14 to 22 mm bolts,
- 6 mm for 24 mm bolts,
- 8 mm for bolts with a diameter of 27 mm or more.

For anchoring bolts in base plates, oversized holes with the clearance indicated by the Designer may be used, provided that said drill holes are covered with a splice plate of suitable size and thickness. The holes in the cover plates shall be of standard diameter. If the anchoring bolts have to withstand forces perpendicular to their axis, the splice plates must be welded to the base plate with a weld strong enough to transmit such forces.

The width of short or long slotted holes perpendicular to the force shall be the same as the diameter of the corresponding standard holes. In the direction of the force, the distance *e* (Figure 58.3) for short slotted holes shall be:

$(d + 4)$ mm for 12 to 14 mm bolts,

$(d + 6)$ mm for 16 to 22 mm bolts,

$(d + 8)$ mm for 24 mm bolts,

$(d + 10)$ mm for bolts of 27 mm or greater.

For long slotted holes, it shall in all cases be as follows:

$e = 2.5\ d$ mm, where d is the diameter of the shank of the corresponding bolt.

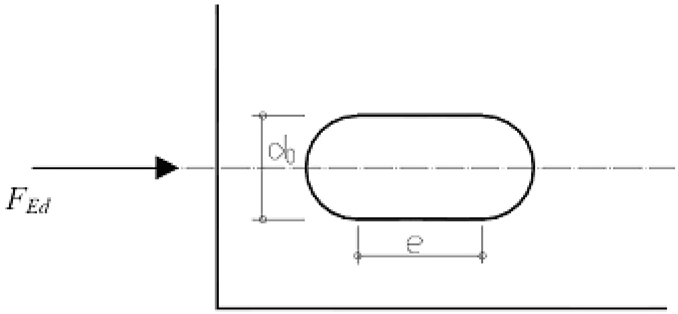


Figure 58.3. End and edge distances for Slotted holes

The use of long slotted holes is permitted if relative movement between the parts to be joined is permitted. In this case, the designer shall determine the length of the drill hole. In any case, to avoid durability issues, long slotted holes on the outer faces of parts must be covered with splice plates of suitable size and thickness having holes no larger than standard.

58.4. Position of holes

Holes for bolts should be arranged so as to hinder corrosion of the parts to be joined, to prevent local buckling, to enable the easy placement of bolts and nuts, and not to unnecessarily reduce the compression strength of the parts to be joined.

Table 58.4.a sets out the maximum and minimum limits for the hole and edge distances defined below and in Figures 58.4.a and 58.4.b, where:

e_1 Distance from the centre of a hole to a contiguous edge, measured in the direction of the force to be transmitted.
e_2 Distance from the centre of a hole to a contiguous edge, measured perpendicular to the force to be transmitted.
p_1 Distance between the centres of contiguous holes, measured in the direction of the force to be transmitted.
p_2 Distances between contiguous rows of bolts or rivets, measured perpendicular to the force to be transmitted.
m Distance from the axis of the drill hole to any surface parallel to said axis.

In the case of slotted holes, the distances e_1, e_2, p_1 and p_2 shall be measured from the centres of the end semicircles.

Table 58.4.a: Minimun and maximum spacing end and edge distances

Distances and spacing	Mandatory minimum	Recommended minimum	Maximum, normal environment	Maximum, weather or corrosive environment
e_1	$1.2d_0$	$2d_0$	125 mm or $8t$	40 mm + $4t$
e_2	$1.2d_0$	$1.5d_0$	125 mm or $8t$	40 mm + $4t$
p_1	$2.2d_0$	$3d_0$	Compression members $14t$ or 200 mm Tensed parts: $28t$ or 400 mm	
p_2	$2.4d_0$	$3d_0$	$14t$ or 200 mm	
m		$2d$		
d_0 Hole diameter. d Bolt diameter. t Thickness of the thinnest part to be joined.				

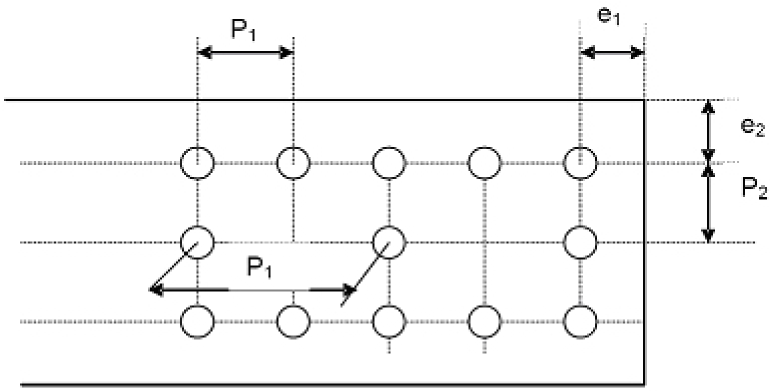


Figure 58.4.a Symbols for spacing of fasteners

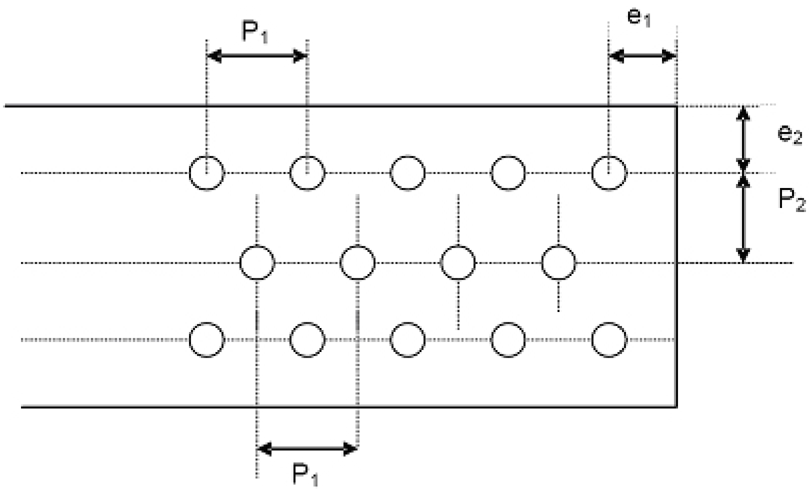


Figure 58.4.b Staggered spacing To ensure that bolts can be tightened easily, the distance *m* from the axis of the drill hole to any surface parallel to said axis (Figure 58.4.c) should be no less than *2d*, where *d* is the diameter of the bolt.

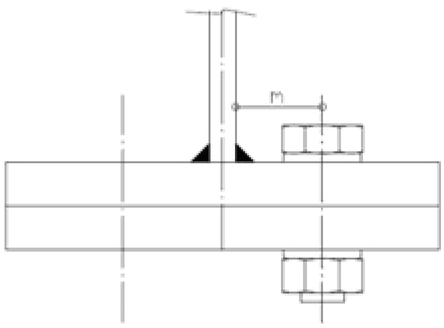


Figure 58.4.c Distance *m*

58.5. Strength of members with holes

The reduction in tensile, compression and bending strength of the parts to be joined by holes intended to hold joining apparatus should conform to the provisions of Section 34 of this Code.

58.5.1. Tear strength

At the ends of beams joined to other beams or to supports using joins requiring the removal of one or both flanges (Figure 58.5.1.a) or at the ends of tensed parts joined by bolts or gusset-plate welds (Figure 58.5.1.b), the tear strength of the parts and gusset plates must be checked.

Figure 58.5.1.a Tearing at part extremities

VIGA	BEAM
DESMEMBRADO	REMOVED SECTION
AREA A CORTANTE	SHEAR AREA
FALLO POR DESGARRO DE LA PARTE SOMBREADA	TEAR FAIL OF THE SHADED PART
AREA A TRACCION	TRACTION AREA

In the case of a join between a beam and another beam or a support using a double angle bearing (section 61.3), if the joint is made—as is normal—using a single column of *n* bolts, it shall only be necessary to make this check if the web is removed from the beam towards which the reaction is directed (Figure 58.5.1.a). In this case, it shall also be necessary to perform a bend check on the beginning of the removed section *d-d.*

1 Small tension force
2 Large shear force
3 Small shear force
4 Largetension force

Figure 58.5.1.b Block tearing

The end of the part stressed by an axial force centred on its axis or the corresponding gusset plate shall be secure if the following is true:

$$N_{Ed} \leq N_{ef.Rd}$$

where N_{Ed} is the design force and $N_{ef.Rd}$ is the tear strength, calculated on the basis of the following expression:

$$N_{ef,Rd} = \frac{f_u A_{nt}}{\gamma_{M2}} + \frac{f_y A_{nv}}{\sqrt{3}\gamma_{M0}}$$

in which:

A_{nt} is the net area of the zone subjected to tension.

A_{nv} is the net area of the zone subjected to shearing.

In joints in which the force acts eccentrically, such as joints with beam-end double angle bearings, A_{nt} in the expression above shall be half of its real value.

58.5.2. Angles connected by one leg and other unsymmetrically connected members in tension

The eccentricity in joints and the effects of the spacing n and edge distance of the bolts should be taken into account in determining the design resistance of:

- Unsymmetrical members.

- Symmetrical members that are connected unsymmetrically, such as angles connected by on one leg.

A single angle in tension connected by a single row of bolts in one side (see Figure 58.5.2) may be treated as concentrically loaded over an effective net section for which the design ultimate resistance should be determined as follows:

With 1 bolt $N_{u,Rd} = \frac{2(e_2 - 0.5d_0)tf_u}{\gamma_{M2}}$

With 2 bolts $N_{u,Rd} = \frac{\beta_2 A_{net} f_u}{\gamma_{M2}}$

With 3 or more bolts $N_{u,Rd} = \frac{\beta_3 A_{net} f_u}{\gamma_{M2}}$

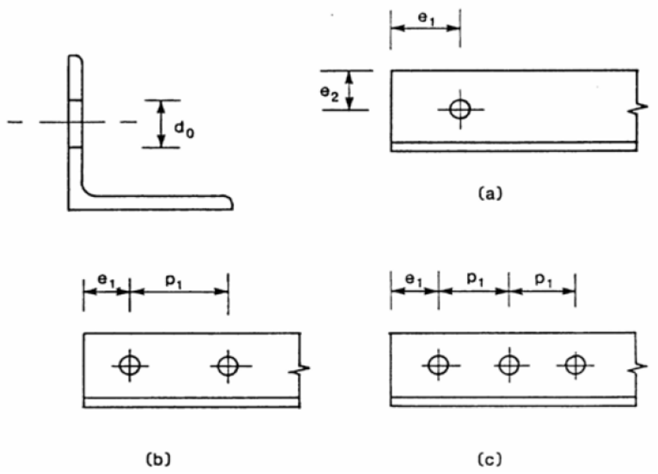
where:

β_2 and β_3 are reduction factors dependent on the pitch p_1, as given in Table 58.5. 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 angle unequal-leg connected by its smaller leg, A_{net} can be taken as equal to the net section area of an equivalent equal-leg of leg size equal to that of the smaller leg.

Table 58.5: Reduction factors β_2 and β_3

Pitch p_1	$\leq 2.5\ d_o$	$\geq 5.0\ d_o$
2 bolts β_2	0.4	0.7
3 bolts or more β_3	0.5	0.7



a) 1 bolt

b) 2 bolts

c) 3 bolts

Angles connected by one leg

58.5.3. Lug angle

If it is necessary to place lug angles between an angled-section part and a gusset plate (Figure 58.5.3), the joint of the lug angle to the part should be designed to resist a force 1,2 times the force corresponding to the flange of the angle iron that it joins, and its join to the gusset plate to resist a force 1,4 times the force of the flange of the angle iron to which it is joined.

If the part has a U section, joined by its web to a gusset plate using two lug angles, the join of each of these to the flanges of the U-part be to resist a force 1,1 times the force corresponding to the flange of the U-section, and its join to the gusset plate to resist a force 1,2 times the force in the flange that [...] of the U-section it joins.

In no cases should less than two bolts be used to attach the lug angle to a gusset or other supporting part.

The connection of lug angle to gusset plate or other supporting parts should terminate at the end of the member connected. The connection between the lug angle and the member should run from the end of the member to beyond the direct connection of the member to the gusset plate or other supporting part.

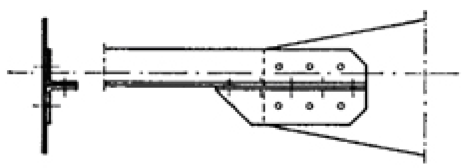


Figure 58.5.3 Lug angles

58.6. Shear and bearing resistance of bolts

If a bolt, placed in a standard hole, is perpendicular to its axis in the ultimate limit states, the force applied to it ($F_{v,Ed}$) shall not be greater than the lesser of the following values:

- The shear resistance of the bolt, $F_{v,Rd}$.

- The bearing resistance of the part in the zone contiguous to the bolt, $F_{b,Rd}$.

If all of the cutting planes pass through the unthreaded area of the bolt shank, the shear strength $F_{v,Rd}$ is determined by the expression:

$$F_{v,Rd} = \frac{0.6\, f_{ub} A n}{\gamma_{M2}}$$

If any of the cutting planes pass through the threaded area of the bolt shank, and the bolts are 4.6, 5.6 or 8.8, the shear strength is determined by the expression:

$$F_{v,Rd} = \frac{0{,}6 f_{ub} A_S n}{\gamma_{M2}}$$

If any of the cutting planes pass through the threaded area of the bolt, and the bolts are grade 6.8 or 10.9, the shear strength is determined by the expression:

$$F_{v,Rd} = \frac{0{,}5 f_{ub} A_S n}{\gamma_{M2}}$$

In the above expressions, A is the area of the bolt shank, A_s is the tension-resistant area (see Table 58.7); f_{ub} is the ultimate tensile stress of the bolt, and n is the number of cutting planes.

The bearing resistance of a part of thickness t against the shank of a bolt of diameter d placed in a standard hole is provided by the following expression:

$$F_{b,Rd} = \frac{\alpha \beta f_u d t}{\gamma_{M2}}$$

where f_u is the ultimate tensile strength of the material of the part, α is a coefficient equal to the smaller of the values:

$$\frac{e_1}{3d_0};\ \frac{p_1}{3d_0} - \frac{1}{4};\ \frac{f_{ub}}{f_u};\quad 1{,}0$$

and β is another coefficient equal to the smaller of the values:

$$\frac{2{,}8 e_2}{d_0} - 1.7;\quad 2{,}5$$

If the bolt is placed in an oversize hole, the value $F_{b,Rd}$ given by the above expressions is multiplied by a factor of 0.8. If the hole is slotted, with the slot direction perpendicular to the force to be transmitted, the value of $F_{b,Rd}$ given in the above expressions is multiplied by a factor of 0.6. Shear and compression forces may not be transmitted in slotted holes in the direction of the force.

For single-lap joins with a single bolt row, washers must be placed beneath the head and beneath the nut of the bolt, and the bearing resistance $F_{b,Rd}$, calculated according to the above expressions, shall be limited to the following maximum value:

$$F_{b,Rd} = 1.5\, f_u d\, t / \gamma_{M2}$$

For countersunk screws, the thickness *t* shall be the thickness of the part in which the head is seated, less half of its height.

Table 58.6.a sets out the single-shear strength in kN of bolts of the most common diameters and grades when the cutting planes do not pass through the threaded area of the shank.

Table 58.6.a.Single-shear strength in kN of most common bolts

CLASS	BOLT DIAMETER (mm)						
	12	14	16	20	22	24	27
4.6	21.71	29.55	38.60	60.32	72.98	86.86	109.93
5.6	27.14	36.95	48.25	75.40	91.23	108.57	137.41
6.8	32.57	44.33	57.90	90.48	109.48	130.28	164.89
8.8	43.43	59.11	77.21	120.64	145.97	173.72	219.86
10.9	54.28	73.89	96.50	150.80	182.46	217.14	274.82

Table 58.6.b sets out the bearing resistance of a part 10 mm thick against bolts of the most common diameters, using the distance-to-edge and distance-between-bolts values given therein.

Table 58.6.b.Compression strength in kN for sheets 10 mm thick

DIAMETER	MINIMUM DISTANCE (mm)					STEEL STRENGTH		
	d_0	e_1	e_2	p_1	p_2	S 235	S 275	S355
12	13	25	20	40	40	55.38	66.15	78.46
14	15	30	25	45	45	67.20	80.27	95.20
16	17	35	30	65	65	79.06	94.43	112.0
20	21	40	35	75	75	91.43	109.21	129.52
22	23	50	35	75	75	114.78	137.10	162.61
24	26	50	40	80	80	120.0	143.33	170.0
27	29	60	45	90	90	134.07	143.33	189.93

If it is necessary to place linings of thickness t_f greater than one third of the diameter of the bolts that are used to guarantee a good fit between the parts to be joined, the shear strength $F_{v,Rd}$ must be multiplied by a reduction coefficient β_f given by:

$$\beta_f = \frac{9d}{8d + 3t_f} \leq 1$$

For double-shear joints with linings on both faces of the joint, t_f shall be the thickness of the thicker of the two linings. Any additional bolt required on account of application of the factor β_f may, at the Designer's discretion, be placed in an extension of the linings provided for this purpose.

58.7. Tensile strength

When a bolt is loaded in the direction of its axis in the ultimate limit states by a tensile force $F_{t,Ed}$, which must include all potential leverage, it must not be greater than the lesser of the following two values:

- The tensile resistance of the bolt $F_{t,Rd}$.

- The punching shear resistance of the part beneath the nut or beneath the bolt head $B_{p,Rd}$.

In joints with an end plate and preloaded, high-strength bolts, the tensile strength of the plate-bolt unit $F_{ch,Rd}$ shall be determined in accordance with the provisions of section 61.2.

The tension resistance $F_{t,Rd}$ of a bolt is given by the expression:

$$F_{t,Rd} = 0.9\, f_{ub} A_s / \gamma_{M2}$$

The tensile strength indicated shall only apply to bolts manufactured in accordance with one of the standards listed in section 58.1. The tensile strength of members, such as anchoring bolts, in which the thread is manufactured using procedures that involve machining shall be the value given in the above expression multiplied by 0.85. Table 58.7 shows the tensile strength of normal-head screws of the most common diameters and classes.

Table 58.7. Ultimate tensile strength in kN

DIAMETER (mm)	As (mm^2)	CLASS			
		4.6	5.6	8.8	10.9
12	84.3	24.28	30.35	48.56	60.70
16	157	45.22	56.52	90.43	113.04
20	245	70.56	88.20	141.12	176.4
22	303	87.26	109.08	174.53	218.16
24	353	101.66	127.08	203.33	254.16
27	456	131.33	164.16	262.66	328.30

For countersunk bolts, the tensile resistance shall be 70 % of the value given by the above expressions.

The punching shear resistance of a sheet of thickness t ($B_{p,Rd}$) acted upon by a bolt subject to tension is given by the expression:

$$B_{p,Rd} = 0.6\, \pi d_m t f_u / \gamma_{M2}$$

in which d_m is the lesser average diameter between the circles circumscribed and inscribed on the nut or the head and f_u is the tensile strength of the steel in the sheet.

The value of $B_{p.Rd}$ need not be checked if the thickness of the sheet meets the following condition:

$$t_{min} \geq \frac{d f_{ub}}{6 f_u}$$

58.7.1. Combined shear and tension

Bolts subjected simultaneously to tensile forces and forces perpendicular to their axis, in addition to complying with section 58.7, must also meet the following condition:

$$\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1{,}4 F_{t,Rd}} \leq 1$$

58.8. Design slip resistance

High-strength bolts (classes 8.8 and 10.9), which should be preloaded be tightened using one of the methods set out in section 76.7 until the tension in the shank reaches 70 % of its ultimate tensile strength. Consequently, the design preloaded force of the bolt shall be:

$$N_0=0.7\ f_{ub}\ A_S$$

If bolts not covered by the standards included in section 58.1, or other members to be pre-stressed (such as threaded rods) are used, the design preloading force N_0 and the means of achieving it must be agreed between the Designer, member Manufacturer and Project Management.

The slip resistance of a preloaded, high-strength bolt should be taken as:

$$F_{S,Rd} = \frac{k_S n \mu}{\gamma_{M3}} N_0$$

Where s_{ki} Factor that depends on the type of holes used. This shall be 1.0 for normal holes, 0.85 for oversized or slotted drill holes that are short perpendicular to the force, and 0.7 for slotted holes that are long perpendicular to the force. If the slot of the hole runs in the direction of the force, k_s shall be 0.76 for short slotted drill holes and 0.63 for long slotted drill holes.

n Number of possible friction surfaces, generally $n = 1$ or $n = 2$.

μ Slip factor, which depends on the state of the contact surfaces in the possible friction interfaces. Its value is:

- 0.5 for sand- or shot-blasted surfaces up to grade SA 2 1/2 in standard UNE-EN ISO 8501-1, free of pitting, or surfaces with no post-treatment if the joint is made immediately after blasted such that there is no time for oxide to form on the contact surfaces, or surfaces post-treated by thermal spraying with aluminium or any other treatment that guarantees this factor, in the opinion of Project Management.
- 0.4 for sand- or shot-blasted surfaces up to grade SA 2 1/2 in standard UNE-EN ISO 8501-1, free of pitting and painted with zinc alkali silicate to a thickness of between 50 and 80 μm.
- 0.3 for surfaces cleaned with a wire brush or by flame cleaning.
- 0.2 for untreated or galvanised surfaces.

γ_{M3} The strength reduction coefficient shall be:

- 1.1 for B joints,
- 1.25 for C and hybrid joints and joints subject to fatigue effects.

If the joint is also subject to an axial tensile force that generates a tensile force in the bolt with the value $F_{t.Ed}$, $F_{s.Rd}$ shall be calculated as follows:

$$F_{S,Rd} = \frac{k_S n \mu}{\gamma_{M3}} (N_0 - 0{,}8 F_{t,Ed})$$

58.9. Connections made with pins

The provisions of this section apply to joints in which the pins need to allow the parts they join to rotate freely in relation to one another. If such rotation is not required, the joint may be designed as a single bolted connection, provided that the length of the pin is less than 3 times the diameter of the pin.

With a part of thickness t joined to others by means of a pin of thickness d seated in a drill hole of diameter d_0, it shall be able to transmit the force F_{Ed}, if its geometry (Figure 58.9.a) should follows:

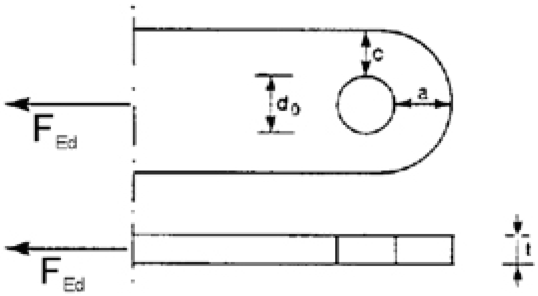


Figure 58.9.a

- Distance of the hole edge to an edge perpendicular to the force to be transmitted:

$$a \geq \frac{F_{Ed}\gamma_{M0}}{2tf_y} + \frac{2d_0}{3}$$

- Distance of the hole edge to an edge parallel to the force to be transmitted:

$$c \geq \frac{F_{Ed}\gamma_{M0}}{2tf_y} + \frac{d_0}{3}$$

where f_y is the elastic limit of the steel in the part and f_u is its tensile strength.

A part that meets the given geometry in Figure 58.9.b will be able to withstand the design force F_{Ed}, provided that the diameter of the drill hole and the thickness of the part comply with the following:

$$d_0 \leq 2{,}5t; \qquad t \geq 0{,}7\sqrt{\frac{F_{Ed}\gamma_{M0}}{f_y}}$$

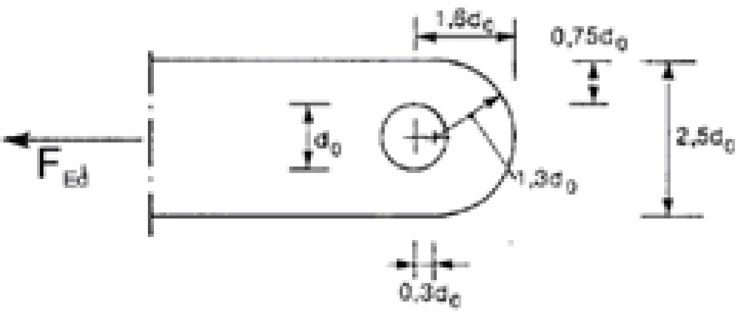


Figure 58.9.b

The reinforcing plates used to increase the net area of the parts or their compression strength must be arranged without any eccentricity and be large enough to transmit the correct force to the part. The joint between both must be dimensioned to withstand this force.

It is advisable to install a bushing made of corrosion-resistant material (such as sintered bronze with PTFE) between the pin and the parts to be joined if rotation of the joint needs to be guaranteed.

The shear resistance of of a pinof diameter d, of cross-sectional area A and made of steel with a tensile strength of f_{up} is determined by:

$$F_{v,Rd} = 0.6\, A\, f_{up} / \gamma_{M2}$$

The bending moment M_{Ed} in a pin joining a central part of thickness b subject to a force F_{Ed} perpendicular to the axis of the pin, with two other side parts of thickness a, separated by a distance c from the central part (Figure 58.9.c) shall be:

$$M_{Ed}=F_{Ed}(b + 4c + 2a)/8$$

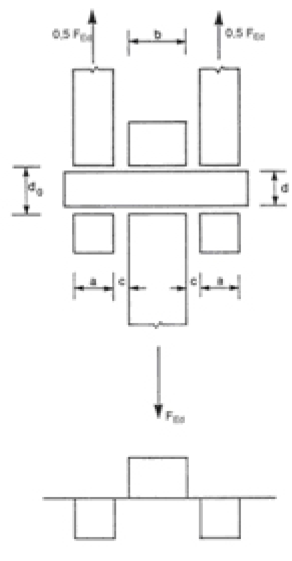


Figure 58.9.c

A pin of diameter d, modulus W_{el}, made of a steel with an elastic limit of f_{yp} shall comply with:

$$M_{Ed} \leq M_{Rd} = 1{,}5\, W_{el} f_{yp}/\gamma_{M0}$$

If the pin intended to be replaceable, the bending moment stressing it in the serviceability limit state $M_{Ed,ser}$ should satisfy the following condition:

$$M_{Ed,ser} \leq M_{Rd,ser} = 0.8\ W_{el}\ f_{yp}$$

The combined shear and bending resistance of the pin shall be sufficient if the following is true:

$$\left[\frac{M_{Ed}}{M_{Rd}}\right]^2 + \left[\frac{F_{V,Ed}}{F_{V,Rd}}\right]^2 \leq 1$$

where M_{Ed} and $F_{v,Ed}$ are the forces applied to the pin in ultimate limit state.

The bearing resistance of the plate and the pin, assumed to be made of steel with a yield strength limit $f_y \leq f_{yp}$ against a pin of diameter d made of steel with an yield strength f_{yp} is given by:

$$F_{b,Rd} = 1.5\ t\ d\ f_y / \gamma_{M0}$$

and the following must be true:

$$F_{b,Ed} \leq F_{b.Rd}.$$

If the pin is intended to be replaceable, the bearing resistance th in serviceability limit state $F_{b,Rd,ser}$ is given by:

$$F_{b,Rd,ser} = 0.6\ t\ d\ f_y$$

and the following should be true:

$$F_{b,Ed,ser} \leq F_{b,Rd,ser}$$

Moreover, if the pin is intended to be replaceable, the local contact bearing stress should satisfied.

$$\sigma_{H,ser} = 0{,}591\sqrt{\frac{E F_{b,Ed,serv}(d_0 - d)}{d^2 t}} \leq 2{,}5 f_y$$

58.10. Distribution of forces between bolts

The distribution of forces between different bolts in a joint in the ultimate limit state shall be made by means of linear methods in the following cases:

- Category C joints.
- Category A or B joints when the shear resistance $F_{v.Rd}$ of the bolt is less than the compression strength $F_{b.Rd}$ of the contiguous part.
- If the joint is subjected to impacts, vibrations or alternating load cycles (except wind).

In all other cases, the distribution may be made using elastic or plastic means, provided that the equilibrium deformation compatibility and plasticity conditions are fulfilled.

In any case, a bolted joint shall be deemed secure if the force F_{Ed} in the most stressed bolt is equal to or less than the shear, compression or slip strength of said bolt F_{Rd}, as applicable, i.e. if the following is true:

$$F_{Ed} \leq F_{Rd}$$

Section 59. Welded connections

59.1. General

The welded joints covered by this Code should conform to provisions of Section 77.

The steel to be joined by welding and the weld materials shall comply with the provisions of section 29.5.

In all cases, the weld material should provide mechanical, yield strength and ultimate tensile strength characteristics at least as good as those of the base metal.

The welds covered by this Code apply to material thickness of 4 mm and over.

59.1.1. Qualification

A welded joint shall not be deemed to have been made in accordance with this Code if it was not made using a qualified welding process, as specified in section 77.4.

59.1.2. Authorised welding methods

Although all of the methods provided for in section 77.3 are valid, the methods most commonly used in the steel structures to which this Code relates are listed below:

- Manual covered-electrode welding, with rutile or basic coatings. This is process 111, manual electric-arc welding including rutile, basic and other coatings.

- Gas-shielded semi-automatic welding, with solid or tubular flux-filled wire and spray transfer. These are processes 135 and 136.

- Semi-automatic welding, with tubular flux-filled wire and no gas shielding, with spray transfer. This is process 114.

Automatic submerged arc welding. This is process 121.

Authorisation should be sought from Project Management to use other procedures or electrodes with different coatings.

In any case, the mechanical characteristics of the weld metal shall be equal to or better than the characteristics of the steel in the parts to be welded.

59.2. Types of joints and chords

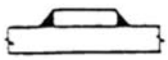
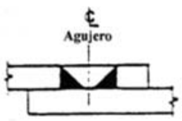
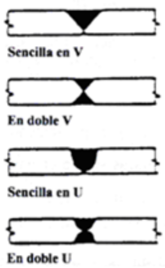
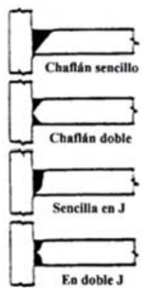
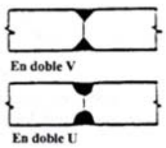
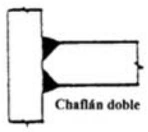
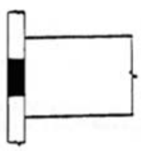
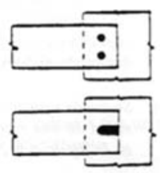
Depending on the relative positions of the parts to be joined, welded joints may be butt, T or lap joints (Table 59.2).

Weld chords to be deposited between parts to be joined to form a welded joint are divided into fillet weld chords and butt weld chords. Moreover, welds can be made on one side or on both sides.

Fillet weld chords are known as side when they are parallel to the direction of the force they transmit, frontal when they are perpendicular to said force and oblique in intermediate cases. In any case, they may be continuous or discontinuous.

Butt weld chords can be full penetration or partial penetration.

Table 59.2. Types of welded joints according to the relative position of the parts

Weld type	**Joint type**		
	Butt joint	**T butt joint**	**Lap joint**
Fillet weld			
Slot weld			 Hole
Full-penetration butt weld*	 Single V, double V, single U, double U	 Single bevel, double bevel, single J, double J	
Partial-penetration butt weld*	 Double V, double u	 Double bevel	
Plug weld			

59.3. Constructional details for fillet weld chords

59.3.1. General

Fillet weld chords may be used to connect parts where fusion faces form angles of between 60º and 120º.

For angle between 45º and 60º, the seam is deemed partial penetration.

For angle greater than 120º or less than 45º, the seam is considered a simple binding with no capacity to withstand forces, unless its strength is proven by testing.

59.3.2. Throat thickness

Notwithstanding the provisions for joints between parts of tubular section or box sections that cannot be accessed internally, the throat thickness a of a fillet weld seam shall not exceed $0,7\ t_{min}$, where t_{min} is the thickness of the thinnest part to be joined.

The throat thickness a of a fillet weld seam (section 59.7) should not be less than 3 mm when depositing on sheets up to 10 mm thick, or less than 4.5 mm when depositing on parts up to 20 mm thick, or less than 5.6 mm when depositing on parts more than 20 mm thick, unless the welding procedure provides for smaller throat thicknesses.

To prevent potential cold cracking issues, if the thickness of one of the parts to be joined is greater than twice that of the other part, it is advisable to prepare a suitable welding process that takes the carbon equivalent C_{eq} of the base material, the heat generated by the welding process and the possibility of having to preheat the base metal into account.

59.3.3. Termination

Fillet weld chords should not terminate in the corners of the parts or members thereof, but they should continue around the corner, provided that the extension can be made in the same plane as the seam, to a length at least three times the throat a of the seam. Said extension must be included in the drawings.

59.3.4. Intermittent fillet welds

In structures stressed by predominantly static loads in environments with C1 or C2 corrosivity, discontinuous fillet chords may be used if the thickness of the throat required by the verification calculations is less than the minimum recommended in section 59.8. Discontinuous chords may not be used in environments with greater than C2 corrosivity.

- In discontinuous chords, the gap L_1 between the ends of the partial chords, on the same or different faces, in parts under tension (Figure 59.3.4) shall be less than the smaller of the following values:200 mm or 16 times the thickness of the thinnest part to be joined.

- In discontinuous chords, the gap L_2 between the ends of the partial chords, on the same or different faces, in parts under compression or shear force (Figure 59.3.4) shall be less than the smaller of the following values: 200 mm, 12 times the thickness of the thinnest part to be joined or *0.25b.*

- The length L_w of each partial seam shall be larger than the greater of the following values:6 times the seam throat or 40 mm.

- The length L_{we} of each partial seam at the ends of the parts to be joined shall be larger than the smaller of the following values:*0.75b* or *0.75b₁*.

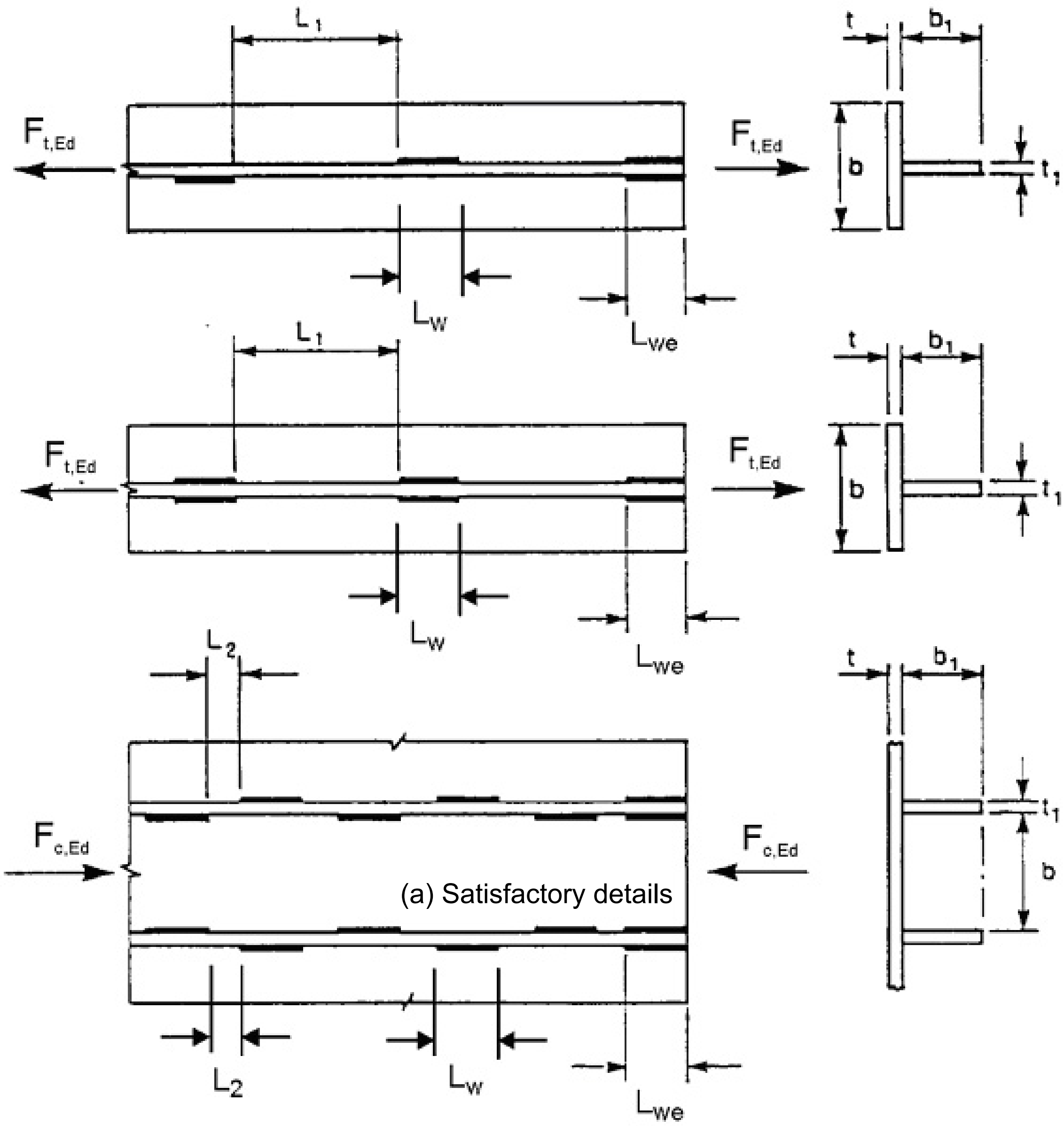
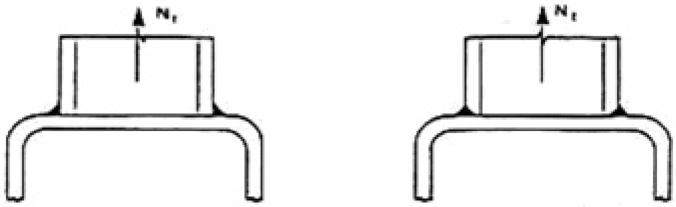


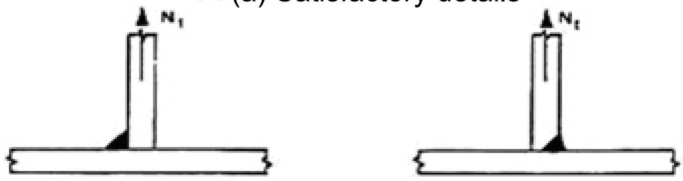
Figure 59.3.4

59.3.5. Eccentricity

Fillet weld chords shall be placed so as to prevent the occurrence of bending moments whose axis is the same as the seam (Figure 59.3.5).



(a) Satisfactory details



(b) Non-satisfactory details

Figure 59.3.5

59.3.6. Minimum length of side chords

Side fillet weld chords that transmit axial forces of bars shall be at least 15 times as long as their throat width and not less than the width of the part to be joined (Figure 59.3.6).

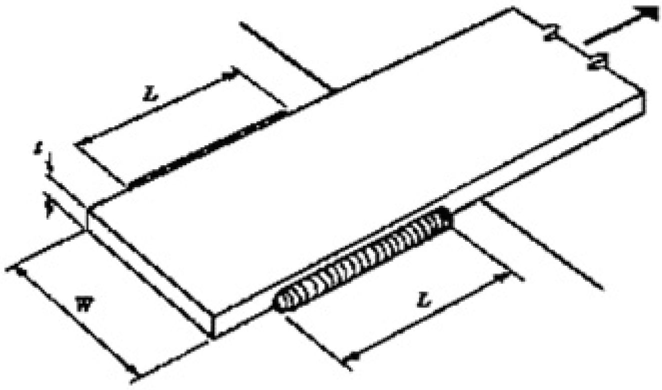


Figure 59.3.6

59.3.7. Laps

In lap joints, the minimum lap shall not be less than *5tmin*, where *tmin* is the smaller thickness of the parts to be joined, or 25 mm. If the joint is required to transmit axial forces, frontal fillet weld chords should be made on both ends of the lapped parts (Figure 59.3.7).

(a) Valid

(b) Not valid

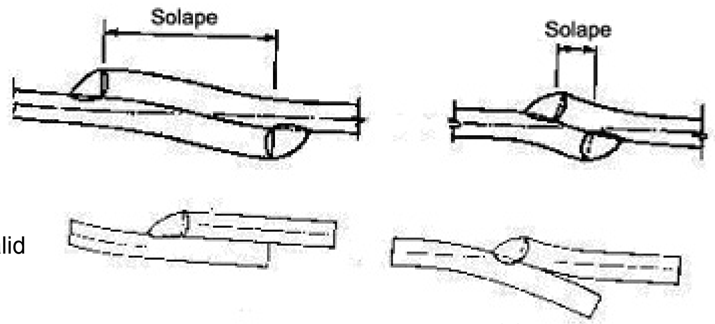


Figure 59.3.7

Solape	Overlap

59.4. Constructional details for butt weld chords

Butt weld chords may be made between parts arranged in extension or in a T (Table 59.2). They are full penetration if the weld metal takes up the full thickness of the parts to be joined with no defects.

They are partial penetration if the thickness taken up by the weld metal is less than the full thickness of the parts joined.

Partial-penetration butt weld chords shall be placed so as to prevent the occurrence of parasitic bending moments whose axis is the same as the seam (Figure 59.3.5).

Intermittent butt weld chords are not permitted.

59.5. Plug and slot welds

Plug welds, which fill circular or slotted holes made in a part overlapping another, may only be used if there is no other way of transmitting shear or preventing buckling or separation of lapped parts. They should not be used to transmit tensile forces.

Suitable provisions must be made, in accordance with section 77.5.10, to prevent possible cracking, and they must be inspected in accordance with the provisions of section 91.2.

The following limitations shall apply:

- The diameter of the circular holes or the width of the slotted holes shall not be less than the thickness of the part they are in plus 8 mm.

- The ends of the slotted holes should either be semi-circular or else should have rounded corners with a radius no less than the thickness of the part, except if the hole reaches the edge of the part.

- The thickness of a plug weld in parent material up to 16 mm thick shall be the same as the thickness of the parent material. If this is over 16 mm, the thickness of the weld metal must be at least half the thickness of the parent material and not less than 16 mm.

- The minimum distance between plug centres perpendicular to the force to be transmitted shall not be less than 4 times the width of the plug. The distance between plug ends should not be less than twice their length.

- The maximum distance between plugs must not be larger than the value required to preventing local buckling of the part.

Filling holes made in the structure to insert provisional assembly bolts with weld metal is strictly forbidden. Consequently, such holes shall be arranged such as not to affect the strength of the structure.

Slot welds (Figure 59.5) formed by fillet weld chords placed inside circular or slotted holes may only be used to transmit shear forces or to prevent buckling or separation of lapped parts.

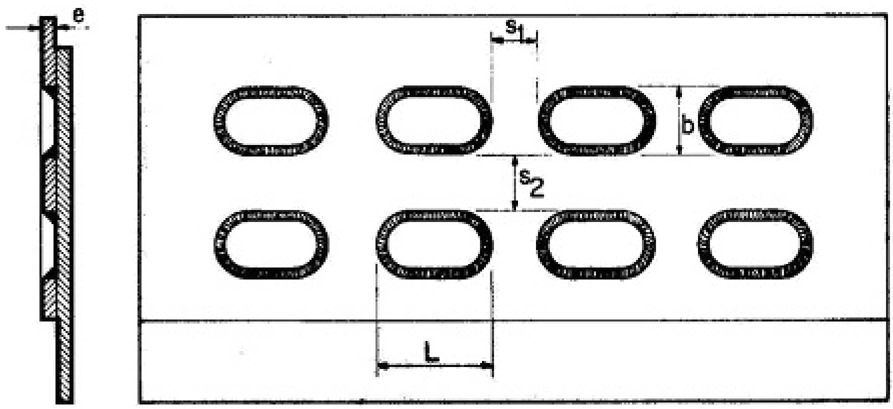


Figure 59.5

The following limitations shall apply to slot dimensions:

- The width *b* of the slot shall be $b \geq 4t$.
- The gap s_2 between rows of slots shall be $2b \leq s2 \leq 30t$.
- The gap between slotss_1 shall be $2b \leq s1 \leq 30t$.
- The total length *L* of a slot shall be $L \leq 10t$.
- Slot ends shall be semi-circular, unless they reach the end of the part.

59.6. Lamellar tearing

Residual through-thickness stresses in the parts to be joined, which could result in lamellar tearing of the parts to be joined, shall be avoided as much as possible.

In particular, if there are through-thickness tensile stresses (whether residual from welding or due to external forces) in flat parts more than 15 mm thick, the welding procedure, the through-thickness properties of the base metal and the details of the joint (Figure 59.6) must be studied to avert this danger.

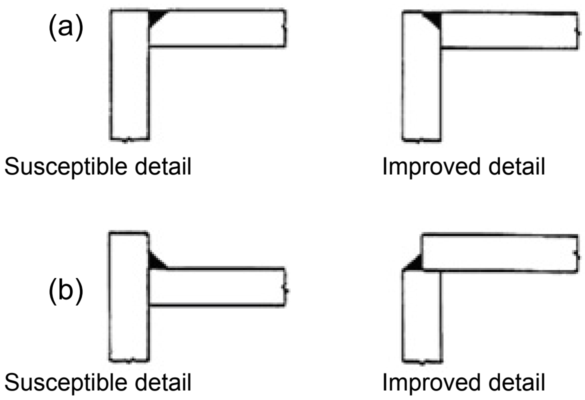


Figure 59.6

59.7. Effective throat thickness

Throat thickness should be taken as the height of the largest triangle that can be inscribed in the section of the weld metal, measured perpendicular the outer side of this triangle (Figure 59.7.a).

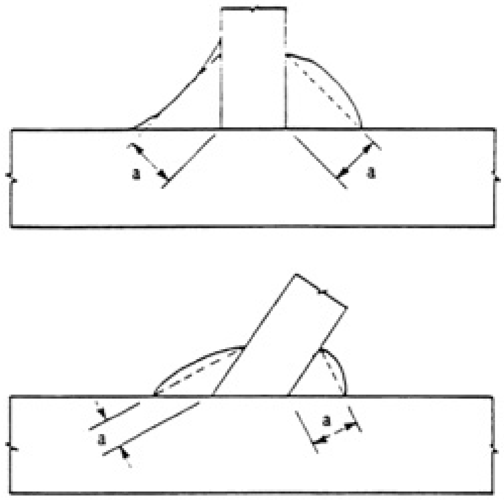


Figure 59.7.a Throat thickness of a fillet weld

If the welding method used enables deep penetration to be achieved, this penetration may be included in the throat thickness value (Figure 59.7.b), provided that preliminary tests show that the required penetration can consistently be achieved constantly.

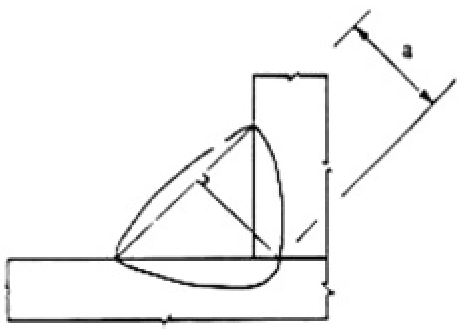


Figure 59.7.b Throat thickness of a deep penetration fillet weld

If the seam is made using the submerged-arc method, throat thickness may be increased by *2 mm* if *a >10 mm* or by *0.2 a* if *a ≤ 10 mm* without the need to perform any tests.

If it is necessary to deposit a weld seam between two curved surfaces (such as on rounds, corners of hollow sections or between a curved and a flat surface, Figure 59.7.c) effective throat thickness should be determined on the basis of the test welds made on test parts of the same section to be used in production.

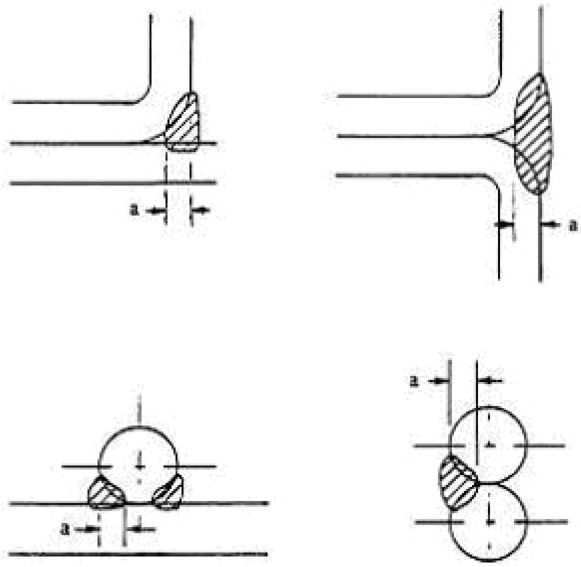


Figure 59.7.c

Specimens shall be cut and the throat thickness measured, repeating the process until a welding technique that guarantees the desired throat thickness in production is found.

The plane of the throat is the plane defined by the throat and the seam axis, the intersection of the two planes to be joined by the weld seam (Figure 59.8).

59.8. Design resistance of fillet weld

This method refers to a section of fillet weld seam that is sufficiently short to assume that it is subject to uniform stress throughout.

It shall be assumed that the forces transmitted by said seam section leading to the direct stresses $\sigma_{//}$, perpendicular to the seam axis and have no effect on seam strength σ and $_{\perp}$, perpendicular to the throat, and in the shear I stresses $\tau_{//}$ and $\tau_{\perp}$, in the plane of the throat parallel and perpendicular to the axis of the weld, respectively, (Figure 59.8).

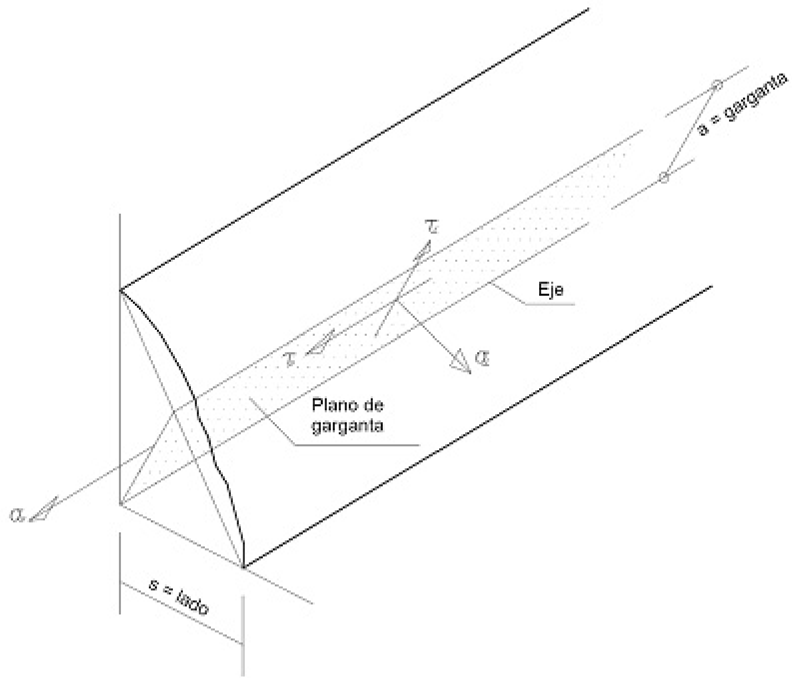


Figure 59.8

Lado	Side
Plano de garganta	Throat plane
Eje	Axis
garganta	Throat

59.8.1. Effective length of a fillet weld seam

The effective length of a fillet weld seam is the length (including corner extensions) over which the nominal throat thickness is maintained.

Chords whose length is less than 30 mm or 6 times the throat thickness are not deemed capable of transmitting forces.

The effective length of weld chords in beam-support joints shall be calculated in accordance with the provisions of section 62.1.1.

In lap joints in which $L_w \geq 150\,a$, where L_w is the seam length, the effective length $L_{w,ef}$ shall be $L_{w,ef} = \beta_1 L_w$, in which β_1 is given by:

$$\beta_1 = 1{,}2 - \frac{0{,}2 L_W}{150 a} \leq 1$$

This reduction shall not apply if the distribution of stresses along the weld is equal to the distribution of stresses in the contiguous base metal, such as when welding a flange-web joint in sheet trussed beams.

For fillet weld chords longer than 1 700 mm that join transverse stiffeners on sheet members, $L_{w,ef} = \beta_2 L_w$ in which:

$$o{,}6 \leq \beta_2 = 1{,}1 - \frac{L_W}{17000} \leq 1{,}0$$

L_w being in mm.

59.8.2. Strength

Seam strength is sufficient if the following two conditions are met simultaneously:

$$\sqrt{\sigma_{\perp}{}^2 + 3(\tau_{\perp}^2 + \tau_{//}^2)} \leq \frac{f_u}{\beta_W \gamma_{M2}}$$

$$\sigma_{\perp} \leq 0{,}9 \frac{f_u}{\gamma_{M2}}$$

where f_u is the tensile strength of the steel of the parts to be welded, β_w is the correlation factor, which depends on the type of steel in the parts to be welded, and γ_{M2} is the strength reduction coefficient, $\gamma_{M2} = 1.25$.

Table 59.8.2 sets out the values of β_w for the most common steels.

Table 59.8.2 Correlation factors

STEEL GRADE	CORRELATION FACTORS β_w
S 235	0.80
S 275	0.85
S 355	0.90
S 420	1.00
S 460	1.00

If a seam of throat thickness a and length L_w has to transmit a force F that forms an angle α with the seam axis, the above conditions shall be met if the average shear stress in the seam meets the following condition:

$$\tau_W = \frac{F}{aL_W} \leq \frac{f_u}{\beta_W \gamma_{M2} \sqrt{1 + 2\cos^2 \alpha}}$$

Safety is ensured if, for any value of *α,* the following is true:

$$\tau_W = \frac{F}{aL_W} \leq \frac{f_u}{\beta_W \gamma_{M2} \sqrt{3}}$$

In flange-web joints in trussed beams that only transmit tangential forces, made with two continuous chords of throat thickness a on a web of thickness t_w, the above conditions shall be met if $a \geq 0.40\ t_w$ for the steels covered by standard UNE-EN 10025 parts 2 and 5, and for steels S275 and S355 in UNE-EN 10025, parts 3 and 4, or if $a \geq 0.55\ t_w$ for steels S420 and S460 in UNE-EN 10025, parts 3 and 4.

If a flat part of thickness t_f, subject to axial force, is joined to another perpendicular to it via two frontal fillet weld chords of the same length as the part and of throat a, the above conditions shall be deemed met if $a \geq 0.55\ t_w$ for steels covered by standard UNE-EN 10025 parts 2 and 5 and for steels S275 and S355 in UNE-EN 10025, parts 3 and 4.

59.9. Design resistance of butt weld

59.9.1. Full-penetration butt welds

Having fulfilled the conditions specified in section 59.1 of this Code, the resistance of a full-penetration weld with no defects is at least the same as that of the weakest contiguous base metal. Therefore, the strength need not be calculated.

59.9.2. Partial-penetration butt welds

Design resistance of a partial-penetration butt weld with no defects is the same as that of a fillet weld of the same throat thickness, and consequently it shall be checked using the method given in section 59.8, above.

The throat thickness used shall be equal to the permanently achievable penetration, which should be checked using suitable tests.

If V, U or J edges are prepared, the throat thickness value (Figure 59.9.2.a) may be taken as the depth of the edge preparation less 2 mm, without the need for testing. A higher value may be used if this is shown to be appropriate by suitable testing.

In T-butt joints (Figure 59.9.2.b) with two partial-penetration butt seam welds reinforced with two fillet weld chords, the joint can be assumed to be equivalent to a full-penetration butt weld if the sum of the nominal throat thicknesses is equal to or greater than the thickness of the part to be joined and the unwelded thickness would be the smaller of t/5 and 53 mm, where *t* is the thickness of the part.

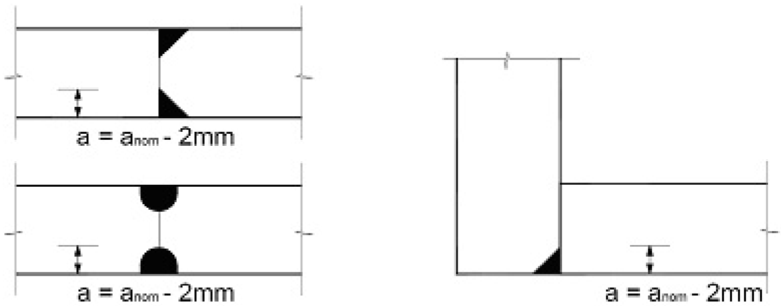


Figure 59.9.2.a

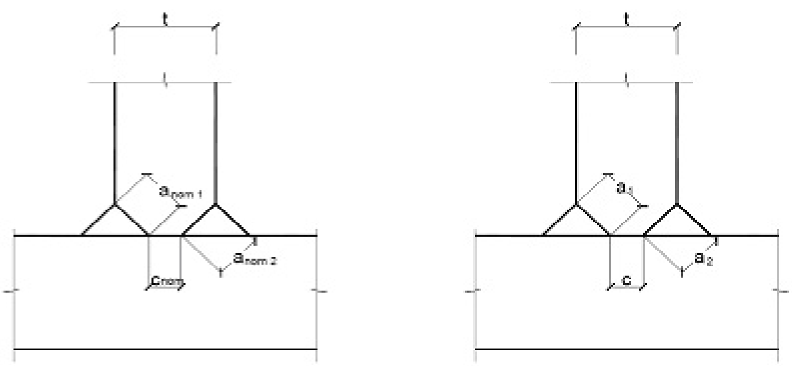


Figure 59.9.2.b

If any one of the above conditions is not met, the throat thickness shall be determined in accordance with provisions of sections 59.7 and 59.8.

This type of seam shall not be used in joints subjected to fatigue stressing or that are required to transmit tensile forces perpendicular to their axis.

59.10. Design resistance of plug or slot welds

The design resistance of a plug or slot weld should be taken as

$$\tau_w = \frac{F_{W,Sd}}{A_W} \leq \frac{f_u}{\beta_W \gamma_{M2} \sqrt{3}}$$

where τ_w is the average shear stress in the weld, $F_{w,\ Ed}$ is the design force to be transmitted and A_w is the area of the hole for plug welds or the product aL_w for slot welds. In this case, a is the throat thickness and L_w the seam length, measured at a distance equal to *a/2* from the inner edge of the slot.

The other symbols have the same meaning as in section 59.8.

59.11. Distribution of forces between joint chords

The forces acting on the joint should conform to provisions of Chapter II (Calculation Bases) and Chapter IV (Structural Analysis) and section 56.2 of this Code.

The distribution of said forces among the different chords of the joint may be made assuming elastic or plastic performance of the joint. In general, it is acceptable to assume a simplified distribution of forces within the joint, provided that the conditions set out in the aforementioned section are met.

It is not necessary to take account of the residual stresses or other stresses that do not affect the transmission of forces between different parts, such as the stress $\sigma_{//}$ on planes perpendicular to the seam axis.

Section 60. Joints between members subject to axial force

60.1. Centred flat joints

Centred flat joint means a joint made using a series of coplanar weld chords or a set of bolts joining essentially coplanar parts, that are intended to transmit, between the parts to be joined, a force $F_{w,Ed}$ contained in the plane of the chords of the parts, and that passes through the centre of gravity of the chords or bolts.

60.1.1. Welded centred flat joints

The calculation method below may be used for joints in which the chords are not in the same plane and for joints in which all of the chords are in the same plane and the force $F_{w,Ed}$ is not within it, provided that it passes through the centre of gravity of the chords (Figure 60.1.1).

If a more precise analysis, taking into account the strength of each seam as a function of its orientation in relation to the force $F_{w,Ed}$, is not required, then a distribution of forces proportional to the throat areas of the different chords may be assumed, and consequently the joint shall be safe it the following is true:

$$\frac{F_{W,Ed}}{\sum a_i L_{Wi}} \leq \frac{f_u}{\beta_W \gamma_{M2} \sqrt{3}}$$

where a_i and L_{wi} are the throat thicknesses and the lengths of the different chords that form the joint. When determining each value of $L_{wi,}$. it should follow the provisions of section 59.8.1

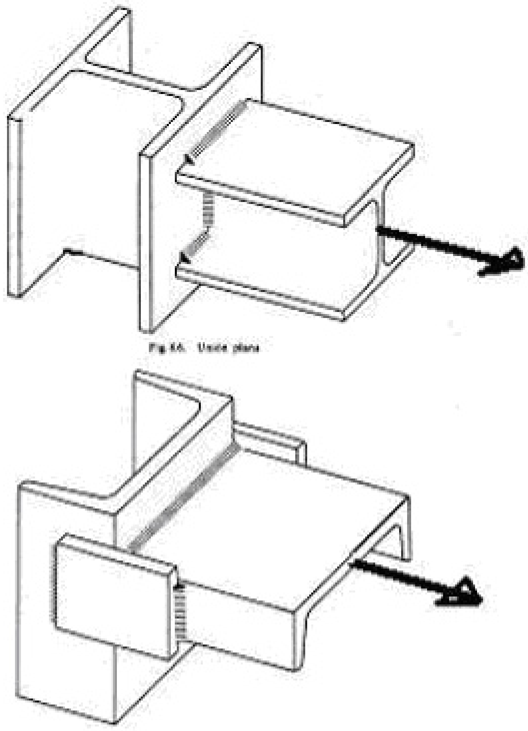


Figure 60.1.1

60.1.2. Bolted centred flat joints

These are joints in which the force to be transmitted passes through the centre of gravity of the group of bolts intended to transmit it.

A uniform distribution between all of the bolts of the joint may be assumed if the length L_j of the joint, measured in the direction of force to be transmitted between the centres of the edge bolts, does not exceed *15d.*In this case, the design strength of the joint shall be:

$$N_{Rd} = n\ F_{Rd}$$

where n is the number of bolts and F_{Rd} the strength of one of them, determined as a function of the joint category and the provisions of Section 58.

If the length of the joint is more than *15 d*, its strength is given by:

$$N_{Rd}=\beta\ n\ F_{Rd}$$

where β is a reduction factor given by:

$$\beta = 1 - \frac{L_j - 15d}{200d}$$

with $0.75 \le \beta \le 1$; and L_j defined in Figure 60.1.2.

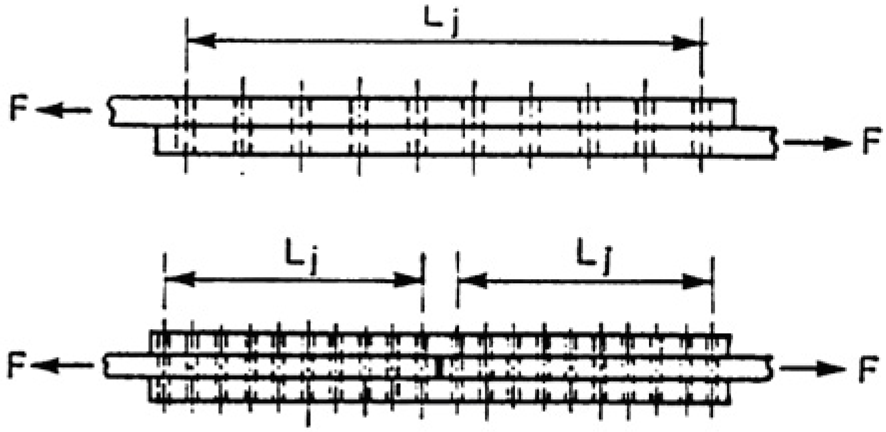


Figure 60.1.2 Long joints

This provision does not apply where the distribution of stresses along the joint is equal to the distribution of stresses in contiguous parts, e.g. in the case of long joints with double angle bearing (section 61.3).

60.2. Eccentric flat joints

These are flat joints, welded or bolted, in which the force to be transmitted $F_{w,Ed}$ passes at a given distance from the centre of gravity of the group of weld chords or bolts (Figure 60.2).

If the joint is studied using plastic methods, it is assumed that one of the parts to be joined moves in relation to the other, rotating around a given instant centre of rotation (Figure 60.2) to be determined.

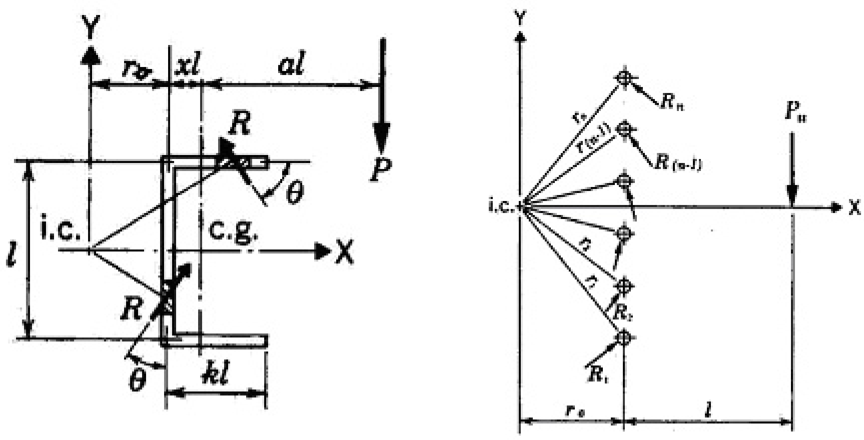


Figure 60.2

The deformation in each point of the weld chords or in each bolt shall be assumed to be proportional to the length of the radius vector joining the point to the instant centre of rotation and perpendicular to it. The average stress in each point τ_w can be determined using specialist literature. The position of the instant centre of rotation shall be determined under equilibrium conditions.

60.2.1. Welded eccentric flat joints

On account of such rotation, the average shear stress τ_w at any point of the weld chords is proportional to the length of the radius vector joining the point to the instant centre of rotation and perpendicular to it.

Where I_p is the polar moment of inertia of the area of the chords in relation to their centre of gravity:

$$I_p = \int_A (y^2 + z^2)\, dA$$

the components τ_{wz} and τ_{wy} are given by:

$$\tau_{wz} = \frac{M_{Ed}\, y}{I_p}; \qquad \tau_{wy} = \frac{M_{Ed}\, z}{I_p}$$

These components should be added vectorially to those generated by $F_{w,Ed}$ acting on the centre of gravity of the chords. The joint will be safe if the following is true at the least favourable point, which is often the point farthest from the centre of gravity of the chords:

$$\left|\tau_{w,\max}\right| \le \frac{f_u}{\beta_w \gamma_{M2} \sqrt{3}}$$

60.2.2. Bolted eccentric flat joints

Equally, the relative rotation between the parts to be joined generates a force in each bolt $F_{i,Ed}$ proportional to the length of the radius vector joining the point with the instant centre of rotation and perpendicular to it.

That is: $$I'_p = \sum_{i=1}^{n} (y_i^2 + z_i^2)$$

the sum of the squares of the lengths of the n radius vectors. The components $F_{iy,Ed}$ and $F_{iz,Ed}$ of the force $F_{i,Ed}$ in each bolt caused by the moment $M_{Ed\,i}$, are given by:

$$F_{iy,Ed} = \frac{M_{Ed} z_i}{I'_p}; F_{iz,Ed} = \frac{M_{Ed} y_i}{I'_p}$$

where z_i and y_i are the coordinates of the axis of the corresponding bolt.

These components should be added vectorially to those generated by F_{Ed} acting on the centre of gravity of the chords. The joint will be safe if the following is true of the least favourable bolt, which is often the bolt farthest from the centre of gravity of the set of all of the bolts:

$F_{Ed} \le F_{Rd}$

where F_{Rd} is the bolt strength, calculated in accordance with the provisions of Section 58.

60.3. Joints with gusset plates

Gusset plates are pieces of sheet that are used to facilitate the joining of brace members to the chords in nodes of triangular structures. It is not common to use them when the bars of the structure are tubular sections.

Brace members are joined to the gusset plate or plates by welding. The axis of each bar should ideally coincide with the axis of the strength members of the joint of said bar, or weld, to the gusset plate or plates. If this is not possible, the existing eccentricity shall be taken into account to check the joint and the bar itself.

If the chord of a Warren triangular structure is an H- or I-beam with the web in the same plane as the structure (Figure 60.3.a), the joint between the gusset plate and said chord is generally subject to a force H equal to the difference between the axial forces on either side of the node, $H = N_1 - N_2$ and a moment $M = H$ e, where e is the distance between the gusset plate-beam joint weld and its centre of gravity.

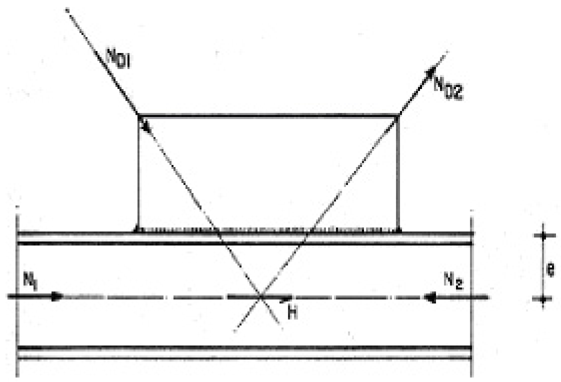


Figure 60.3.a

In Pratt-truss nodes (Figure 60.3.b) it can be assumed that the force in the diagonal is balanced with horizontal and vertical forces on the sides of the gusset plates joined to the upright and to the chord, selected such that they comply with the equilibrium conditions and the existing weld chords between gusset plates and parts are able to transmit them.

Reasonable results can be obtained assuming that said forces pass via theoretical intersection points *B* and *D* of the edges of the gusset plate, if extended, with the axes of the upright and the chord; via the centre *A* of the longest side of the gusset plate and that cross over the axis of the diagonal, point *C*.

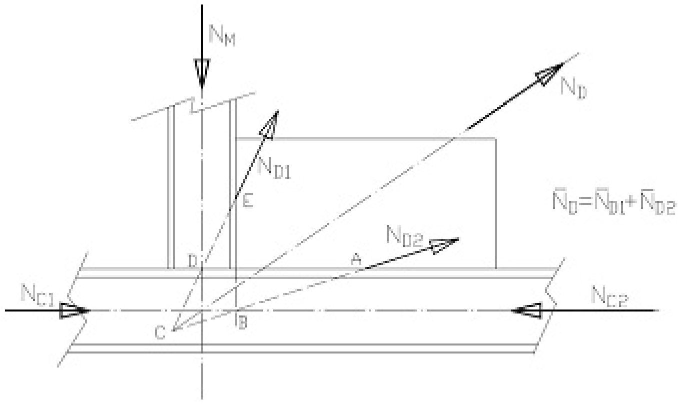


Figure 60.3.b

If, as a result of the shape of the section of brace members (I, U or H) it is necessary to use a double gusset plate and the web of the diagonal or upright is not extended, this should be taken into account, using the value as the design section of member for local checks on the node (Figure 60.3.c).

$$A_{ef} = A\left(1 - \frac{x}{L_w}\right)$$

This area reduction shall not be taken into account when performing the general part stability check.

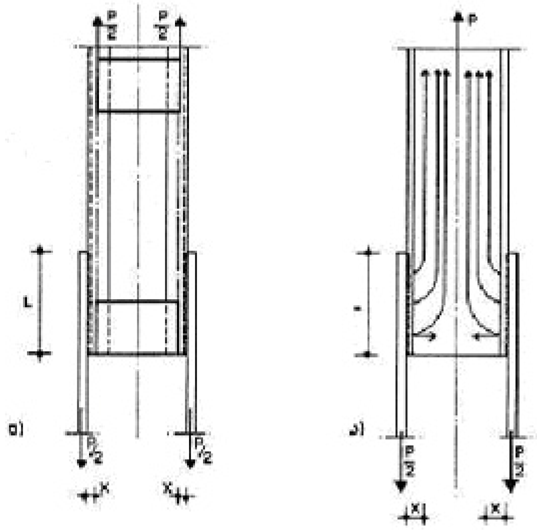


Figure 60.3.c

The same expression may be used to take account of the eccentricity caused if two paired beams form the brace member. In this case, a batten plate should be placed on the end of the part located between the gusset plates.

The thickness of the gusset plate t shall be such it complies with Figure 60.3.d:

$$\frac{F_{c,Ed}}{t(b+L)} \leq \frac{f_y}{\gamma_{M0}}$$

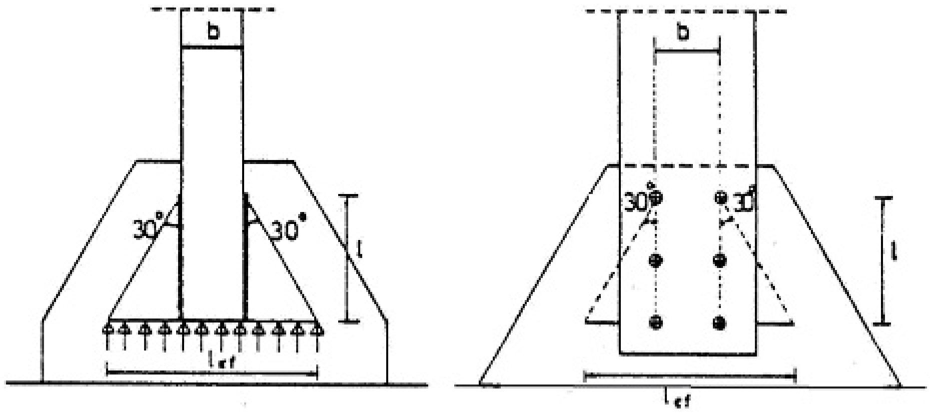


Figure 60.3.d

In the expression above, $F_{c,Ed}$ is the force transmitted to the gusset plate, b is the width of the part perpendicular to the force $F_{c,Ed}$, L is the length of the weld chords, measured in the direction of the force $F_{c,Ed}$, f_y is the yield strength of the steel in the gusset plate and $\gamma_{M0}$is the strength reduction coefficient.

Section 61. Joints between members subjected to bending and shear

61.1. Splices with splice plates

Splices with splice plates are used almost exclusively with bolted joints. They may only be used in welded joints in repair or reinforcement work.

They are joints between parts with identical or similar profiles, with the axes in line or forming a sufficiently small angle so as to make any deviation force negligible, made using rectangular sheet parts, called splice plates, bolted to the two parts to be joined.

Consequently, a splice with splice plate has two groups of joints: the group formed by the joints between one of the parts and the splice cover, and the group formed by the joints between these and the other part.

The joints between the splice plates and the parts may be category A, B or C, although they are typically designed in category C.

Each portion, flange or web, of the parts to be joined shall have its own splice plates. The flange splice plates may be single (placed on one face only) or double (placed on both faces), while the web splice plates shall be double, unless there are special grounds to do otherwise. Parts that, exceptionally, do not have splice plates, shall not be taken into consideration when checking the strength of the part around the splice.

The centre of gravity of the section of the splice plates used for a given part shall match that of the part section and, unless otherwise justified by a detailed study, its area and second moments of area must be the same or slightly higher.

Regardless of the category of the joints to be spliced, the joint may be rigid and the splice section may contain the same forces that would exist if the two parts to be joined were a single part. To ensure it is stiff there must be no relative rotation caused by potential slipping of the sheets and play in the bolt holes, for which reason category C joints are recommended.

These forces shall be distributed between the different portions of the part, and shall be transmitted through the bolted joints with the corresponding splice plates, in accordance with the following criteria:

- The axial force N_{Ed} to be transmitted between the parts to be joined shall be distributed between the joints of each of the groups proportionally to the section of the portion of the corresponding part.

- The bending moment M_{Ed} to be transmitted between the parts to be joined shall be distributed between the joints of each of the groups proportionally to the second moment of area of the portion of the corresponding part in relation to the appropriate axis of the total part. Alternatively, it may be assumed to be transmitted in full through the flange joints, provided that the flanges are able to transmit the moment without the help of the web.

- The shear force V_{Ed} to be transmitted between the parts to be joined shall be distributed between the joints of each of the groups as a function of the distribution of shear stresses that said shear force generates in the part. In I, H and U sections, this may be resisted exclusively by the web joints.

- The bending moment that the shear force, acting on the splice section, generates on the centre of gravity of the joints on which it is acting, located at a given distance from said splice section, shall be taken into account.

As a result of the forces distributed, each of the flange joints must be able to withstand an axial force N_{Edf}, which may be distributed uniformly between all of the bolts of the joint. The value N_{Edf} is given by:

$$N_{Edf} = N_{Ed}\,\frac{A_f}{A} + M_{Ed}\,\frac{{}_{f}}{\;}\,\frac{1}{h}$$

where N_{Ed} and M_{Ed} are the forces in the splice section, A_f is the area of a flange, I_f is the second moment of area of the set of both flanges relative to the axis of inertia of the part, A and/or the area and second moment of area of the part, and *h*is the moment lever arm. If the flange joints are made with a double splice plate, h shall be the depth of the part less the thickness of the flange. If the joint is made with splice plates on only one side of the flanges, h shall be the distance between the planes of the flange-splice plate union (Figure 61.1).

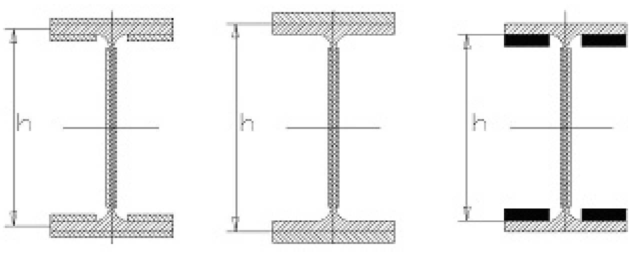


Figure 61.1

The joints between the webs and splice plates shall be subjected to:

- An axial force N_{Edw}, oriented along the directrix of the part, which is assumed to be uniformly distributed between all of the bolts in the joint. The value of N_{Edw} is:

$$N_{Edw} = N_{Ed} \frac{A_w}{A}$$

where N_{Ed} is the axial force in the splice section, Aw is the area of the web and A is the area of the part.

- The shear force V_{Ed}, oriented perpendicular to the directrix of the part, which is distributed uniformly between all bolts of the joint.

- A moment M_{Edw} caused by bending in the splice section and eccentricity of the shear force, having the following value:

$$M_{Edw} = M_{Ed} \frac{w}{\;} + V_{Ed} d$$

where M_{Ed} and V_{Ed} are the bending moment and the shear force in the splice section, I_w is the second moment of area of the web and d is the distance between the splice section and the centre of gravity of the area of the bolts of the joint between the splice plates and one of the parts.

Elastic or plastic methods may be used to determine the force that this moment M_{Edw} generates in each bolt, provided that the conditions specified in section 58.10 are satisfied.

If the elastic method is used, the provisions of section 60.2 shall apply.

61.2. Splices with end-plate

Beam splices using an end-plate (Figure 61.2.a) should be designed as a rigid joint, and the forces in the splice section shall be assumed to be the same as would be present if the two parts to be joined were a single part.

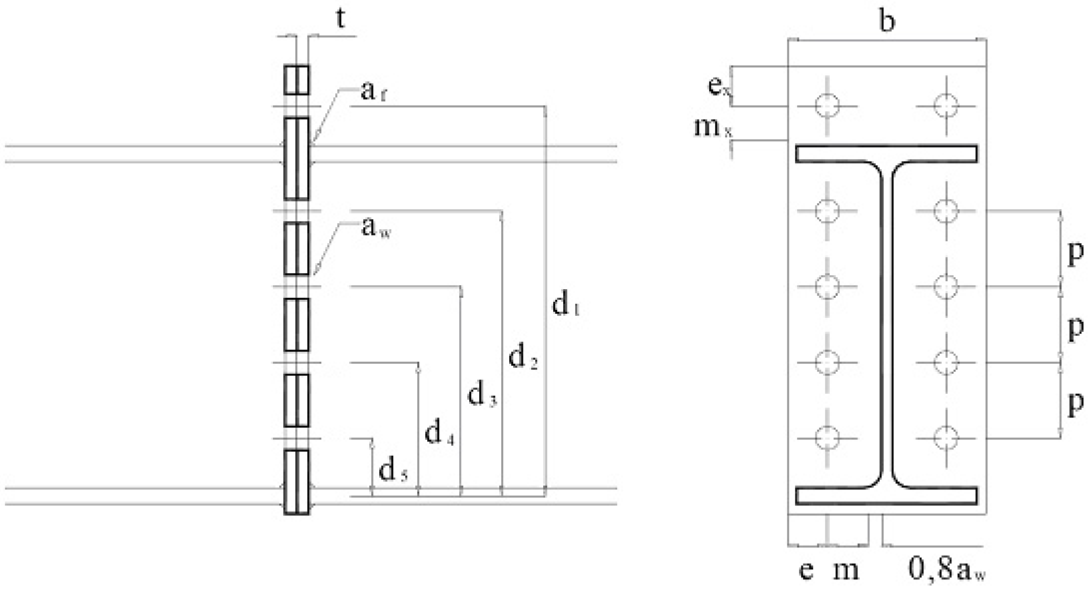


Figure 61.2.a

The thickness of the end-plate should ideally be equal to or greater than the diameter of the bolts.

The distances between the bolts and the beam should ideally be as short as possible, provided that this enables the correct placement and tightening of the bolts ($m_x \approx m \approx 2d$, where *d* is the diameter of the bolts).

Two columns of bolts shall be considered exclusively capable of withstanding axial forces or bending moments, one on either side of the beam web and specifically those closed to it, unless thick or suitably stiffened end plates are used and a detailed study of the joint is carried out. If these precautions are not taken, the remaining columns implemented shall only be deemed capable of withstanding shear forces.

The welded joint between the beams and the end-plates shall be made as strong as the beam itself. To ensure this, the provisions of section 59.8 simply need to be followed. The compatibility of these throats with the thickness of the end plate shall be taken into account.

The use of this type of joint in parts subject to critical fatigue stressing is not recommended without detailed study.

To model the bending strength of the end plate (along with the bolts) and the tensile resistance of the beam web, an equivalent T-stub may be used in tension. The equivalent T-stub may be used for other basic members of bolted beam-support joints (as described below) such as:

- Column flange in bending.
- Column web intension.

If the equivalent T-stub method is used to model a group of bolt rows, the group must be divided into separate bolt rows and an equivalent T-stub should be used to model each bolt row.

If the equivalent T-stub method is used to model a group of bolt rows, the following conditions must be satisfied:

a) The force in each bolt row not exceeds the design resistance determined in consideration of this individual bolt row only.

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

The following parameters must be calculated when determining the design tensile resistance of a basic member represented by an equivalent T-stub flange:

a) The design force in an individual bolt row, determined in consideration of this bolt row only.

b) The contribution of each bolt row to the design resistance of two or more adjacent bolt rows within a group of bolts, determined in consideration of these bolt rows only.

There are three failure mechanisms or failure modes for each equivalent T-stub. A design tension resistance $F_{T,i,Rd}$ shall be determined for each of these failure methods with $i = 1, 2$ *or* 3 and in accordance with the expressions in Table 61.2.a.

In cases when leverage may be generated (see Table 61.2.a) the design resistance of an equivalent T-stub flange $F_{T,Rd}$ must be taken as the lowest of the three possible failure modes (1, 2 and 3).

$$F_{T,Rd} = min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$$

In cases when leverage cannot be generated, the design resistance of an equivalent T-stub flange $F_{T,Rd}$ must be taken as the lower of the two possible failure modes, as per Table 61.2.a.

$$F_{T,Rd} = min(F_{T,1\text{-}2,Rd}, F_{T,3,Rd})$$

The effective lengths for the end plate shall be taken from Table 61.2.b, and the parameters m, e and e_{min} are defined in Figure 61.2.b.

In the case of an individual bolt row, $\Sigma\ell_{eff}$ shall be taken as the effective length ℓeff given in Table 61.2.b for this bolt row taken as an individual bolt row.

In the case of a group of bolt rows, $\Sigma\ell_{eff}$ shall be taken as the sum of all of the effective lengths ℓeff given in Table 61.2.b for each corresponding bolt row taken as part of a bolt group.

Table 61.2.a. Design resistance $F_{T,Rd}$ of T-stub flange

	Leverage may be generated, i.e. $L_b \leq L_b^*$.	No leverage.
Mode 1 (no reinforcing plates).	$F_{T,1,Rd} = \frac{4M_{pl,1,Rd}}{m}$	$F_{T,1\text{-}2,Rd} = \frac{2M_{pl,1,Rd}}{m}$
Mode 1 (with reinforcing plates).	$F_{T,1,Rd} = \frac{4M_{pl,1,Rd} + 2M_{bp,Rd}}{m}$	
Mode 2	$F_{T,2,Rd} = \frac{2M_{pl,1,Rd}}{m}$	
Mode 3	$F_{T,3,Rd} = \Sigma F_{t,Rd}$	

Mode 1: Full plastification of the end-plate or flange.

Mode 2: Bolt failure with plastification of the end-plate or the flange

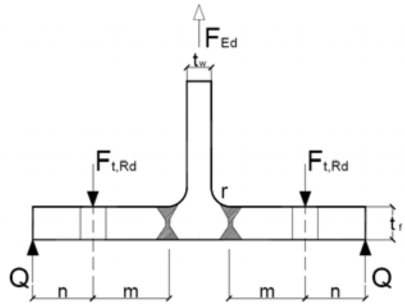
Mode 3: Failure of the bolts under tension.

L_b Stretch length of the bolts, which is equal to the grip length (total thickness of the material and washers), plus half the sum of the height of the bolt head and the height of the nut or:

$L_b{}^* = \dfrac{8{,}8m^3 A_S n_b}{\sum \ell_{eff,1} t_f^3}$

$F_{T,Rd}$ Design tensile resistance of the T-stub flange.

Q Leverage.



$M_{p\ell,1,Rd} = 0{,}25 \sum \ell_{eff,1} t_f^2 f_y / \gamma_{M0}$

$M_{p\ell,2,Rd} = 0{,}25 \sum \ell_{eff,2} t_f^2 f_y / \gamma_{M0}$

$M_{bp,Rd} = 0{,}25 \sum \ell_{eff,2} t_{bp}^2 f_{y,bp} / \gamma_{M0}$

$n = e_{min}$ but $n \leq 1{,}25m$

$F_{t,Rd}$ Design tensile resistance of a bolt (see section 58.7).

$\Sigma F_{t,Rd}$ Total value of $F_{t,Rd}$ for all bolts in the equivalent T-stub.

$\Sigma \ell_{eff}$,1 Value of $\Sigma \ell_{eff}$ for mode 1.

$\Sigma \ell_{eff}$,2 Value of $\Sigma \ell_{eff}$ for mode 2.

e_{min} , m and t_f are as indicated in Figure 61.2.b.

f_y, b_p Yield strength of reinforcing plates.

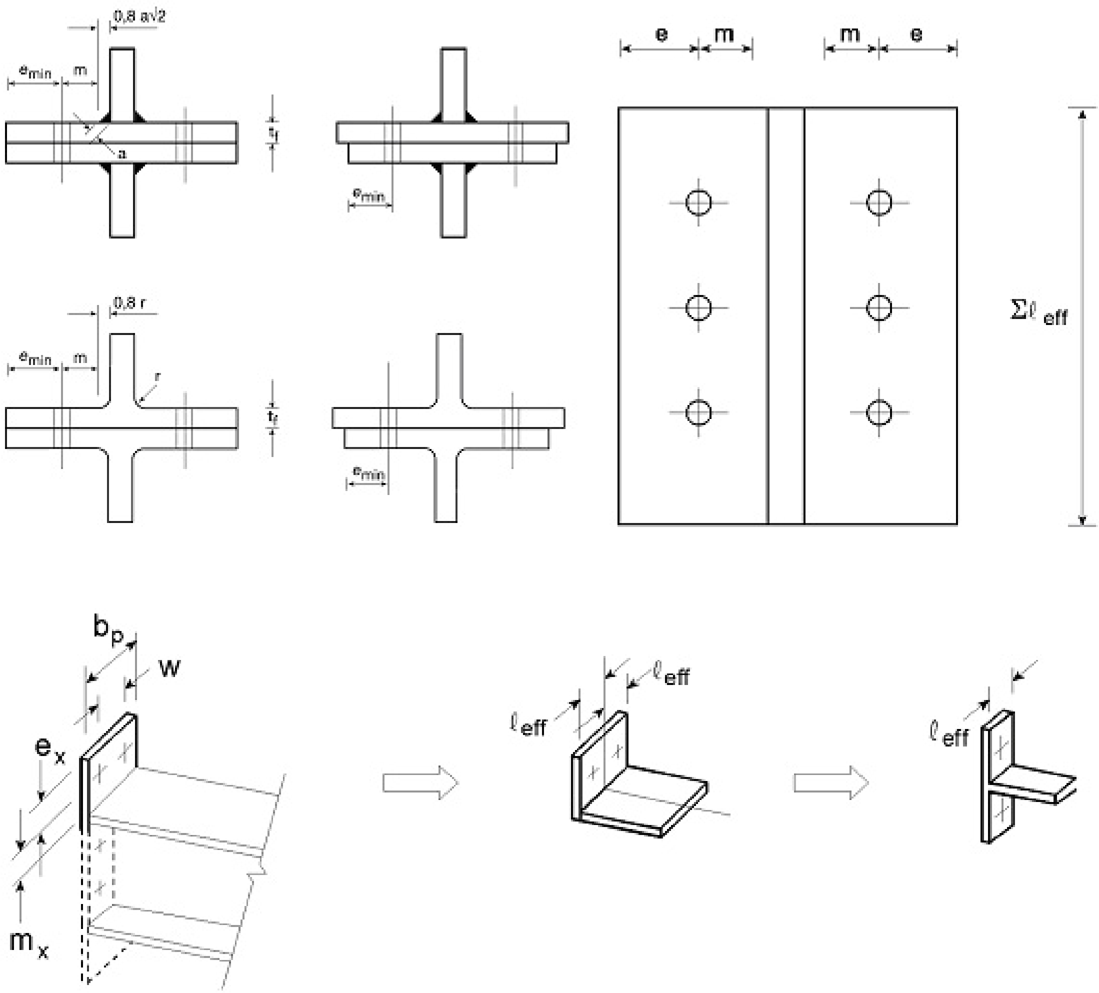
t_{bp} Thickness of reinforcing plates.

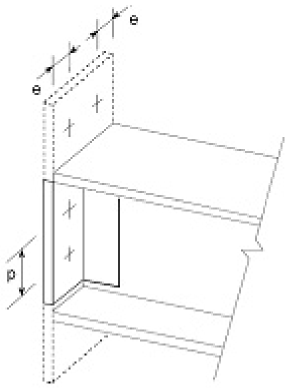
$e_w = d_w / 4$

d_w I Diameter of the washer or width of the head of the bolt or the nut, as applicable.

n_b I Number of bolt rows (with two bolts per row).

NOTE 1: In bolted beam-post joints or beam splices, it may be assumed that leverage is generated.





The extension part of the end-plate and the section between the flanges of the beam are modelled as two separate equivalent T-stubs flanges.

For the end-plate extension, use e_x and m_x instead of e and m to determine the design resistance of the flange of the equivalent T-stub flange.

Figure 61.2.b Modelling an extended front-plate as separate equivalent T-stubs

Table 61.2.b. Effective lengths for an end-plate

of bolt row	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	Smaller of: $2\pi m_x$ $\pi m_x + w$ $\pi m_x + 2e$	Smaller 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	$2\pi m$	Am	πm + p	$0,5p + \alpha m - (2m + 0.625e)$
Other inner bolt row	$2\pi m$	$4m + 1.25\ e$	$2p$	p
Other end bolt row	$2\pi m$	$4m + 1.25\ e$	$\pi m + p$	$2m+0.625e+0,5p$
Mode 1:	$\ell_{eff,1} = \ell_{eff,nc}$ but $\ell_{eff,1} \leq \ell_{eff,cp}$		$\Sigma\ell_{eff,1} = \Sigma\ell_{eff,nc}$ 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 61.2.c.				

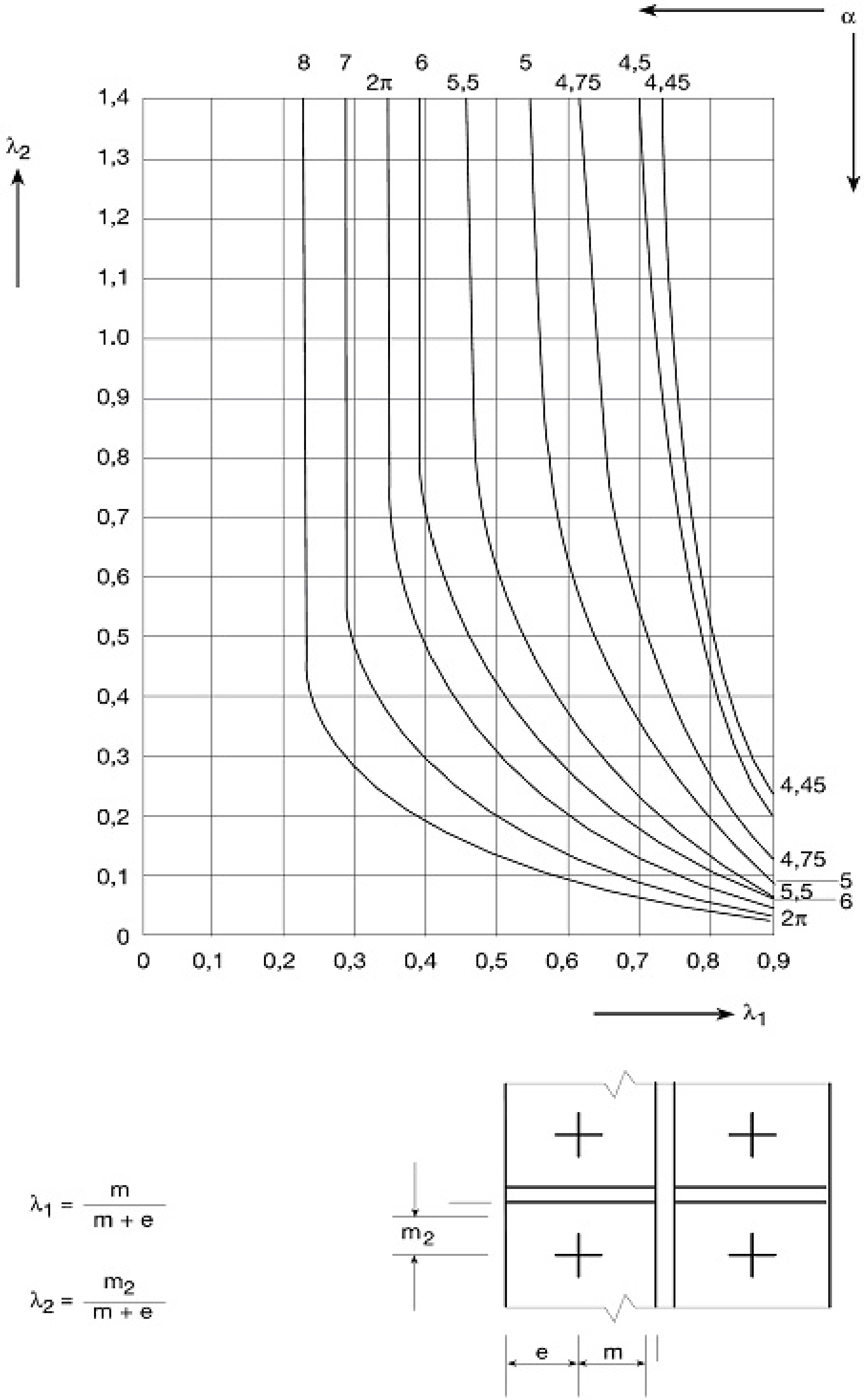


Figure 61.2.c Values of α for stiffened column flanges and end-plates

61.2.1. Moment resistance of a joint

If the splice section is subjected in general to an axial force N_{Ed}, positive for tension, a shearing force V_{Ed} and a bending moment M_{Ed}, the joint shall be checked as follows:

- The value of $F_{T,Rd}$ shall be determined for each of the T-stubs in accordance with the provisions of this section.

- The moment $M_{j,Rd}$ that the joint can resist is given by:

$$M_{j,Rd} = \sum (F_{tr,Rd.i} d_i)$$

where the distance d_i is defined in Figure 61.2.a.

- The following should be true for the joint to be safe $M_{Ed} \leq M_{j,Rd}$.

- The method given in this section to determine the design moment resistance of a joint $M_{j,Rd}$ does not consider any coexisting axial force N_{Ed}in the connected member. Consequently, they should not be used if the axial force in the connected member is greater than 5 % of the design plastic resistance $N_{p\ell,Rd}$ of its cross-section.

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

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

where:

  - $M_{j,Rd}$ is the design moment resistance of the joint, assuming no axial force.

- $N_{j,Rd}$ is the design axial resistance of the joint, assuming t no applied moment - The effective design tension resistance $F_{tr,Rd}$ of bolt row r, as an individual bolt row, shall be the lesser design tension resistance value for an individual bolt row of the following basic members:

  - End-plate in bending: $F_{T,ep,Rd}$

  - Beam web in tension: $F_{t,wb,Rd}$

In a bolted joint with end-plate, the design tensile resistance of the beam web must be obtained by:

$$F_{t,wb,Rd} = b_{eff,t,wb} t_{wb} f_{y,wb} / \gamma_{M0}$$

The effective width $b_{eff,t,wb}$ of the beam web in tension shall be equal to the effective length of the equivalent T-stub that represents the end plate in bending, obtained as per Table 61.2.b, for an individual bolt row or a bolt group.

- The design effective tension resistance $F_{tr,Rd}$ of bolt row r should , if necessary, be reduced below the value obtained previously to ensure that the sum of the design resistance of the preceding bolt rows, including bolt row r which is part of the same group of bolt rows, does not exceed the design strength of such group as a whole. This should be checked for the following basic members:

  - End-plate in bending: $F_{T,ep,Rd}$

  - Beam web in tension: $F_{t,wb,Rd}$

- The design effective tension resistance of $F_{tr,Rd}$ of bolt row r must, if necessary, be reduced below the value obtained in the point above to ensure that, when taking into account all of the previous bolt rows including bolt row r, the total design resistance $\Sigma F_{T,Rd}$ does not exceed the design strength of the flange and web of the beam in compression $F_{c,fb,Rd}$ given by:

$$F_{c,fb,Rd} = M_{c,Rd} / (h - t_{fb})$$

where:

- h is the depth of the connected beam.

- $M_{c,Rd}$ Design moment resistance of the cross-section of the beam, reduced if necessary to take the shear force into account. For a beam reinforced with a bracket, $M_{c,Rd}$ may be calculated ignoring the intermediate flange.

- t_{fb} Thickness of the connected beam flange.

If the depth of the beam including the gusset plate exceeds 600 mm, the contribution of the beam web to the design compression resistance should be limited to 20 %.

- The shear force V_{Ed} acting on the joint shall be assumed to be resisted by bolts working as part of the category A, B or C joint.

61.3. Joints using double angle bearing

A joint using a double angle bracket (Figure 61.3) can be assumed to be a nominally pinned connection, both if it is intended to join a beam to a support and if it is intended to join a beam to a girder or main beam perpendicular to it. In general, its use is recommended if an articulated joint is to be placed at the end of a beam.

The only force to consider is the support reaction V_{Ed} of the beam in the support or mean beam, which is assumed to act on the contact face of the bearings with the support or main beam.

If the joint is to be articulated, the joint between the beam and the angle bearings, as well as the joint between these and the support or the main beam, may be category A bolted or welded.

If the joint is bolted, the bolts may be arranged along the gauge line of the angle irons. If in the joint between the bearings and the beam web, n double-shear bolts are put in place and in the joint between the bearings and the support or main beam $2n$ single-shear bolts identical to the aforementioned bolts are put in place, it is only necessary to check the first of said joints, since the second is working under more favourable conditions.

To maintain the flexibility of the joint, if the joint between the bearings and the support or main beam is made by welding, it should only be welded on the vertical side of the bearings, notwithstanding the extensions to the weld chord made in accordance with the provisions of Section 59.3.3. On the other hand, the weld between the bearings and the beam web shall be made using three welds on each bearing, one vertical and two horizontal.

In the elastic method, the force F_{Ed} on the most stressed screw is given by :

$$F_{Ed} = \frac{V_{Ed}}{n}\sqrt{1+\left(\frac{6w}{b(n+1)}\right)^2}$$

where (Figure 61.3) *w* is the gauge line of the angle and *b* the distance between bolts.

If the joint between the bearings and the support or main beam is made by welding, only the following need be checked:

$$\frac{V_{Ed}}{2ha} \leq \frac{f_u}{\beta\gamma_{M2}\sqrt{3}}$$

The welded joint of the bearings to the beam web shall be checked in accordance with the provisions of section 60.2.1.

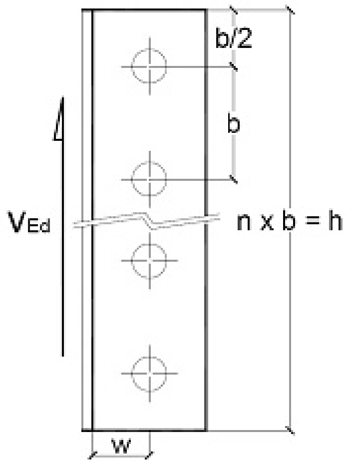


Figure 61.3

61.4. Welded joints

This section includes other types of articulated joints between beams and supports or main beams, done by welding.

61.4.1. Beam joints welded directly to the web

The joint between one beam and another beam or a support (Figure 61.4) using only weld on the beam web may be considered, for calculation purposes, to be a nominally pinned connection that only transmits a shear force V_{Ed}.

In the two chords of this joint, it is advisable to use the maximum throat thickness compatible with the web thickness t_w, $a = 0.7t_w$ to ensure that their length L_w is as short as possible:

$$L_w = \frac{V_{Ed}\beta_w\gamma_{M2}\sqrt{3}}{2a_w f_u} = \frac{1{,}24V_{Ed}\beta_w\gamma_{M2}}{t_w f_u}$$

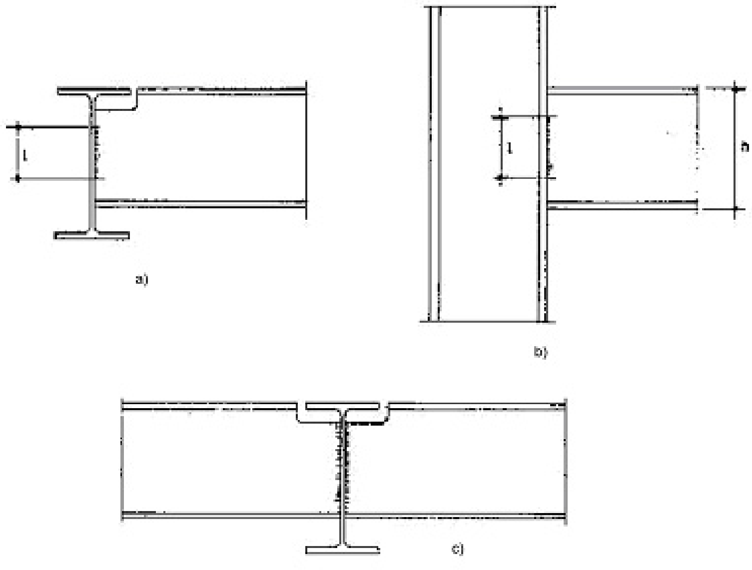


Figure 61.4

To avoid exceeding the breaking stress at the ends of the weld chords, this type of joint may not be used if a length $L_w > 14\ t_w$ is required.

This restriction shall not apply if the beam receiving the joint is sufficiently flexible to enable, without significant restraint, the rotation of the end of the supported beam, as in case (a) in Figure 61.4.

61.5. Support on unstiffened bearing

This type of joint (Figure 61.5) is recommended as a nominally pinned connection at the end of beams or joists in their joint to supports or main beams when the reaction to be transmitted V_{Ed} is not very large.

The angle bearing has a length *b* measured perpendicular to the directrix of the beam, and is welded to the support or main beam with a seam of throat *a* and length *b* effected in the area of the vertex of the angle iron. The seam applied to the lower edge of the flange is not considered for resistance purposes.

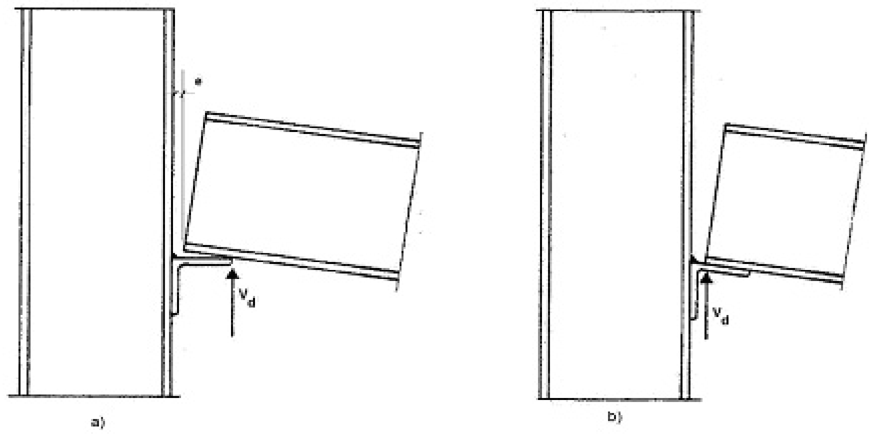


Figure 61.5

The gap e between the end of the beam and the face of the support or main beam should be less than the sum of the thickness of the angle iron plus 50 % of its fillet radius.

In this type of joint, the reaction passes through the end of the beam on account of the flexural stiffness of the flange of the bearing, unless large bearings are used.

For the joint to be safe, the reaction to be transmitted V_{Ed} should be less than or equal to the smaller of the following three values:

Local compression resistance of the beam web:

$$V_{Rd1} = 2{,}5(t_r + r)t_w f_y$$

where t_f, t_w and r are the thicknesses of the flange and beam web and its flange-web fillet radius, and f_y its yield strength.

Weld resistance:

$$V_{Rd2} = ba\frac{f_u}{\beta\sqrt{3}\gamma_{M2}}$$

Shear strength of the angle flange:

$$V_{Rd3} = bt\frac{f_y}{\sqrt{3}}$$

where t is the thickness of the angle flange and f_y its yield strength.

61.6. Support on stiffened bearing

If the value of the reaction to be transmitted is high, or if the beam and the support are not in the same plane, stiffened bearings may be used (Figure 61.6.a).

Where V_{Ed} is the reaction to be transmitted and d is the distance from it to the surface of the support. If the beam and the support are coplanar, it shall be assumed to pass through the end of the bearing.

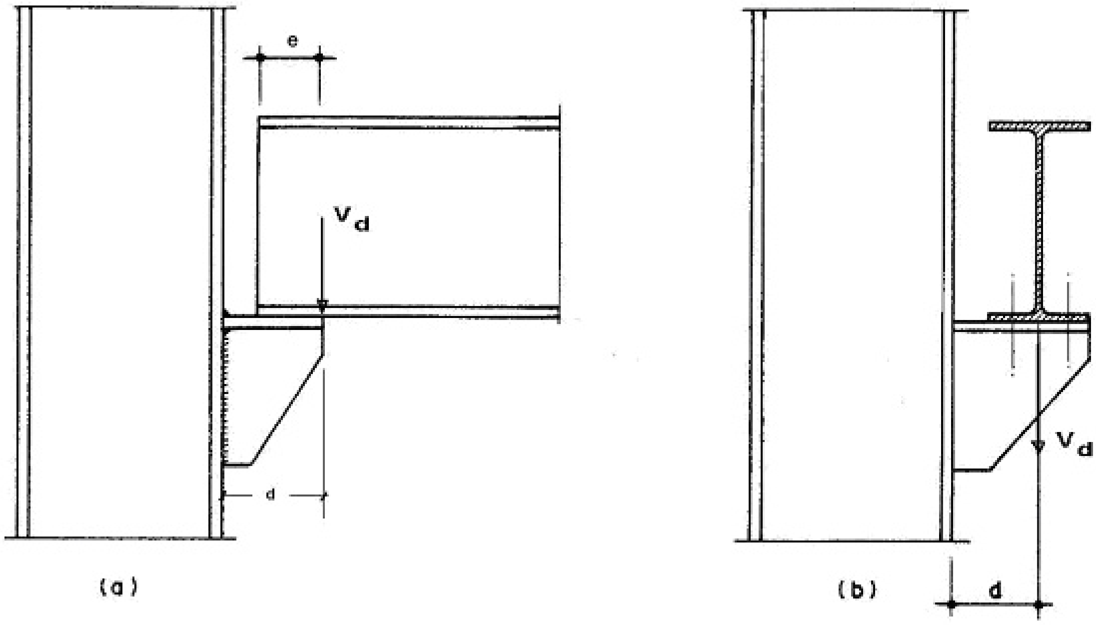


Figure 61.6.a

For the joint to be safe, the reaction to be transmitted V_{Ed} should be less than or equal to the smaller of the following values:

Local flattening resistance of the beam web:

$$V_{Rd1} = 5(t_r + r)t_w f_y$$

where t_f , t_w and r are the thicknesses of the flange and beam web and its flange-web fillet radius, and f_y its yield strength. if the distance e from the reaction crossing point to the end of the beam is less than $2.5(t_f + r)$, the value $2e$ shall be used in the above expression instead of $5(t_f + r)$.

Stiffener local buckling resistance:

$$V_{Rd2} = C_E \frac{M_{plRd}}{d}$$

in which:

$$M_{plRd} = \frac{tc^2 f_y}{4}$$

Plastic moment of the section of the stiffener with the notation indicated in Figure 61.6.b, of thickness t, effective depth c, measured perpendicular to its free edge, and yield strength f_y (see Figure 61.6 B).

$$C_E = 0{,}14\bar{\lambda}^2 - 1{,}07\bar{\lambda} + 2{,}3$$

Square coefficient.

$$\bar{\lambda} = 0{,}805 \frac{c}{t}\sqrt{\frac{f_y}{E}}$$

Non-dimensional slenderness of the stiffener.

- Resistance of weld 1 (Figure 61.6.b):

$$V_{Rd,w1} = \frac{2a_1 Lsen\theta f_u}{\beta_w \sqrt{(2+3\tan^2\theta)}\gamma_{M2}}$$

- Resistance of weld 2 (Figure 61.6.b):

$$V_{Rd,w2} = \frac{2a_2 \cos\theta f_u}{\beta_w \sqrt{(3+2\tan^2\theta)}\gamma_{M2}}$$

Resistance of weld 3 (Figure 61.6.b):

$$V_{Rd,w3} = \frac{\sqrt{2} b a_3 f_u}{\beta_w \tan\theta \gamma_{M2}}$$

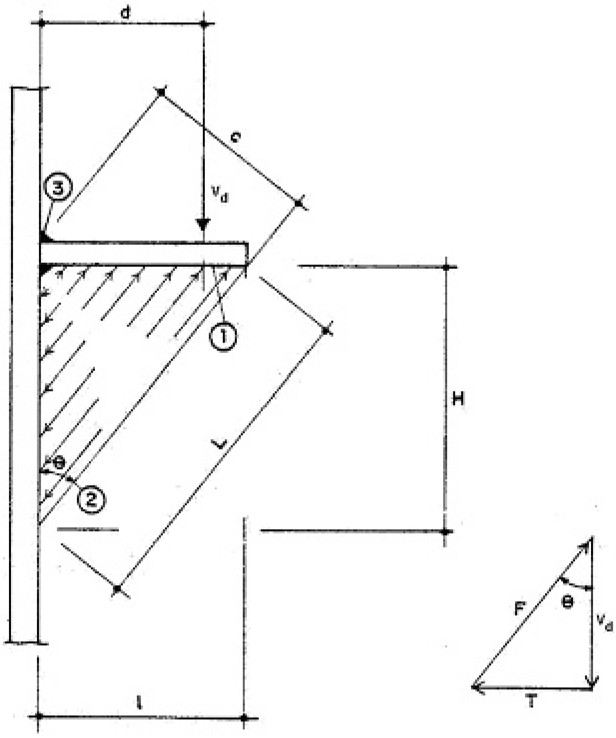


Figure 61.6.b

Section 62. Beam-to-column joints

Joints between rolled or trussed I-beams and I- or H-section column may be made by welding the flanges of the beam directly to the flanges of the column, or by means of end plates welded to the beam and bolted to the flanges of the column.

This section does not expressly cover joints in which the beams press the web of the column or with column of sections other than those indicated, which must be resolved by applying the general principles set out in the sections above and in Section 64 if the column is of hollow section.

62.1. Welded beam-to-column joints

62.1.1. Effective width of flange and weld

If the beam is welded to a column that does not have stiffeners extending the flanges of the beam (Figure 62.1.1) it shall be assumed that the effective width b_{ef} of the beam flange and the effective length of the weld chords joining the tension flange of the beam to the flange of the column shall be equal to the smaller of the following values:

$$b_b,\ b_c,\ t_{wc} + 2r_c + 7\frac{f_{yc}}{f_{yb}}\frac{t_{fc}^{\ 2}}{t_{fb}} \text{ or } t_{wc} + 2r_c + 7t_{fc}$$

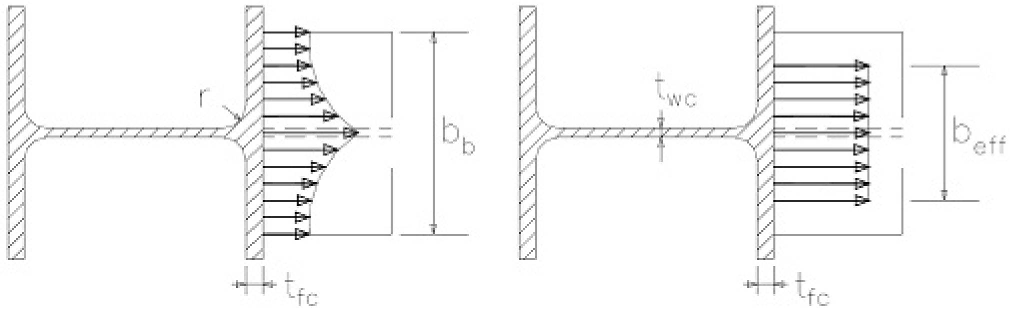


Figure 62.1.1

In the expressions above, b_b and b_c are the widths of the flanges of the beam and the column, r_c the flange-web fillet radius of the column, assuming that this is rolled, t_{fc} and t_{fb} are the thicknesses of the flange of the column and the flange of the beam, and f_{yc} and f_{yb} are the yield strengths of the steels in the flange of the column and the flange of the beam respectively.

If the column is plate-reinforced, r_c shall be replaced by $\sqrt{2}\, a_c$ in the expressions above, where a_c is the throat thickness of the chords of the flange-web joint of the column.

If $b_{ef} \leq b_b \dfrac{f_{yb}}{f_{ub}}$, a pair of stiffeners should be fitted to the column, extending the flange of the beam and having a combined section equal to or greater than that of said flange.

The welds of the joint between the flanges of the beam and that of the column shall be designed such that they are able to withstand the forces determined in accordance with the provisions of section 56.1, although it is recommended that they be able to withstand, along with the flange of the beam, $f_{yb}t_{fb}b_b\ /\gamma_{M0}$, for which it is sufficient to choose a_b in accordance with the provisions of section 59.8.

To check the strength of the joint between the tension flange of the beam and the flange of the column, if this has not been stiffened, the length of the chords shall be the effective width of the flange b_{ef}, instead of the real width, which may be greater.

The welds of the join between the web of the beam and the flange of the column shall be dimensioned to withstand the full shearing force V_{Ed} and part of the bending moment M_{Ed} not resisted by the flanges.

62.1.2. Support resistance. Unstiffened tension and compression zones

If stiffeners are not used to extend the flanges of the beam, the maximum tensile force that the tension zone can withstand (Figure 62.1.2.a) is the smaller of the resistance of the flange and web of the column, as given below:

- Column flange resistance: $F_{t,fc,Rd} = \dfrac{f_{yb} t_{fb} b_{ef}}{\gamma_{M0}}$

where the meaning of the variables is as explained in section 62.1.1.

- Column web resistance: $F_{t,wc,Rd} = \dfrac{\omega f_{yc} t_{wc} h_{ef}}{\gamma_{M0}}$

where f_{yc} is the yield strength of the steel of the web of the column and t_{wc} is its thickness; γ_{M0} is the strength reduction coefficient and h_{ef} is the effective height of the web, which is given by:

$h_{ef}=t_{fb}+2\sqrt{2}\, a_b + 5(t_{fc} + r_c)$ for rolled columns and by:

$h_{ef}=t_{fb}+2\sqrt{2}\, a_b + 5(t_{fc} + r_c)$ for trussed columns.

In the above expressions, a_b and a_c are the throat thicknesses of the weld chords of the joint of the flange of the beam to the flange of the column and of the flange-web joint of the column respectively. The meaning of the remaining symbols is the same as in previous sections.

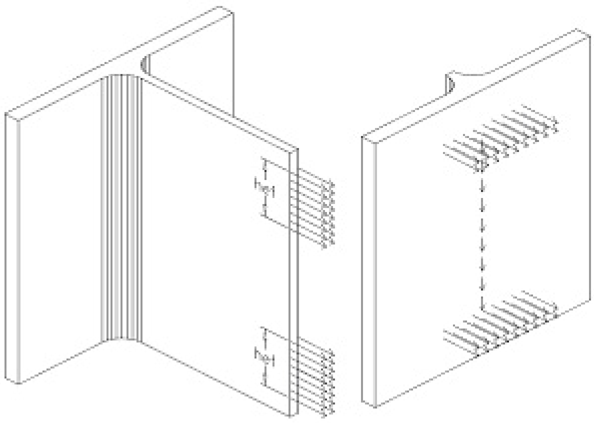


Figure 62.1.2.a

The maximum compression force that can be withstood by the compression zone is given by the compression strength of the web of the column. If it is not stiffened, it is given by:

$$F_{c,wc,Rd} = \frac{\omega k_{wc} \rho f_{yc} t_{wc} h_{ef}}{\gamma_{M1}}$$

In the expressions above, ω is a factor that considers the interaction with the shear force in the column web. Its value depends on the ratio $M_{b1,Ed}/M_{b2,Ed}$ between the moments on either side of the column. If there are moments equal in value and sign on both sides (as per Figure 62.1.2.b) then ω = 1.0. If there are moments of the same sign but of different values, or if one of the two is null, the following shall apply:

$$\omega = \frac{1}{\sqrt{1+1{,}3\left(\frac{h_{ef}t_{wc}}{A_{VC}}\right)^2}}$$

where h_c is the depth of the column and A_{vc} is the shear area of the column. If the moments on either side of the column are of different signs, the following shall apply:

$$\omega = \frac{1}{\sqrt{1+5{,}2\left(\frac{h_{ef}t_{wc}}{A_{VC}}\right)^2}}$$

The factor k_{wc} considers the influence of the maximum compression force σ_{nEd} existing in the web of the column, generated by the axial force and design bending moment to which the column is subjected in its joint to the beam. Its value is given by:

$$k_{wc} = 1{,}70 - \frac{\sigma_{n.Ed}}{f_{yc}}$$

If $\sigma_{nEd} \leq 0.7 f_{yc}$, then $k_{wc} = 1.0$

The factor ρ considers the potential local buckling of the web of the column. In which:

$\overline{\lambda}_p = 0{,}932\sqrt{\frac{h_{ef}h_1 f_{yc}}{E t_{wc}^2}}$ Non-dimensional slenderness of the column web

h_1 Height of its straight part.

f_{yc} Yield strength.

ρ is:

$\rho=1$ if $\overline{\lambda}_p \leq 0.72$

$\rho = \frac{\overline{\lambda}_p - 0{,}22}{\overline{\lambda}_p^2}$ if $\overline{\lambda}_p > 0.72$

The other symbols have the same meaning as above.

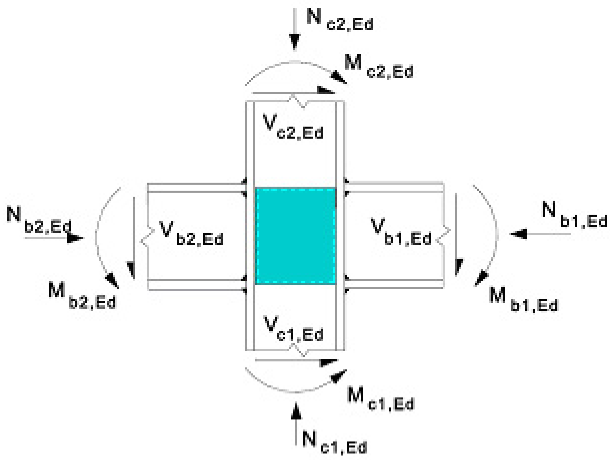


Figure 62.1.2.b

62.1.3. Column t resistance, stiffened tension and compression zones

The resistance of the tension and compression zones of a column t that have been stiffened with stiffeners shall be equal to the resistance of the beam flange if the stiffeners satisfy the following conditions:

The area of the pair of stiffeners of a zone, which may include the portion of the web of the support between the two stiffeners of the pair, A_r, should not be less than that of the beam flange A_{fb}, $A_r \geq A_{fb}$.

- If the steel used to make the stiffeners has a lower yield strength than the steel in the beam, it will be necessary to check its capacity to withstand the forces applied.
- The welds between the stiffener and the flange of the support should be dimensioned to withstand the forces transmitted by the beam flange.
- The welds between the stiffener and the web of the support must be dimensioned to withstand the forces that need to be transmitted to said web, which shall be $F_{Ed} - F_{Rd}$, whose values are defined in the previous section. Under no circumstances shall the throat thickness of the chords be less than the smaller of $0.4t_{wc}$ or $0.4t_r$, where t_r is the thickness of the stiffeners.

62.1.4. Column t resistance. Shear zone

The design methods given in this section shall apply if the slenderness of the column web satisfies the condition $d_{wc}/t_w \leq 69\varepsilon$, where d_{wc} is the depth of the column web.

For a single-sided joint, or for a double-sided joint where the depths of the beams are similar, the design plastic shear resistance $V_{wp,Rd}$ of an unstiffened column web subject to a design shear force $V_{wp,Ed}$ shall be obtained as follows:

$$V_{wp,Rd} = \frac{0{,}9 f_{y,wc} A_{vc}}{\sqrt{3}\gamma_{M0}}$$

where A_{vc} is the shear zone of the column.

The resulting shear force $V_{wp,Ed}$ in the web shall be obtained as follows:

$V_{wp,Ed} = (M_{b1,Ed} - M_{b2,Ed})/z - (V_{c1,Ed} - V_{c2,Ed})/2$

where z is the lever arm, $M_{b1,Ed}$ and $M_{b2,Ed}$ are the internal bending moments, and $V_{b1,Ed}$and $V_{b2,Ed}$ are the shear forces applied to the joints by the connected members (see Figure 62.1.2.b).

If the design shear force $V_{wp,Ed}$ is greater than the design shear plastic resistance $V_{wp,Rd}$ the web of the support must be reinforced, either using a pair of oblique stiffeners (Figure 62.1.4.a) or using web reinforcing plates (Figure 62.1.4.b).

If the column web is reinforced by adding a plate, the shear area A_{vc} may be increased to $b_s t_{wc}$. If another reinforcing plate is added to the other side of the web, the shear area may not be further increased.

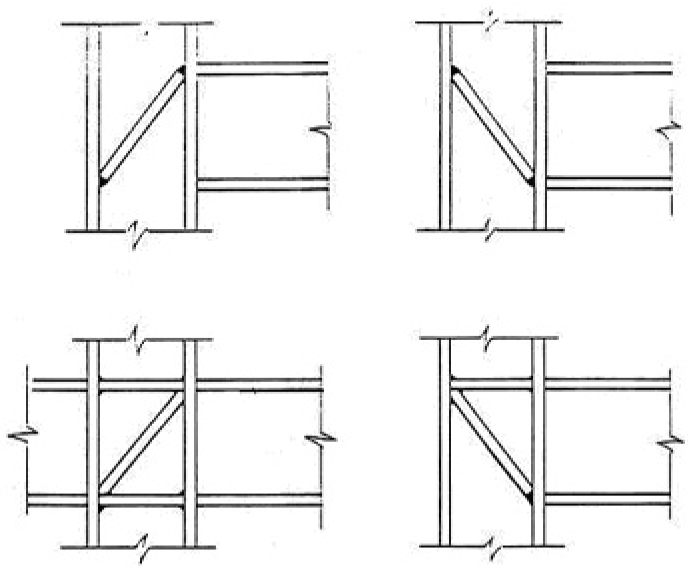


Figure 62.1.4.a

If it is reinforced with a pair of diagonal stiffeners of length *d* (Figure 62.1.4.a) the area of the cross-section of the pair of stiffeners must comply with the following:

$$A_d = \frac{d}{\sqrt{3}}\left(\frac{\sqrt{3}\gamma_{M0}(M_{b1,Ed} - M_{b2,Ed})}{f_y(h_c - 2t_{fc})(h_b - t_{fb})} - t_{wc}\right)$$

62.1.5. Moment resistance of the joint

The moment $M_{j,Rd}$ that the joint can resist is given by (Figure 62.1.5):

$$M_{j,Rd} = F_{Rd} z$$

where:

$z = h - t_{fb}$

h	Beam height
t_{fb}	Beam flange thickness
F_{Rd}	The smaller of the resistances of the tension zone and the compression zone

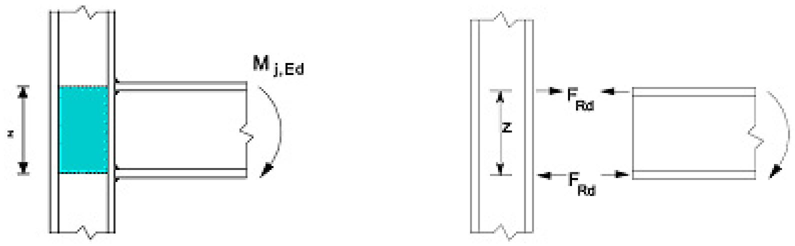


Figure 62.1.5

62.2. Bolted beam-to-column joints

Bolted beam-to-column joints are typically made using end plates. Their characteristics are similar to beam splices with end plates (section 61.2).

The thickness of the flange of the column is frequently less than the thickness of the end plate welded to the beam. On the other hand, the resistance and stiffness of said flange depend to a large extent on the presence or absence of stiffeners prolonging the flanges of the beam, on account of which this type of joint shall be checked in accordance with the following sections.

The design tensile strength of the bolts on the beam side shall be determined in accordance with the provisions of section 61.2 above. On the support side, the tensile strength of the bolts shall be determined similarly to the beam side, on an equivalent T-stub basis, considering the three possible failure modes, but using the effective lengths given in section 62.2.1.

The resistance of the tension, compression and shear zones on the support side shall be determined in accordance with sections 62.1.2, 62.1.3, and 62.1.4, however, in the case of a column web in tension, hef shall be taken as the effective length of the equivalent T-stub, which represents the flange of the column (see 62.2.1).

62.2.1. Support-side resistance

The design tensile strength of each row or group of bolts on the support side shall be determined in accordance with the failure modes set out in section 61.2, but using the effective widths set out in tables 62.2.1.a and b, depending on whether or not the column as transverse stiffeners. The dimensions m and e_{min} are defined in Figure 62.2.1.a and e_1 is the distance, measured in the direction of the column axis, from the upper bolt row to the end of the column.

Modelling of the stiffened flange of a column as T-stubs can be seen in Figure 62.2.1.b.

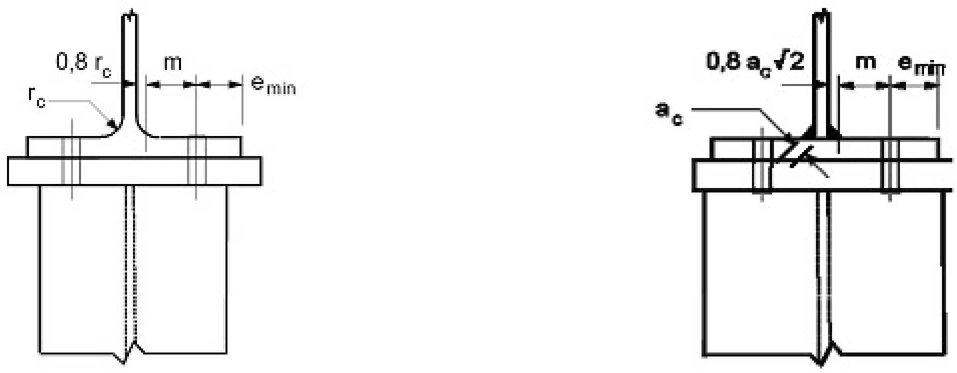
Table 62.2.1.a.Effective lengths for an unstiffened column flange

Location of bolt row	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: $2\pi m$ $\pi m + 2e1$	The smaller of: $4m + 1.25e$ $2m + 0.625e + e1$	The smaller of: $\pi m + p$ $2e1 + p$	The smaller of: $2m + 0.625e + 0.5p$ $e1 + 0.5p$
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:	$\ell_{eff,2} = \ell_{eff,nc}$		$\Sigma\ell_{eff,2} = \Sigma\ell_{eff,nc}$	

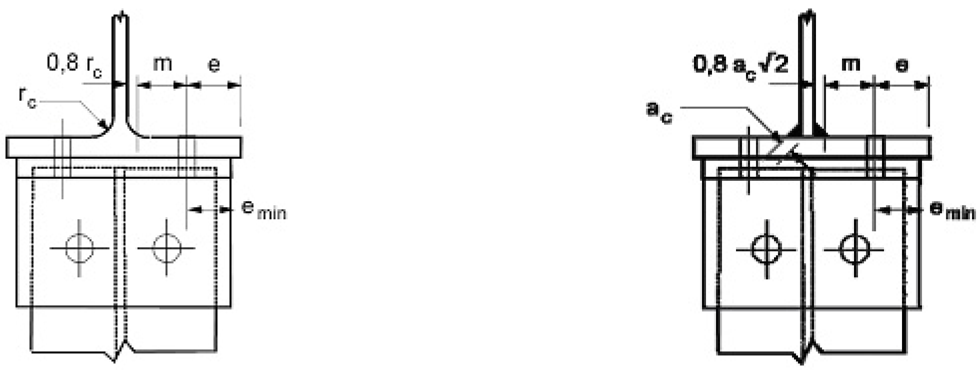
Table 62.2.1.b.Effective lengths for a stiffened column flange

Location of bolt-row	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 obtained from Figure 61.2.c.				

a) Welded end-plate narrower than column flange.



b) Welded end-plate wider than column flange.

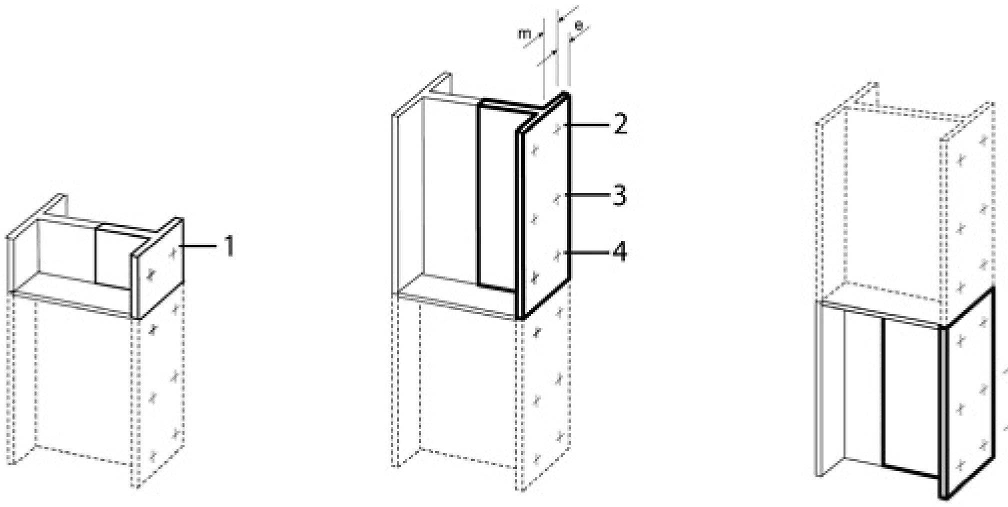


c) Angle flange cleats.

Figure 62.2.1.a Definitions of *e, e_{min}, r_c and m*

Once the above values have been determined, the value of F_{Rd} for each bolt row shall be taken as the lesser of those determined for the beam side and the support side.

The resistance of the joint $M_{j,Rd}$ shall be checked in accordance with the provisions of section 62.2.2.



1 End bolt row adjacent to a stiffener.

2 End bolt row.

3 Inner bolt row.

4 Bolt row adjacent to a stiffener.

Figure 62.2.1.b Modelling a stiffened column flange as separate T-stubs

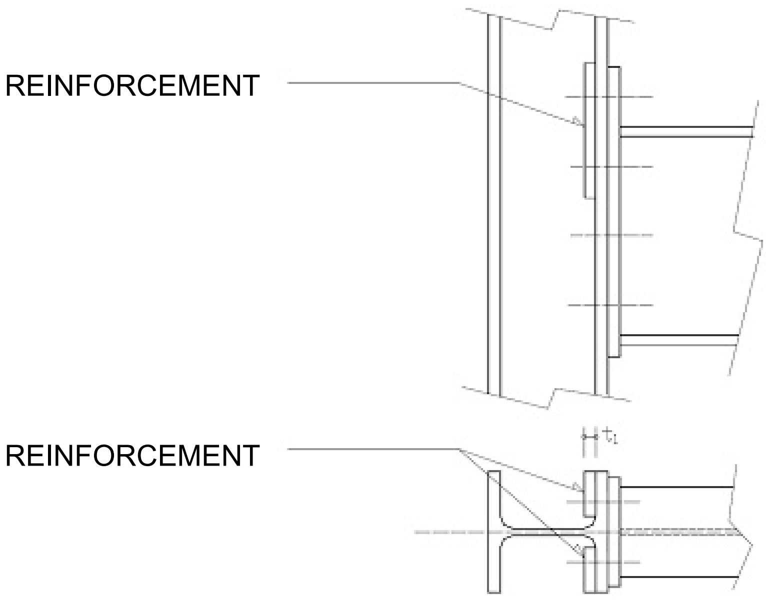


Figure 62.2.1.c

In certain cases in which it is not necessary to stiffen the support but the design ultimate tensile strength of the bolts on the support side is insufficient, it may be economical to strengthen its flange with two reinforcing plates, of thickness t_1, held only with the bolts of the beam joint (Figure 62.2.1.c).

If reinforcing plates are used, the design resistance of the T-stub $F_{T,Rd}$ shall be determined using the method given in Table 61.2.a.

62.2.2. Moment resistance of a joint

If a joint is subjected in general to an axial force N_{Ed}, positive for tension, a shear force V_{Ed} and a bending moment M_{Ed}, the joint shall be checked as follows:

- The value of $F_{T,Rd}$ shall be determined for each of the T-stubs in accordance with the provisions of section 61.2.

- The moment $M_{j,Rd}$ that the joint can resist is given by:

$$m_{j,Rd} = \sum (f_{tr,Rd,i} d_i)$$

Where di is defined in Figure 61.2.a.

- The following should be true for the joint to be safe $M_{Ed} \leq M_{j,Rd}$.

- The method given in this section to determine the design moment resistance of a joint $M_{j,Rd}$ does not consider any coexisting axial force N_{Ed} in the connected member. Consequently, they should not be used if the axial force in the connected member is greater than 5 % of the design plastic resistance $N_{p\ell,Rd}$ of its cross-section.

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

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

in which:

$M_{j,Rd}$ Design moment resistance of the joint, assuming that no axial forces are acting.

$N_{j,Rd}$ Design moment resistance of the joint, assuming that no applied moment exists.

- The effective design tension resistance $F_{tr,Rd}$ of a bolt row r, as an individual bolt row, shall be the smaller design ultimate tensile strength value for an individual bolt row of the following basic components:

  - Column web in tension $F_{t,wc,Rd}$ (defined in section 62.1.2).
  - Column flange in bending. $F_{T,fc,Rd}$ (defined in section 62.2.1) .
  - End plate in bending: $F_{T,ep,Rd}$ (defined in section 61.2).
  - Beam web in tension: $F_{t,wb,Rd}$ (defined in section 61.2.1).

- The design effective tensile resistance $F_{tr,Rd}$ of the bolt row *r* should , if necessary, be reduced to ensure that, when all of the aforementioned bolt rows including bolt row *r,* are taken into account, the following conditions are satisfied:

- The total design resistance $\Sigma_{Ft,Rd} \leq V_{wp,Rd} /\beta$ (with β as per point 62.3).

- The total design resistance $\Sigma F_{t,Rd}$ does not exceed the smaller of:

  - The design compression resistance of the column web $F_{c,wc,Rd}$ (see section 62.1.2).
  - The design compression resistance of the flange and web of the beam $F_{c,fb,Rd}$ (see section 61.2.1).

- The design effective tension resistance $F_{tr,Rd}$ of bolt row r should , if necessary, be reduced to ensure that the sum of the design strengths of the preceding bolt rows, including bolt row r which is part of the same group of bolt rows, does not exceed the design strength of said group as a whole. This must be checked for the following basic components:

- The column web in tension $F_{t,wc,Rd}$

- The column flange in bending $F_{T,fc,Rd}$

- The end plate in bending $F_{T,ep,Rd}$

- The beam web in tension $F_{t,wb,Rd}$

- It shall be assumed that the shear force V_{Ed} acting on the joint is resisted by the bolts working as part of the corresponding joint category: A, B or C.

62.3 Joint stiffness

It shall be necessary to determine the stiffness Sj of the joint as indicated below, and it shall be checked, in accordance with the provisions of 57.4, that the hypothesis that it is rigid is correct. If the hypothesis is incorrect, it shall be necessary to predetermine the forces in consideration of the stiffness Sj of the joint.

The initial rotationl stiffness of a beam-to-column joint is given by the following expression, provided that the axial force N_{Ed} in the connected element does not exceed 5 % of the design resistance $N_{p\ell,Rd}$ of its cross-section:

$$S_{j,ini} = \frac{E_z^2}{\sum \frac{1}{k_i}}$$

where:

z Joint lever arm. For welded joints, z shall be as set out in section 62.1.5. For bolted joints, z shall be z_{eq} as defined below. Approximately, for joints with bolted extended end-plates, z may be taken as the distance from the centre of compression to the intermediate point between the two further bolt rows in tension.

k_i Stiffness of each basic component, as defined below:

a) Shear stiffness of the column web:

$$k_{wv} = 0{,}38 \frac{A_{vc}}{\beta z}$$

If the column web is stiffened with oblique stiffeners $k_{wv} = \infty$

b) Tension stiffness of the column web:

$$k_{wt} = 0{,}7 \frac{h_{ef} t_{wc}}{d_c}$$

If the column web is stiffened $k_{wt} = \infty$

c) Compression stiffness of the column web:

$$k_{wc} = 0{,}7 \frac{h_{ef} t_{wc}}{d_c}$$

If the column web is stiffened $k_{wc} = \infty$

d) Bending stiffness of the column flange, corresponding to a bolt row:

$$k_f = 0{,}9 \frac{b_{ef,f} t_{fc}^3}{m^3}$$

e) Bending stiffness of the end-plate, corresponding to a bolt row:

$$k_p = 0{,}9 \frac{b_{ef,p} t_p^3}{m^3}$$

f) Tension stiffness of a bolt row:

$$k_b = 1{,}6 \frac{A_S}{L_b}$$

In all of the above:

A_{vc} Column shear area.

β Transformation parameter, function of the beam moments (see Figure 62.1.2.b).

For the right joint: $\beta_1 = |1-(M_{b2,Ed} / M_{b1,Ed})|) \leq 2$

For the left joint: $\beta_2 = |1-(M_{b1,Ed} / M_{b2,Ed})|) \leq 2$

h_{ef} Effective width of the column web. Defined in section 62.1.2 for the column web in compression and web in tension in welded joints and in section 62.2.1 for the column web in tension in bolted joints.

t_{wc} and t_{fc} Thickness of column web and flange respectively.

d_c Straight part of the column web, $d_c = h_c – 2(t_{fc} – r_c)$

t_p Thickness of the end-plate.

A_s Resistance area of bolts in tension (see Table 58.7).

L_b Bolt grip length, distance from the middle of the thickness of the nut to the middle of the thickness of the head.

$b_{ef,f}$ Effective width of the column flange in bending. This shall be the smaller of the effective lengths (individually or as part of a bolt group) for the bolt row given in section 62.2.1 for unstiffened and stiffened flanges.

$b_{ef,p}$ Effective width of the end plate in bending. The smaller of the effective lengths (individually or as part of a bolt group) given for the bolt row in section 61.2 shall be taken.

m Defined in general in Figure 61.2.a and 61.2.b, but for a bolt row located in the extended portion of an extended end plate $m=m_x$, where m_x is as defined in Figure 61.2.b.

For joints with end-plates 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 ratio k_{ea} determined on the basis of:

$$k_{eq} = \frac{\sum_r k_{eff,r} h_r}{z_{eq}} \qquad \text{with} \qquad k_{eff,r} = \frac{1}{\sum_i \frac{1}{k_{i,r}}}$$

where:

h_r Distance between bolt row r and the centre of compression.

$k_{eff,r}$ Effective stiffness ratio for bolt row r in consideration of the stiffness ratios $k_{i,r}$ for the basic components i (for beam-column joints, these are the column web in tension, the column flange in bending, the end-plate in bending and the bolts in tension, and for beam splices with bolted end plate, these are the end plates in bending and the bolts in tension).

z_{eq} Equivalent lever arm, with the value:

$$z_{eq} = \frac{\sum_r k_{eff,r} h_r^2}{\sum_r k_{eff,r} h_r}$$

As a simplification, in a bolted joint with end plate with more than one bolt row in tension, the contribution of any bolt row may be disregarded, provided that the contributions of the remaining bolt rows closest to the centre of compression are also disregarded. The number of bolt rows considered does not necessarily have to be the same as when determining the design moment resistance.

62.4. Rotation capacity of a joint

62.4.1. General

If the global analysis of the structure is carried on using plastic methods and a plastic hinge is expected to form in the joint, the joint should have sufficient rotation capacity.

The clauses of sections 64.4.2 and 64.4.3 are only valid for steels S235, S275 and S355 and for joints in which design value of the axial force N_{Ed} in the member connected does not exceed 5 % of the design plastic resistance $N_{pl,Rd}$ of its cross-section.

The rotation capacity of joint need not be checked, provided that the design moment resistance of the joint $M_{j,Rd}$ is at least 1.2 times the plastic moment resistance $M_{pl,Rd}$ of the cross-section of connected member.

62.4.2 Welded joints

A welded beam-support joint, designed in accordance with the provisions of the above sections, unstiffened, has a rotation capacity $\Phi_{Cd} = 0.015$ *rad*.

A full-strength welded beam-column joint has sufficient rotation capacity to enable analysis of the structure using the plastic method.

A welded beam-column joint in which the moment it can resist is limited by the resistance of the shear zone has sufficient rotation capacity to enable analysis of the structure using the plastic method.

A welded beam-column joint with stiffened tension zone has sufficient rotation capacity to enable analysis of the structure using the plastic method, although it is not full strength.

In a welded beam-column joint with stiffened compression zone (but not tension zone) and in which the moment it can resist is not limited by the resistance of the shear zone, the rotation capacity shall be taken as:

$$\phi_{Cd} = 0,025\frac{h_c}{h_b}$$

62.4.3. Bolted joints

A beam-to-column joint in which the design moment resistance of the joint 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 $d_{wc} / t_w \leq 69\ \varepsilon$, where d_{wc} is the depth of the column web.

A joint with either a bolted-end plate or flange angle cleat connection may be assumed to have sufficient rotation capacity for plastic analysis provided that both of the following conditions are satisfied

a) The design moment resistance of the joint is governed by either on the resistance of the column flange in bending, or the resistance of the end-plate or angle ~~iron~~ of the flange in bending, i.e. in determining the design tension resistance F_{Rd} of all of the bolts in tension, failure mode 1 is key, either on the beam side or on the support side.

b) The thickness t of either the column flange,or the beam end-plate or tension flange cleat satisfies:

$$t \leq 0,36d\sqrt{f_{ub} / f_y}$$

A bolted joint whose design moment resistance $M_{j,Rd}$ depends on the design resistance of its bolts in shear should not be deemed to have sufficient rotation capacity for a plastic global analysis.

Section 63. Hybrid connections with bolts and welds

63.1. Bolt types

If, in a single joint between two parts, weld chords and bolts are required to help transmit the shear force, the connecting means that provides the greatest stiffness should be designed to withstand the entire load.

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

The strength of the connection shall be the sum of the resistance provided by the weld and the bolts, calculated in accordance with the provisions of sections 59.8, 59.9 and 58.8 respectively.

63.2. Making the connection

High-strength bolts are not tightened until welding is complete to prevent them being affected by residual welding stresses.

63.3. Strengthening

If it is necessary to strengthen an existing bolted or riveted connection, weld chords may be used, provided that they are calculated to be able to resist all of the forces generated by the permanent loads that the strengthening itself introduces, and all of the overloads that can act once the strengthening has been effected.

The forces generated by the permanent loads existing before the strengthening was done must be resisted by existing connection means.

Section 64. Hollow section joints

64.1. Scope

This section sets out the application rules to calculate the static resistance of uniplanar and multiplanar joints made between hollow, circular or rectangular section bars or between bars of this type and H- or U-section hot-rolled bars that in general are part of structures or lattice members or Vierendeel, flat or spatial.

For fatigue assessment see the provisions of Chapter XI Fatigue of this Code.

These application rules are valid both for hot-finished hollow sections to UNE-EN 10210 and for cold-formed hollow sections to UNE-EN 10219.

For parts manufactured with S420 or S460 steel, the resistance calculated in accordance with the rules set out in the sections below must be reduced by multiplying it by a factor of 0.9.

The minimun value thickness of hollow sections should not be less than 2.5 mm. If the thickness of the node zone is greater than 25.0 mm, the steel of the chord in this zone must be type Z.

The value $\gamma_{M5} = 1.00$ shall be taken for the partial safety coefficient γ_{M5} in this type of joint.

The angle θ between adjacent bars should satisfy the condition $\theta \geq 30º$.

Given that the expressions providing the resistance of these joints has been calculated primarily on the basis of analysis of a wide base of experimental results, its scope is strictly limited to that indicated in this section, and they may not be extrapolated to arrangements or dimensions not expressly admitted therein, since such extrapolation may give unsafe results.

64.2 Definitions and terms

The types of joints covered are indicated in fig. 64.2.a, although it is preferable to use those listed first:

- Direct joints by welding between hollow sections or between hollow sections and hot-rolled H- or U-bars, without modifying the cross-section of any bar.

- Direct joints by welding between hollow sections or between hollow sections and hot-rolled H- or U-bars, modifying the cross-section of one of the circular hollow bars in the joint by flattening.

- Joints with welded blocks, without modifying the cross-section of any of the bars involved.

The following shall apply in particular to direct joints:

CHS , circular hollow section bar.

RHS , rectangular or square hollow section bar.

Chord, main bar that neither starts nor ends in the node in which the joint is made, but continues through it. It is expressly forbidden to perforate it to enable other bars can penetrate its interior, although this may be done with node blocks, if they are used.

Diagonal, secondary bar that starts or ends in the node, and that forms an angle $\theta \neq 90º$ with the corresponding chord.

Upright, secondary bar that starts or ends in the node, and that forms an angle $\theta = 90º$ with the corresponding chord.

Flat joint, joint in which the axes of all of the bars involved in the joint are located in the same plane.

Spatial joint, joint in which the axes of all of the bars involved in the joint are not located in the same plane.

T joint, flat joint between a chord and an upright, in which the forces in the upright are balanced by the shear and bending forces in the chord (Figure 64.2.a).

Y joint, flat joint between a chord and a diagonal, in which the forces in it are balanced by the shear and bending forces in the chord (Figure 64.2.a).

X joint, flat joint between a chord and two diagonals or two uprights arranged in extension, in which the forces in the diagonals or uprights balance each other, passing through the chord (Figure 64.2.a).

K joint, flat joint between a chord and two diagonals, in which the forces in it are balanced by axial forces in the chord (Figure 64.2.a). In general, this applies to Warren trusses. They may be gap or overlap joints.

N joint, flat joint between a chord and brace member, in which the forces in the brace member are balanced by axial forces in the chord (Figure 64.2.a). In general, this applies to Howe or Pratt trusses. They may be gap or lap joints.

KT joint, flat joint between a chord, two diagonals and one upright, in which the forces in the diagonals and the upright are balanced by axial forces in the chord (Figure 64.2.a). In general, this applies to Warren trusses with uprights. They may be gap or lap joints.

DK joint, flat joint between one chord and two pairs of diagonals that press the chord symmetrically in respect of its axis (Figure 64.2.a). The force in each diagonal is balanced by the force in the extended diagonal, passing through the chord. They may be gap or lap joints.

DY joint, flat joint between one chord and two diagonals that press the chord symmetrically in respect of its axis (Figure 64.2.a). Axial forces in the chord balance the forces in the diagonals.

TT joint, spatial joint between a chord and two uprights. The plane defined by the two uprights is perpendicular to the chord (Figure 64.2.a). The force in the uprights is balanced by shear and bending forces in the chord.

XX joint, spatial joint between a chord and two pairs of uprights. Within each pair, the two uprights are in extension. The planes defined by the chord and each of the two pairs of uprights is perpendicular to each other (Figure 64.2.a). The force in the extended diagonal, passing through the chord, balances the force in each upright.

KK joint, spatial joint between a chord and two pairs of diagonals. The planes defined by the chord and each of the two pairs of […] form a specific angle with each other (Figure 64.2.a). The force in the diagonals is balanced by shear and bending forces in the chord.

Toe and heel, vertices of the most obtuse and most acute angles respectively in the joint between a diagonal or upright and the chord.

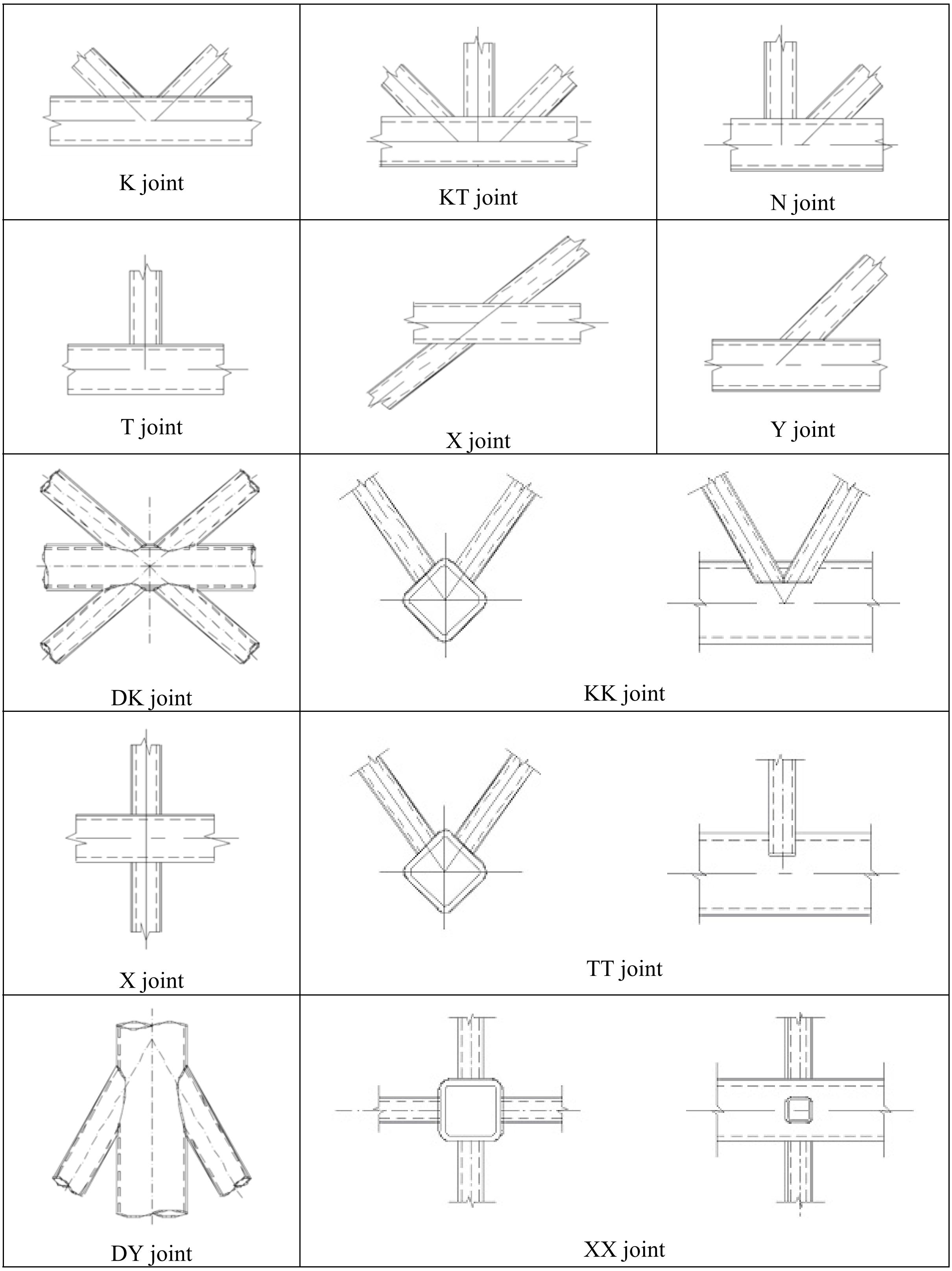


Figure 64.2.a Types of joints in hollow section lattice girders

Joint with gap, K, N or similar joint in which the feet of two contiguous diagonals or uprights do not touch (Figure 64.2.b).

Joint with overlap, K, N or similar joint in which the feet of two contiguous diagonals or uprights penetrate one another (Figure 64.2.c).The bar with the highest product t_if_y or that is the widest (overlapped bar) must continue to the chord; the other bar (overlapping bar) must cross the overlapped bar and the chord.

d, diameter of a bar (figure 64.2.b).

b, *h*, Transversal dimensions of a bar, perpendicular to the plane of the joint and in the plane of the joint respectively (Figure 64.2.c).

t, thickness of a bar.

Subscripts:

0, relating to the chord.

i, relating to diagonals or uprights (I = 1, 2 or 3).

f, relating to a beam.

p, relating to a plate.

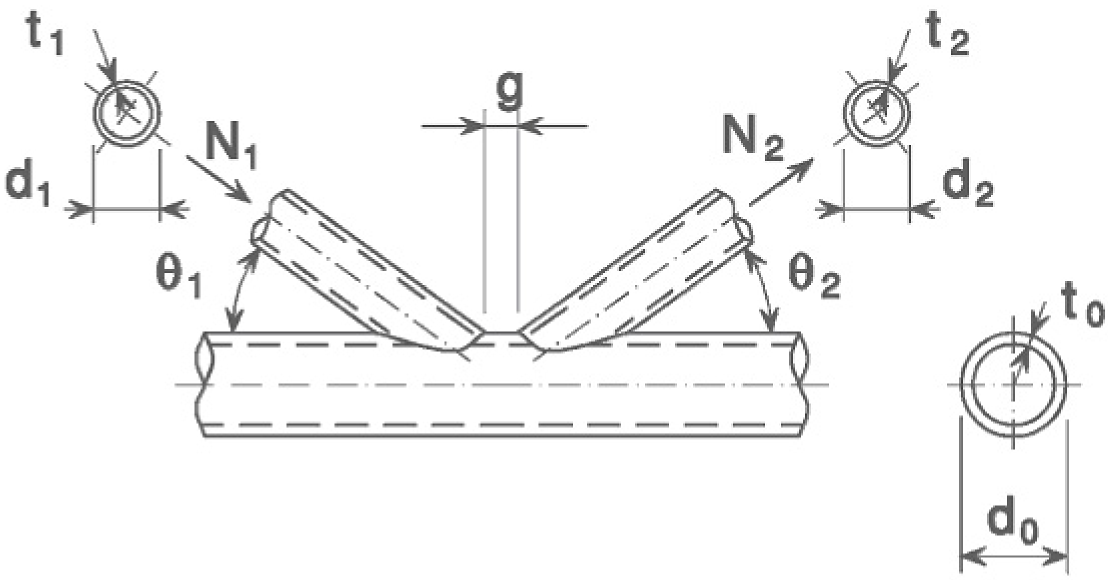


Figure 64.2.b

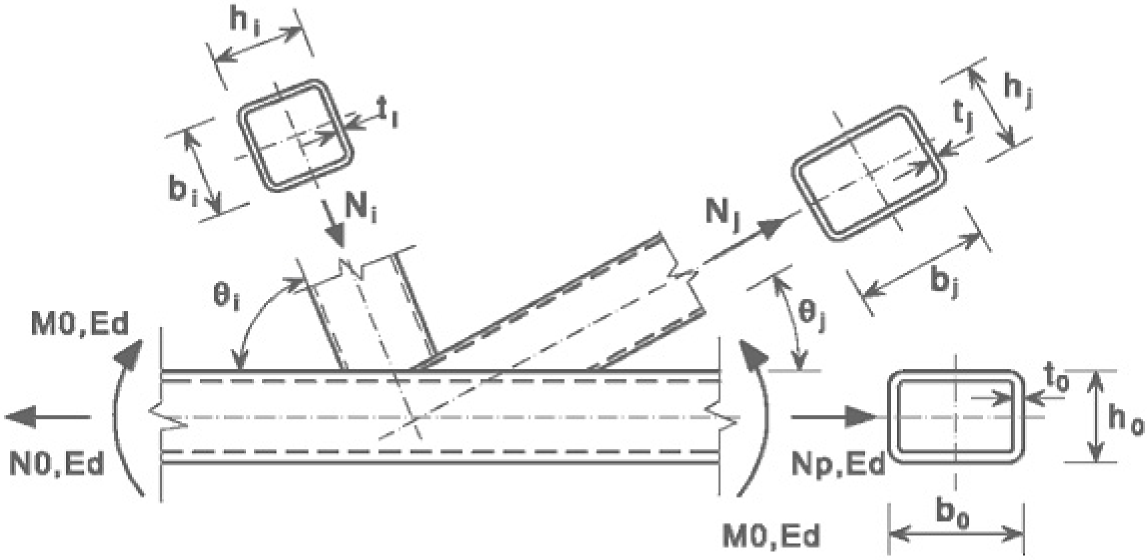


Figure 64.2.c

g, gap between the feet of two contiguous brace members in K, N or similar joints, measured along the chord face in the plane of the joint, regardless of the thickness of the weld (Figure 64.2.d).The following shall be true in all cases:

$g \geq t_1 + t_2$

where t_1 and t_2 are the thicknesses of adjacent brace members.

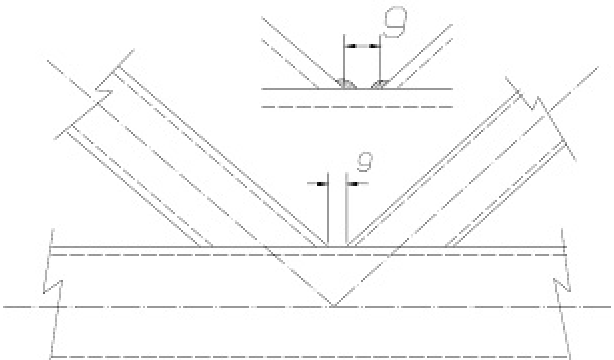


Figure 64.2.d

Gap joints

p, in K, N or similar joints, length of the intersection of the overlapped member with the chord, measured along the face of the chord in the plane of the joint (Figure 64.2.e).

q, in K, N or similar joints, theoretical overlap length, measured along the face of the chord in the plane of the joint (Figure 64.2.e).

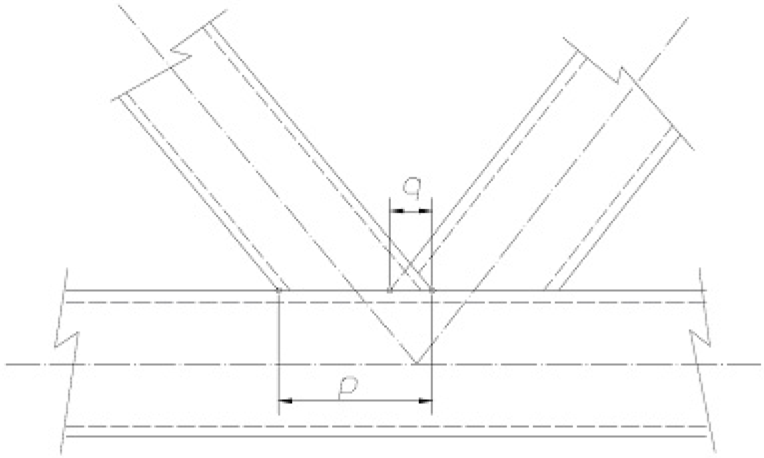
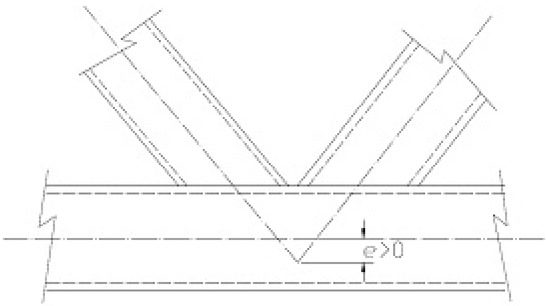


Figure 64.2.e

e, Eccentricity in the node, positive if the crossing point of the braces is on a different side, in consideration of the axis of the chord, to the diagonals brace members themselves, or negative if this is not the case (Figures 64.2.f.a and 64.2.f.b).



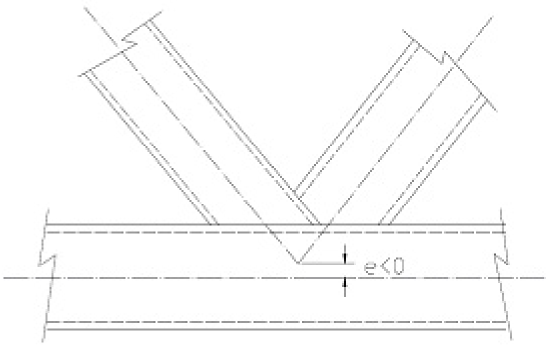


Figure 64.2.f.a

Figure 64.2.f.b

When checking chords subjects to compression, it is advisable to take account of any moments arising from the existence of eccentricities. If the chord is in tension and the eccentricity is within the following range:

$$-0{,}55 \leq \frac{e}{d_0} \leq 0{,}25 \quad \text{or} \quad -0{,}55 \leq \frac{e}{h_0} \leq 0{,}25$$

The resulting bending moments may be ignored in node and chord calculations.

β, ratio of dimensions between a brace and the corresponding chord. The following values shall be taken as a function of joint type:

For T, Y or X joints:

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

For K or N joints:

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

For KT joints:

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

γ, ratio between the dimension of a chord and twice its thickness. Depending on the bar type, this shall be:

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

η, ratio between the height of a brace member and the diameter or width of the corresponding chord:

$$\eta = \frac{h_i}{d_0} \text{ or } \eta = \frac{h_i}{b_0}$$

λ_{ov}, overlap ratio as a percentage:

$\lambda_{ov} = 100\frac{q}{p}$ (see Figure 64.2.e)

$\lambda_{ov,lim}$ limit or critical overlap ratio.

To ensure the proper transfer of forces between the two bars, the following condition should be satisfied:

$\lambda_{ov} \geq 25$

$N_{0,Ed}$, $M_{0,ed}$, design forces (axial and bending, respectively) in the chord.

$N_{i,Ed}$, design axial force in the brace member i.

A_0, $W_{el,0}$ section and elastic modulus of the chord.

$$N_{p,Ed} = N_{0,Ed} - \sum_{i>0} N_{i,Ed} \cos\theta_i$$

$N_{p,Ed}$, is the value of the axial force disregarding the members of the diagonals or uprights parallel to the axis of the chord.

$$\sigma_{0,Ed} = \frac{N_{o,Ed}}{A_0} + \frac{M_{0,Ed}}{W_{el,0}}$$

$$\sigma_{p,Ed} = \frac{N_{p,Ed}}{A_0} + \frac{M_{0,Ed}}{W_{el,0}}$$

$$n = \frac{\sigma_{0,Ed}}{f_{y,0}}$$ (in chords of rectangular or square section);

$$n_p = \frac{\sigma_{p,Ed}}{f_{y,0}}$$ (in chords of circular section).

64.3. Welds

Direct joints between bars shall be made by welding. Joints between blocks and chords and joints between braces and blocks shall also be welded.

In direct joints between bars, the weld shall be made along the entire contact perimeter of the diagonal or upright with the chord. Exceptionally, in joints with overlap, the overlapped part need not be welded, providing that the members perpendicular to the chord of the axial forces in the other two bars in the node do not differ from one another by more than 20 % of the larger of said members. If this is not the case, the most stressed bar shall be the passing bar, and it shall be welded in full to the chord before being overlapped by the other diagonal or upright.

Said weld may be a butt weld with partial penetration or a fillet weld or a combination of these. In any case, the resistance of the weld must not be less than that of the bar, diagonal or upright, connected to the chord or to another brace. In specific lightly stressed zones, a lower weld resistance may be permitted provided that the Designer explicitly demonstrates that the welds are sufficiently resistant, both to enable non-uniform distribution of stresses and to ensure the deformation capacity necessary to enable the redistribution of secondary bending.

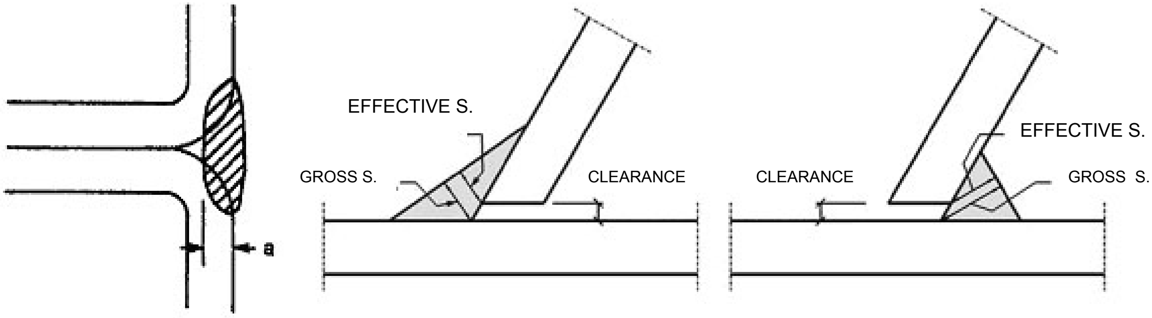
This condition shall be assumed to be satisfied if the weld is full penetration or if its throat “a” is at least equal to the thickness t_i of the bar to be joined. For this purpose, “A” shall be the effective throat determined by discounting the throat thickness provided, which depends on the clearance between the diagonal or upright and the chord, as shown in Figure 64.3.a.In any case, the clearance must always be less than 3 mm, as shown in the Figures 64.3.b and 64.3.c.

In the case of fillet welds, or partial penetration welds, the project execution monitoring plan shall include approval of the welding procedures, as per UNE-EN ISO 15614-1 or UNE-EN ISO 15613, for all of the joint types required for the different geometry of diameters, thicknesses, number and angles of members in the node, degrees of gap or overlap, etc. Said procedures must explicitly define the following in particular:

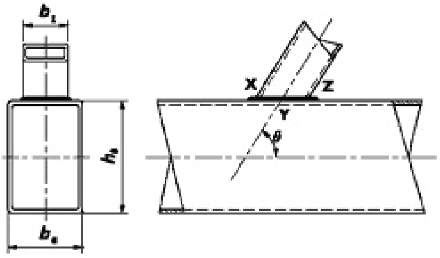
- Accreditation of the maximum clearances throughout the perimeter of the joint.
- The start and end points of each pass and the number of passes required to guarantee the design thickness of the effective throat of the fillet weld along the entire perimeter of the joint, which requires a variable number of passes between the toe, heel and side zones of the joint.
- Accreditation of the external geometry of the joint, once finished, to guarantee that the design thickness of the effective throat has been achieved throughout the entire perimeter.

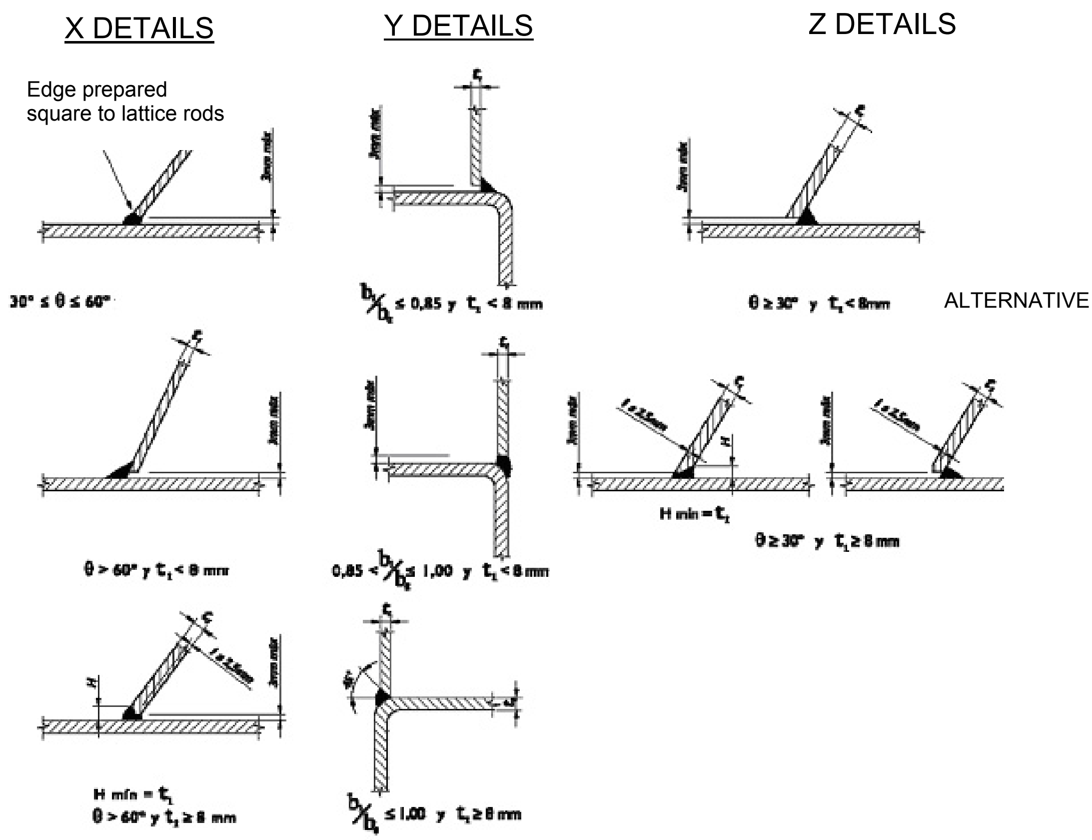
The throat between curved edges is defined in Figure 64.3.a.

For rectangular structural hollow the design throat thickness is defined in figures 64.3.b and 64.3.c and circular sections respectively. More detailed recommendations are given in standard EN 1090-2.



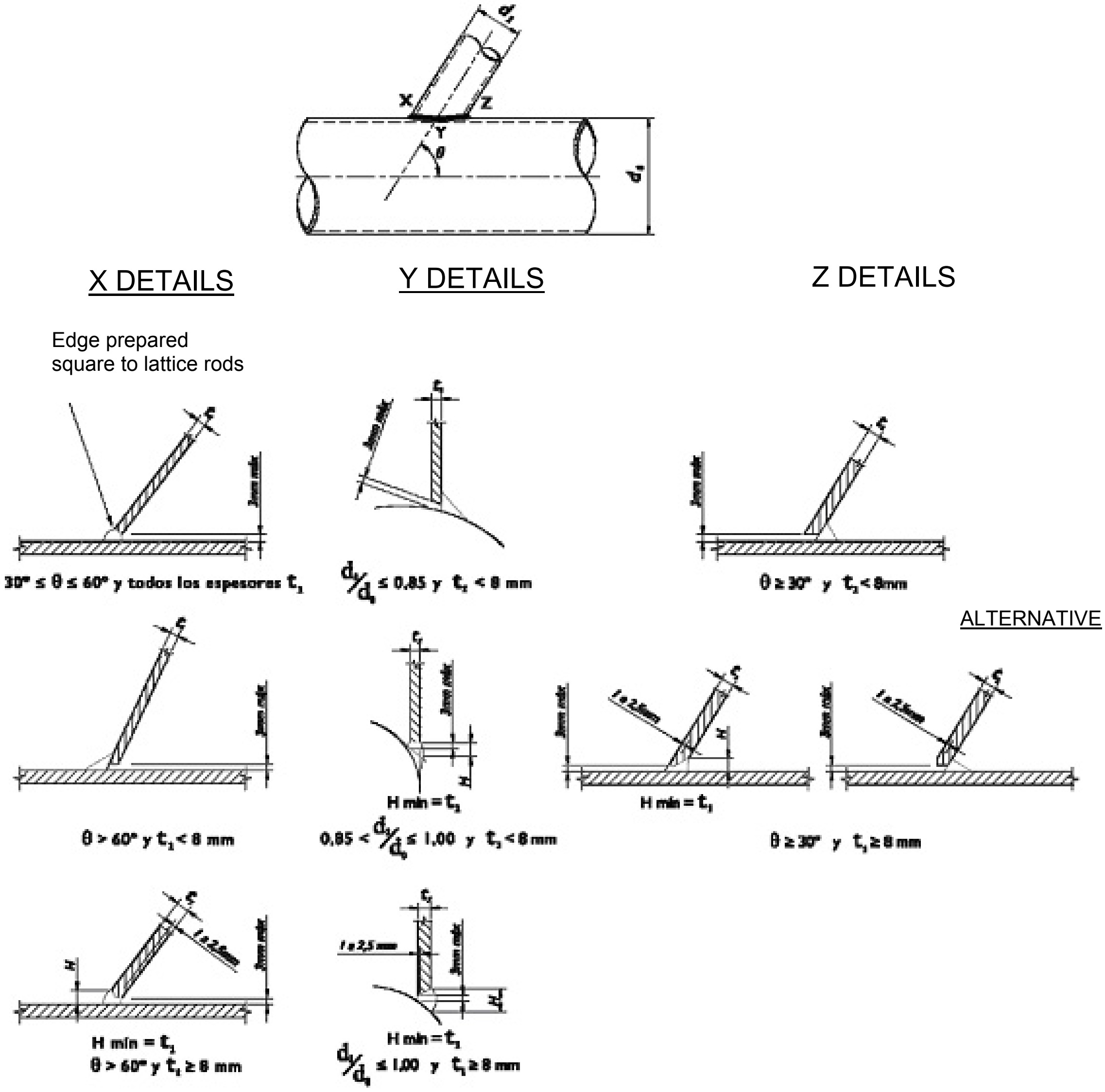
Blocks and their welded joints to chords shall be dimensioned to resist the forces resulting from the forces transmitted to the same by the diagonals or uprights in it, determined in accordance with the provisions of section 60.3.





y todos los espesores	and all thicknesses

Figure 64.3.b Weld preparation and fit-up Construction details of typical welds rectangular hollow sections



y todos los espesores	and all thicknesses

Figure 64.3.c Weld preparation and fit-up. Construction details of typical welds circular hollow sections

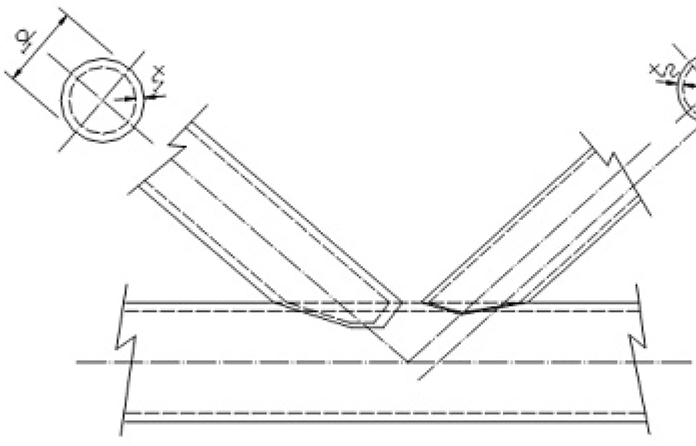
64.4. Manufacture

In direct joints, the ends of brace members shall be cut to fit the chord. Cutting the chord to enable the passage of diagonals or uprights is expressly forbidden.

The use of automatic machines is recommended for said cut. If automatic machines are not available, the use of flat, single, double or triple cuts (Figure 64.4.a) shall be permitted provided that they satisfy the following limitations (Figure 64.4.b).

$g_1 \leq t_1$; $g_1 \leq t_0$; $g_2 \leq 3$ mm

Where g_1 is the distance from the outer face of the diagonal or upright to the face of the chord, and g_2 is the distance from the inner face of the diagonal or upright to the face of the chord.



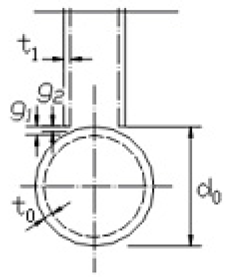


Figure 64.4.a Figure 64.4.b

64.5. Failure modes for hollow sections joints

The design joint resistance between bars of hollow section or between bars of hollow section with bars of open section should be determined by studying the different failure modes of the joint, as detailed below.

The sections below and Annex 9 provide formulas to calculate such resistance for given failure modes. To determine the design resistance for other failure modes, the general methods provided for in this Code should be applied.

Although in general the resistance of a correctly welded joint is greater in tension than in compression, the design resistances given are based on the resistance of compressed joints to prevent possible excessive local deformation or reduced or inadequate rotation capacity.

The possible failure modes to be studied in general are as follows:

a) Failure of the chord face, by plastic failure of the chord face or of the entire section of the chord itself.

b) Failure of the chord web or the chord side wall failure by plastification, flattening or local buckling under the compressed diagonal or upright.

c) Shear failure of the chord.

d) Punching shear of the chord face by crack initiation leading to the separation of the tensioned diagonal or upright of the chord.

e) Brace failure as a result of insufficient effective width.

f) Failure by local buckling of a brace or of a hollow section chord member close to the joint.

The different types of failure for joints with CHS or RHS beams and open-section beams are shown in Figures A-9-1, A-9-2 and A-9-3 of Annex 9.

The design force of a brace shall not be greater than the resistance of the bar itself, calculated in accordance with the general principles contained in this Code, or the lesser of the resistances of its end joints, calculated in accordance with the provisions of this section.

64.6. Welded joints between circular-section hollow beams CHS

64.6.1. General

The design resistance of the joint between circular-section hollow bars shall satisfy the conditions set out in sections 64.6.2 if the joint is flat or 64.6.3 if it is spatial.

For joints that satisfied the geometric conditions set out in Table 64.6.1, it shall only be necessary to check the failure of the chord face and its punching shear. The design resistance shall be the smaller of the resistances obtained for the two possible failure mechanisms.

For joints that do not satisfy the geometric conditions set out in Table 64.6.1, it shall be necessary to check all of the failure mechanisms indicated in section 64.5 above. Furthermore, the secondary moments caused by the rotation stiffness of the node itself must be taken into account, on account of which, in this case, design models that consider the ends of diagonals or uprights to be articulated shall not be valid.

Table 64.6.1 Geometric conditions for joints between circular-section hollow bars

Ratio of diameters		$0.2 \le d_i/d_0 \le 1.0$
Chords	Tension	$10 \le d_0/t_0 \le 50$ (in general) Except: $10 \le d=/t_0 \le 40$ (for X joints)
	Compression	Class 1 or 2 and $10 \le d_0/t_0 \le 50$ (in general), Except: $10 \le d_0/t_0 \le 40$ (for X joints)
Braces	Tension	$d_i/t_i \le 50$
	Compression	Class 1 or 2
Overlap		$25\ \% \le \lambda_{ov} \le \lambda_{ov,lim}$ (see note below)
Gap		$g \ge t_1 + t_2$

NB: Joints between brace members and the chord must be checked for shear if any of the following conditions are true:

a) If the overlap exceeds $\lambda_{ov,lim} = 60\ \%$ and the hidden face of the overlapped upright or diagonal is not welded.

b) If the overlap exceeds $\lambda_{ov,lim} = 80\ \%$ and the hidden face of the overlapped upright or diagonal is welded.

c) If the braces are rectangular hollow section with $h_i<b_i$ and/or $h_j<b_j$

64.6.2. Flat joints

In joints whose diagonals or uprights are only subjected to axial forces, the design axial force $N_{i,Ed}$ must not be greater than the design resistance of the joint $N_{i,Rd}$, as shown in Tables A-9-1, A-9-2 or A-9-3 of Annex 9.

Joints whose braces are subject to axial force and bending moment must satisfy the following:

$$\frac{N_{i,Ed}}{N_{i,Rd}} + \left[\frac{M_{ip,i,Ed}}{M_{ip,i,Rd}}\right]^2 + \frac{\left|M_{op,i,Ed}\right|}{M_{op,i,Rd}} \leq 1{,}0$$

where:

$N_{i,Ed}$, $M_{ip,i,Ed}$, $M_{op,i,Ed}$ are respectively the design axial force, the design bending moment in the plane of the joint and the design bending moment in the plane perpendicular to the joint. These bending moments shall be determined at the point of intersection between the axis of the diagonal or upright and the chord face.

$N_{i,Rd}$, $M_{ip,i,Rd}$, $M_{op,i,Rd}$ are the resistances of the joint to axial force, to bending moment in the plane of the joint and to bending moment in the plane perpendicular to the joint, as given in Tables A-9-1, A-9-2, A-9-3 or A-9-4 of Annex 9.

DY, KT, DX and DK Joints should satisfy the additional requirements set out in Table A-9-5 of Annex 9.

The values of the parameter kg used in Table A-9-1 of Annex 9 for K, N and KT joints are given in Figure A-9-4.

64.6.3. Spatial joints

The resistance of the joint of each brace member to the corresponding chord in the spatial joint is equal to that of the same joint if flat (section 64.6.2), but multiplied by a reduction factor μ given in Table A-9-6 of Annex 9 and in consideration of the suitable value of the parameter k_p given in the corresponding table.

64.7. Welded joints between CHS or RHS brace members and RHS chords

64.7.1. General points

The design resistance of the joint between hollow sections CHS or RHS brace members and RHS chords shall satisfy the conditions set out in sections 64.7.2 if the joint is flat or 64.7.3 if it is spatial.

For joints within the range of validity given in Table 64.7.1, it shall only be necessary to check the failure mode given in the applicable table of Annex 9. The design resistance shall be the smaller of the values obtained for the possible failure mechanisms.

Table 64.7.1 Geometric conditions for joints between diagonals or uprights CHS or RHS and RHS chords

<table>
<tr><td rowspan="3">Joint type</td><td colspan="6">Joint parameters [$i = 1$ or 2, j = overlapped brace]</td></tr>
<tr><td rowspan="2">b_i / b_0 o d_i / b_0</td><td colspan="2">b_i / t_i y h_i / t_i or d_i / t_i</td><td>h_0 / b_0 and h_i / b_i</td><td>b_0 / t_0 and h_0 / t_0</td><td>Gap or overlap b_i / b_j</td></tr>
<tr><td>Compression</td><td>Tension</td><td></td><td></td><td></td></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 1 or 2</td><td rowspan="3">$b_i / t_i \leq 35$ and $h_i / t_i \leq 35$</td><td rowspan="3">≥ 0,5but≤ 2,0</td><td>≤ 35 and Class 1 or 2</td><td>-</td></tr>
<tr><td>K gap N with gap</td><td>$b_i / b_0 \geq 0.35$y$\geq 0.1 + 0.01\, b_0/t_0$</td><td>≤ 35 and Class 1 or 2</td><td>$g/b_0 \geq 0.5(1-\beta)$ but $\leq 1.5(1-\beta)$ 1) and at least $g \geq t_1 + t_2$</td></tr>
<tr><td>K overlap N overlap</td><td>$b_i / b_0 \geq 0.25$</td><td>Class 1</td><td>Class 1 or 2</td><td>$25\,\% \leq \lambda_{ov} \leq \lambda_{ov,lim}$ 2) and $b_i / b_j \leq 0.75$</td></tr>
<tr><td>Circular brace member</td><td>di /b0 ≥ 0.4 but ≤ 0.8</td><td>Class 1</td><td>di /ti≤ 50</td><td colspan="3">As above, but with di replacing b_I and d_i replacing b_j.</td></tr>
<tr><td colspan="7">1) If $g / b_0 > 1.5(1-\beta)$ and $g > t_1 + t_2$, treat the joint as two separate T or Y joints.
2) $\lambda_{ov,lim} = 60\,\%$ if the hidden part is not welded to the chord, and $80\,\%$ if the hidden part is welded to the chord. If the coating exceeds the value $\lambda_{ov,lim}$ or if the brace members are of rectangular section with $h_i < b_i$ and/or $h_j < b_j$, the joint between the braces and the chord face must be checked for shear.</td></tr>
</table>

For joints that do not satisfy the geometric conditions set out in Table 64.7.1, it shall be necessary to check all of the failure mechanisms indicated in section 64.5 above. Furthermore, the secondary moments caused by the rotation stiffness of the node itself must be taken into account, on account of which, in this case, design models that consider the ends of diagonals or uprights to be hingen shall not be valid.

64.7.2. Flat joints

64.7.2.1. Un-reinforced joints

In joints whose brace members are only subjected to axial forces, the design axial force $N_{i,Ed}$ must not be greater than the design axial resistance of the joint $N_{i,Rd}$ determined as follows:

a) For welded joints between brace members of square or circular hollow section and chords of square hollow section, if the geometric conditions in Table 64.7.1 are met and the additional conditions given in Table 64.7.2 are met, the design axial resistance may be determined from the expressions given in Table A-9-7.

b) For joints within the range of validity of Table 64.7.1, it shall only be necessary to check failure of the chord face and brace failure with reduced effective width. The design axial resistance of the joint should be taken as the minimum value arising from these two conditions.
(NB: The design axial resistance for joints between braces of hollow section and chords of square section given in Table A-9-7 have been simplified, omitting those failure criteria that are not critical for joints that are within the range of validity of Table 64.7.2).

Design resistance of any welded non-reinforced joint between SHC or SHR braces and SHR chords that satisfy the geometric conditions of Table 64.7.1 may be determined using the expressions given in Tables A-9-8, A-9-9 and A-9-10, as applicable. For reinforced joints, see 64.7.2.2.

Table 64.7.2 Additional conditions for the use of Table 64.7.1

Type of brace	Joint type	Joint parameters	
Square hollow section	T, Y or X	$b_i/b_0 \leq 0.85$	$b_0/t_0 \geq 10$
	K or N with gap	$0.6 \leq \frac{b_1+b_2}{2b_1} \leq 1.3$	$b_0/t_0 \geq 15$
Circular hollow section	T, Y or X		$b_0/t_0 \geq 10$
	K or N with gap	$0.6 \leq \frac{d_1+d_2}{2d_1} \leq 1.3$	$b_0/t_0 \geq 15$

Brace members connections subjected Joints to combined axial force and bending moment must satisfy the following requirement:

$$\frac{N_{i,Ed}}{N_{i,Rd}} + \frac{M_{ip,i,Ed}}{M_{ip,i,Rd}} + \frac{M_{op,i,Ed}}{M_{op,i,Rd}}$$

where:

$N_{i,Ed}$, $M_{ip,i,Ed}$, $M_{op,i,Ed}$ are respectively the design axial force, the design in-plane internal moment and the design out-of-plane internal moment. These bending moments shall be determined at the point of intersection between the axis of the diagonal or upright and the chord face.

$N_{i,Rd}$, $M_{ip,i,Rd}$, $M_{op,i,Rd}$ are the resistances of the joint to axial force, the design in-plane moment resistance and the design out-of-plane moment resistance, as given in Tables A-9-7, A-9-8, A-9-9, A-9-10 or A-9-11 of Annex 9. For reinforced joints, see section 64.7.2.2.

The special types of welded joints included in Tables A-9-12 and A-9-13 must comply with the design criteria set out for each type of joint in these tables.

64.7.2.2 Reinforced joints

Various types of reinforcements may be used. The correct type depends upon the predominant failure mode in the design resistance of the joint, in the absence of reinforcement.

Flange reinforcement plates may be used to increase the resistance of the joint to chord face failure, punching shear or failure of the brace on account of insufficient effective width. A pair of side plates may be used to reinforce joint against chord side wall failure or shear failure of the chord.

The design resistance of reinforced joints must be determined using Tables A-9-14 and A-9-15.

64.7.3. Spatial joints

Spatial joints must observe the design criteria for flat joints.

The resistance of the joint of each brace to the corresponding chord in the spatial joint is equal to that of the same joint if flat (section 64.7.2), but multiplied by a reduction factor μ given in Table A-9-16 of Annex 9 and in consideration of the suitable value of the parameter kn given in the corresponding table.

64.8 Welded joints between CHS or RHS brace members and I- or H-section chords

For welded joints between braces of hollow section and chords of I or H section that comply with the geometric conditions in Table 64.8, the design resistance shall be determined using the expressions given in Table A-9-17 or A-9-18, as applicable.

Table 64.8 Geometric conditions for welded joints between CHS or RHS diagonals or uprights and I- or H-section chords

<table>
<tr><th rowspan="3">Joint type</th><th colspan="6">Joint parameters [i = 1 or 2, j= overlapped brace member]</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 < 400 mm</td><td rowspan="4">Class 1 or 2 and $h//t<35$
$b / t / < 35$
$d / t / < 50$</td><td rowspan="4">$h/t<35$ $b / t < 35$
$d / t < 50$</td><td>> 0.5 but < 2.0</td><td rowspan="4">Class 1 or 2</td><td></td></tr>
<tr><td>T or Y</td><td rowspan="3">Class 1 or 2 and d_w < 400 mm</td><td rowspan="2">1.0</td><td rowspan="2">-</td></tr>
<tr><td>K with gap
N with gap (see note)</td></tr>
<tr><td>K with overlap
K with overlap</td><td>> 0.5 but < 2.0</td><td>> 0.75</td></tr>
<tr><td colspan="7">NB: Joints between braces and the chord must be checked for shear if any of the following conditions are true:
a) If the overlap exceeds $\lambda_{ov,lim}$ = 60 % and the hidden face of the overlapped brace is not welded.
b) If the overlap exceeds $\lambda_{ov,lim}$ = 80 % and the hidden face of the overlapped upright or diagonal is welded.
c) If the brace members are rectangular hollow section with $h_i<b_i$ and/or $h_j<b_j$</td></tr>
</table>

For joints that satisfy the geometric conditions in Table 64.8, it shall only be necessary to consider the design criteria set out in the corresponding table. The design resistance of the joint shall be the smaller of the values obtained from all of the applicable criteria.

For joints that do not satisfy the geometric conditions set out in Table 64.8, it shall be necessary to check all of the failure mechanisms indicated in section 64.5.Furthermore the secondary moments caused by the rotation stiffness of the node itself should be considered .

In joints whose brace members are only subjected to axial forces, the design axial force $N_{i,Ed}$ must not be greater than the design resistance of the joint $N_{i,Rd}$ obtained in Table A-9-17.

Joints whose brace members are subject to axial force and bending moment must satisfy the following:

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

where:

$M_{ip,i,Ed}$ and M_{ipjiRd} are the design moments and moment resistance in the plane. These bending moments shall be determined at the point of intersection between the axis of the member diagonal or upright and the chord face. The moment resistance $M_{ip,i,Rd}$ shall be obtained using Table A-9-18.

If stiffeners are used in the chord (see Figure 64.8) the resistance of the member $N_{i,Rd}$ for T, X, Y or K joints with spacing and N joints with spacing (Table A-9-17) shall be determined as follows:

$N_{i,Rd} = 2\, f_{yi}\, t_i\, (b_{eff} + b_{eff,s})$

where:

$b_{eff} = t_w + 2r + 7\, t_f\, f_{yo} / f_{yi}$ but< $b_i + h_i - 2t_i$

$b_{eff,s} = t_s + 2a + 7\, t_f\, f_{yo} / f_{yi}$ but< $b_i + h_i - 2t_i$

$b_{eff} + b_{eff,s} < b_i + h_i - 2t_i$

in which:

a Weld throat of the stiffener, ‘2a’ becomes ‘a’ if fillet welds are used on each side of the stiffener;

s Relates to the stiffener.

The stiffener shall have a thickness no less than the thickness of the web of the I-beam.

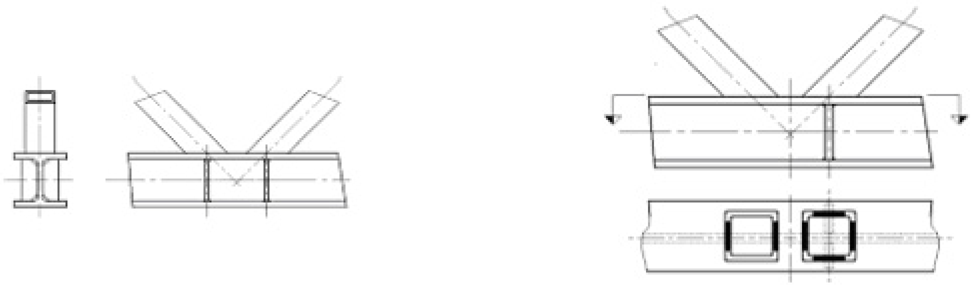


Figure 64.8 Effective perimeter of the diagonal or upright without (left) and with (right) stiffeners

64.9. Welded joints between CHS or RHS brace members and U-section chords

For welded joints between braces of hollow section and U-section chords that satisfy the geometric conditions in Table 64.9, the design resistance shall be determined using the expressions given in Table A-9-19.

Table 64.9 Geometric conditions for welded joints between CHS or RHS braces and U-section chords

<table>
<tr><td rowspan="3">Joint type</td><td colspan="6">Joint parameters [$i = 1$ or 2, j = brace member]</td></tr>
<tr><td rowspan="2">b_i / b_0</td><td colspan="2">b_i / t_i y h_i / t_i o d_i / t_i</td><td>h_i / b_i</td><td>b_o / t_o</td><td>Gap or overlap b_i / b_j</td></tr>
<tr><td>Compression</td><td>Tension</td><td></td><td></td><td></td></tr>
<tr><td>K gap
N gap</td><td>≥ 0.4 and $b_o \leq 400$ mm</td><td rowspan="2">Class 1 or 2 y $h_i/t_i \leq 35$
$b_i/t_i \leq 35$
$d_i/t_i \leq 50$</td><td rowspan="2">$h_i/t_i \leq 35$
$b_i/t_i \leq 35$ $d_i/t_i \leq 50$</td><td rowspan="2">≥ 0.5 but ≤ 2.0</td><td rowspan="2">Class 1 or 2</td><td>$0.5(1-\beta^*) \leq g/b_o^* \leq 1.5(1-\beta^*)$ 1) and $g \geq t_1 + t_2$</td></tr>
<tr><td>K overlap
N overlap</td><td>≥ 0.25 y $b_o \leq 400$ mm</td><td>25 % $\leq \lambda_{ov} \leq \lambda_{ov,lim}$ 2)
$b_i/b_j > 0.75$</td></tr>
<tr><td colspan="7">$\beta^* = b_1/b_0^*$
$b_0^* = b_0 - 2(t_w + r_0)$
1) This condition shall only apply if $\beta \leq 0.85$
2) Joints between members and the chord must be checked for shear if any of the following conditions are true:
a) If the overlap exceeds $\lambda_{ov,lim} = 60$ % and the hidden face of the overlapped upright or diagonal is not welded.
b) If the overlap exceeds $\lambda_{ov,lim} = 80$ % and the hidden face of the overlapped upright or diagonal is welded.
c) If the uprights or diagonals are rectangular tubular with $h_i < b_i$ and/or $h_j < b_j$</td></tr>
</table>

The secondary moments in the joints caused by their stiffness must be taken into account.

In gap joints, the design resistance to axial force of the chord $N_{0,Rd}$ shall be determined in consideration of the shear force transmitted by the diagonals or uprights to the chord, disregarding the related secondary moment. This shall be checked in accordance with the provisions of Chapter IX.

Section 65. Joints in foundations

65.1. General

The support of the structure shall comply as closely as possible with the binding conditions provided for in the design.

The support mechanisms shall be designed such that:

- they can transmit to the foundations the forces considered in the design without causing in it any stresses that it cannot properly resist.
- they enable the movements considered in the design without causing stresses not provided for therein.

- they can be inspected and maintained easily.

The foundations shall in turn be dimensioned such that they are able to transmit to the ground the forces that they receive from the structure such that it can resist them without causing any settlement or movement that invalidates the support conditions considered in the structural design.

The maintenance plan for the structure shall give details of how to change support mechanisms if required.

The zones around these mechanisms shall be designed such that they are able to resist the stresses to which they may be subjected during such a change.

65.2. Base plates

Base plates are the support mechanism most frequently used to join a support to its foundations.

The joint between a support and its foundations using a base plate may be considered to be a rigid joint or perfect restraint if the conditions set out in section 65.2.5 are satisfied. If the end of the support is not intended to bend (hinged joint) a pin or similar device must be placed between the plate and the support.

The base plate shall be dimensioned to transmit the axial tension and compression forces, shear forces, bending moments and torsional moments determined in the design.

65.2.1. Transmission of shear stresses

The shear stresses generated by the shear forces and potential torsional moment may be transmitted to the foundations by:

- Friction. Where $N_{c,Ed}$ is the design value of the normal compression force , which shall include any pre-loaded force of the anchor bolts. The maximum shear force that can be transmitted by friction is:

$$V_{Rd}=C_{f,d}\,N_{c,Ed} \geq V_{Ed,ef}=\sqrt{V_{y,Ed}^2+V_{z,Ed}^2}+\frac{M_{x,Ed}}{0{,}25b}$$

where b is the smaller dimension of the base plate, $V_{y,Ed}$ and $V_{z,Ed}$ are the components of the designed shear force and $M_{x,Ed}$ the design torsional force, concomitant with $N_{c,Ed}$.

The Friction coefficient $C_{f,d}$ between base plate and the concrete shall be $C_{f,d}$ = *0.20* for sand-cement mortar. For other types of mortar, the Friction coefficient should be determined by testing in accordance with UNE-EN 1990, Annex D - Using one of the connector types admitted in the Recommendations for the design of composite highway bridges, RPX-95, of the Directorate General of Highways (DGC) and calculated in accordance with the same.

- Using the anchor bolts provided for. This method is not recommended if the bolts are also required to resist significant tensile forces.

The Designer may consider together the friction resistance and the resistance of the anchor bolts jointly to resist the total shear and torsional forces.

If anchor bolts are used to transmit shear forces, the following conditions shall be taken into account:

- The holes made in the base plate for the bolts may have the clearance indicated by the Designer to facilitate their assembly, but in this case washers with standard drill holes shall be placed on them and welded to the base plate using a weld strong enough to transmit the shear force that the bolt is required to absorb.

- The shear and flattening resistance of the bolt against the base plate or against the lock washer shall be the smaller of those determined in accordance with the provisions of Sections 58.6 and 58.7, and the value obtained from:

$$F_{V2.Rd} = \frac{\alpha_b f_{ub} A_S}{\gamma_{M2}},$$

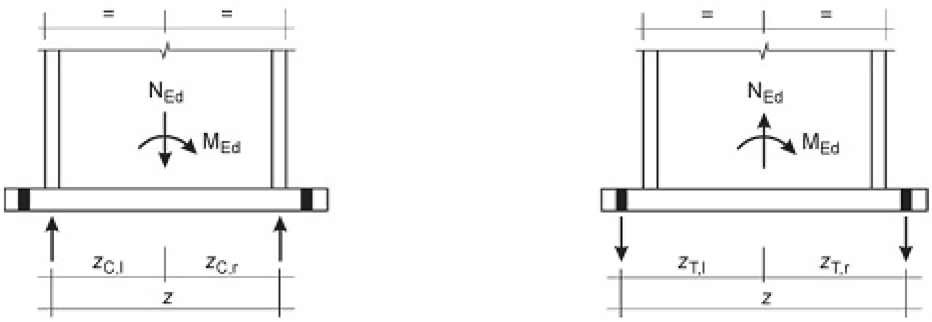
where $\alpha_b = 0.44 - 0.0003 f_{yb}$, where f_{yb} is the yield strength of the bolt anchor (in N/mm^2), where $235\ N/mm^2 \leq f_{yb} \leq 640\ N/mm^2$.

- The flattening resistance to concrete shall be calculated in accordance with the provisions of Article 7.3.2.1 of Recommendations RPX-95 for designing composite highway bridges, RPX-95.

65.2.2. Transmission of compression forces

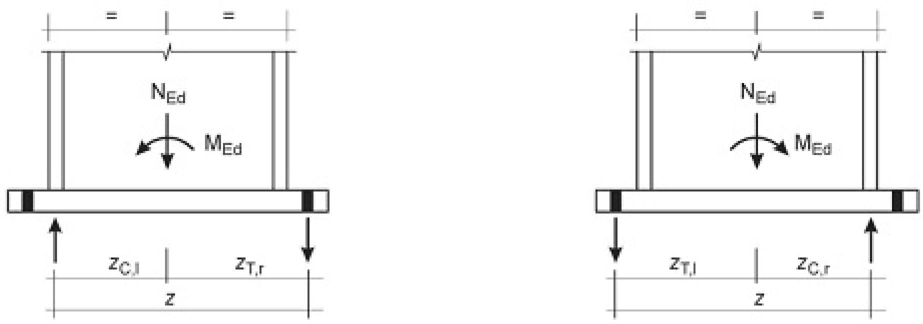
The compression forces generated by the axial force and bending moments may be transmitted from the compressed members of the support to the foundations, distributed via the base plate.

The distribution of forces shall be assumed to be in equilibrium with the axial force and the bending moment of the support at start, in accordance with section 56.2. As a simplification, the criteria given in Figure 65.2.2.a may be used.



a) Dominant compression axial force

b) Dominant tensile force



c) Dominant moment

d) Dominant moment

Figure 65.2.2.a Determination of the lever arm z for column base connections

The resistance of the compression zone shall be determined as follows:

$F_{C,Rd}=f_{jd}A_0$,

where f_{jd} is the maximum resistance of the concrete and A_0 is the maximum distribution area in compression, both parameters as defined below.

The maximum distribution area in compression shall be delimited by lines parallel to the surfaces of the profiles of the column, at a maximum distance (Figure 65.2.2.b).

$$c = t\sqrt{\frac{f_y}{3 f_{jd}\gamma_{M0}}}$$

Where f_y is the yield strength of the steel of the base plate and t its thickness.

The maximum resistance of the concrete is given by:

$$f_{jd} = \frac{\beta F_{Rdu}}{A_0'}$$

where β_j is a factor that may be taken as 2/3 if the levelling mortar between the plate and the concrete of the foundations has a resistance at least equal to $0.2f_{ck}$ and the thickness no greater than 0.2 times the smaller dimension of the plate; A'_0 is an approximation of the maximum distribution area in compression and F_{Rdu} is the maximum concentrated compression force that can act on the concrete in accordance with the EHE-08 Code (taking A'_0 as the restricted area in which the forces applied).

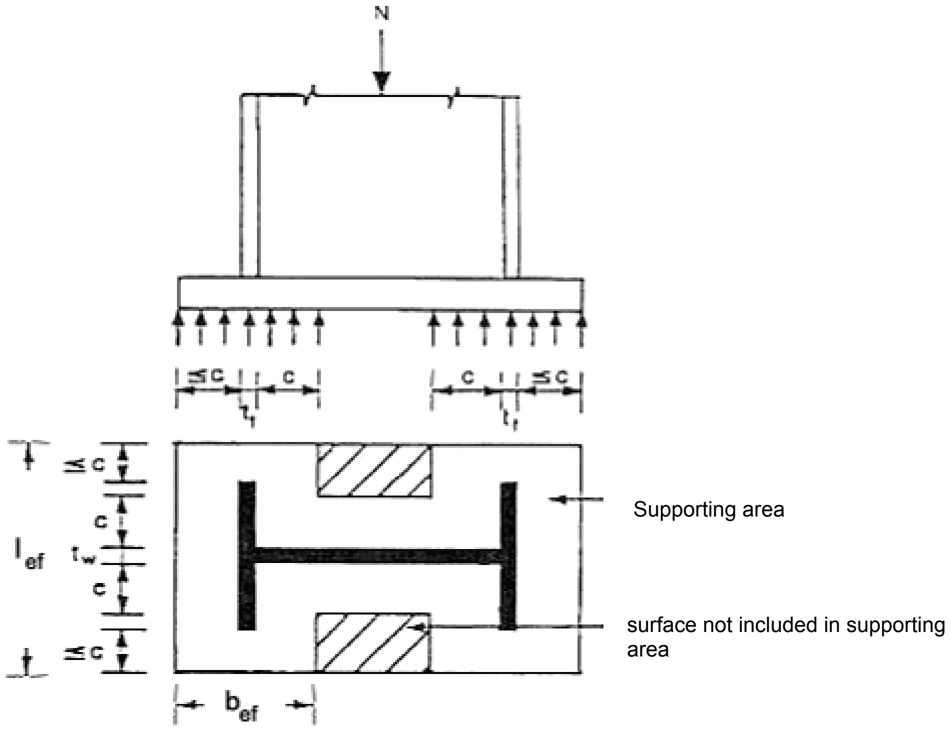


Figure 65.2.2.b

65.2.3. Transmission of tensile forces

Tensile forces generated by the axial force and bending moments should be resisted by the anchor bolts. It is recommended that the bolts be screw to the plate. A washer shall always be placed between the nut and the base plate.

Once the structure is assembled and the nuts tightened, they shall be immobilised, preferably using a lock nut or peening the thread. Weld spots may not be used for this task.

The tensile resistance of an anchor bolt shall be determined in accordance with the provisions of Section 58.7.The shear resistance of the bolt shall be taken as the lowest value obtained from section 65.2.1.If the bolts are welded to the plate, the material in them must be weldable and the join weld shall be full strength (Figure 65.2.3.a).In this case, suitable measures shall be taken to prevent the occurrence of lamellar tearing.

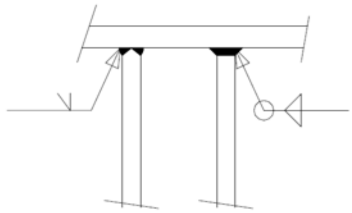
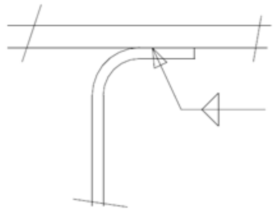


Figure 65.2.3.a

Bolts should not be welded to the base plates using fillet weld on a foot formed at the end of the same to be welded to the plate (Figure 65.2.3.b).



NO

Figure 65.2.3.b Detailed to be avoided when welding the bolt to the base plate

Bolts may be preloaded if they are of suitable quality. The preloading force shall be determined by the Designer, and shall not exceed the value N_0 given in section 58.8.This solution is recommended for structures subject to dynamic effects, impacts or fatigue loads.

The transfer of the tensile force of an anchor bolt to the concrete may be done by adhesion, in which case the bolt shall end in a normal foot and its length shall be as provided for in the EHE-08 Code, or to an end plate, attached to the bolt by means of a nut and lock nut.

In this case, the following should be true:

$$\frac{T_{Sd}}{A_{pl}} \leq f_j$$

where T_{Ed} is the tensile force in the bolt, A_{pl} is the area of the plate and f_j is the value of the pressure against the concrete. In this calculation, the depth of the plate L shall be used instead of the thickness of the shoe h.

Transmission to the concrete shall be made by means of shear stresses in the side surface of a truncated cone or pyramid, depending on whether the plate is circular or square, extending from the plate to the surface of the footing with a half-angle at the vertex of 10°.

These shear stresses shall be checked in accordance with the provisions of the EHE Code.

65.2.4. Transmission of bending forces

The resistance of the base plate and the anchor bolts shall be checked in accordance with the provisions of section 61.2, treating it as a joint with endplate. In this case, to define the compression surface, the distance *c* (see Figure 65.2.2.b) defined above shall be taken instead of *k* in the zone of the flange in compression, and the distance *c* on both sides of the zone of the web in compression.

65.2.5. Base plate stiffness

Bases shall be deemed stiff in the following cases:

For non-translational portal frames, if any of the following three conditions are satisfied:

$$\lambda_0 \leq 0{,}5$$

$$0{,}5 \leq \lambda_0 < 3{,}93 \quad y \quad S_j \geq \frac{7(2\lambda_0 - 1)EI_C}{L_C}$$

$$\lambda_0 > 3{,}93 \text{ and } S_j \geq \frac{48EI_C}{L_C}$$

In any other case if $S_j \geq \dfrac{30EI_C}{L_C}$

In the above expressions, λ_0 is the slenderness of the column assuming it is bihinged, I_c and L_c are its corresponding second moment of area and length, and S_j the rotation stiffness corresponding to the base plate.

The rotation stiffness of the base plate is given by the following expression:

$$S_{j,ini} = \frac{E_z^2}{\sum \frac{1}{k}},$$

where k_i is the stiffness coefficient for basic component (detailed below) and z is the lever arm (see Figure 65.2.2.a).

The bending stiffness of the plate is given by:

$$K_p = 0{,}85\frac{b_{ef}t^3}{m^3}$$

where b_{ef} is the effective width as per Section 61.2 (establishing an analogy between plate and the endplate joints), *t* is the thickness of the base plate and *m* the corresponding distance between the bolt and the corresponding plastic hinge formation line.

The stiffness of each bolt row is:

$$k_{at} = 1{,}6\frac{A_s}{L_b}$$

where A_s is the resistance area of the bolt and L_b its length.

Leverage may be generated in the plate if:

$$L_b \leq \frac{8{,}8m^3 A_s}{b_{ef}t^3}$$

In this case, the factor 0.425 shall be taken instead of 0.85 for the bending stiffness of the plate, and the value 2.0 shall be taken instead of 1.6 for the stiffness of the bolt row.

The stiffness provided by the concrete and base plate in compression shall be calculated as:

$$k_c = \frac{E_C \sqrt{b_{ef} \ell_{ef}}}{1{,}275\, E}$$

where b_{ef} and ℓ_{ef} are the effective dimensions of the concrete area beneath the compressed flange of the column (Figure 65.2.2.b).

65.3. Other methods for connecting columns to foundations

A column may be connected to the foundations by embedding a certain length of the column into the foundations (Figure 65.3).

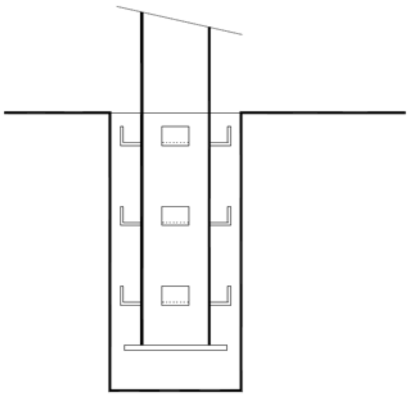


Figure 65.3

In this case, the axial force (tension or compression) is transmitted by means of connectors arranged in the shaft of the column. In the concrete, the necessary reinforcements shall be installed to transmit it from the grout to the concrete itself.

The shear force and bending moments shall be transmitted by means of pairs of forces that compress the shaft to the foundations, chosen such that:

- At no point shall the pressure of the shaft to the concrete be greater than its design resistance, f_{cd}.

- The shaft can resist the stresses generated by such forces.

Section 66. Support members

66.1. Neoprene support devices

Neoprene support devices are used to create a non-slip-resistant nominally pinned connection at the end of the beam, enabling simultaneous movement in two directions and the absorption of vertical and horizontal loads.

The support may be made of neoprene plates only (unreinforced neoprene), although it is advisable to insert steel plates between the neoprene plates (reinforced neoprene) to restrain lateral expansion and reduce vertical settling of the support.

The dimensions of the neoprene support should comply with the following conditions:

Stability: the following condition must be satisfied to prevent buckling:

$$e_n \leq 0.2\ b$$

where e_n is the total thickness of the neoprene (excluding the thickness of the steel plates) and *b* is the smaller horizontal dimension of the support.

Shape: The following condition must be satisfied to limit vertical shortening:

Rectangular shape: $\frac{ab}{t(a+b)} > 12$

Strip shape: $\frac{a}{t} > 12$

Circular shape: $\frac{d}{t} > 12$

where *a* is the larger horizontal dimension of the support, *d* is the diameter of the circular support and *t* is the thickness of each of the layers of neoprene.

Contact pressure: The horizontal dimensions of the support should satisfy the following:

$$\frac{N_{\max}}{\sigma_n} \leq ab \leq \frac{N_{\min}}{3}$$

where:

a and *b shall* be provided in mm,

N_{max} and N_{min} are the maximum and minimum vertical reactions in N,

σ_n is the admissible compression stress of the neoprene in N/mm^2.Unless data is provided by the Manufacturer, the following values may be taken: $\sigma_n = 3\ N/mm^2$ for unreinforced neoprene and $\sigma_n = 5\ N/mm^2$ for reinforced neoprene.

The condition $ab \leq \frac{N_{\min}}{3}$ need not be observed if the neoprene support is fitted into one or both of the parts to be joined.

Angular distortion: Angular distortion γ should satisfy the following condition:

$$\gamma = \frac{\delta}{e_n} \leq 0{,}5$$

where δ is the relative movement between the upper surface and lower surface of the support.

As a result, the horizontal force H that the support can transmit shall be limited by the following expression:

- $H = \frac{\delta G a b}{e_n} \leq 0{,}5 Gab$

where G is the transverse modulus of elasticity of the neoprene. If the Manufacturer provides no data, 1 N/mm^2 may be taken for long-term loads and 2 N/mm^2 for instantaneous loads.

66.2. Metal support devices

Metal support devices are used to create a slip-resistant or non-slip-resistant nominally pinned connection at the end of the beam.

When transmitting compression reactions only and if the support device comprises a pair of plates, one connected to the structure and the other to the foundations, the following must be taken into account when dimensioning:

- The plates should be rectangular, with a ratio of dimensions between 1:1 and 2:3.
- Their centre shall coincide with the crossing point of the reaction F_{Ed}.
- The effect of friction between plates shall be taken into account, for which it shall be assumed that $\mu = 0.3$.
- If it is possible that the reaction could be a tensile reaction or, even if it is a compression reaction, its value is small, suitable devices shall be installed to prevent the separation of the plates. Such devices shall enable the movement of the plates in their contact plane if this is provided for in the part binding conditions.
- The contact pressure of the plate to the concrete shall not exceed the value f_j defined in section 65.2.2.
- The contact pressure between metal plates shall not exceed $0.80f_y$.

If the support comprises a sphere resting between two flat plates, the reaction $F_{Ed,ser}$ in serviceability limit state, expressed in kN, shall satisfy the following:

$$F_{Sd,ser} \leq 1{,}64 f_y^3 \left(\frac{r}{E}\right)^2$$

where f_y is the lower yield strength of the sphere or of the plates, in N/mm^2, r is the radius of the sphere in mm and E the modulus of elasticity of the steel in N/mm^2.

If the support comprises a cylinder of length ℓ resting between two flat plates, the reaction $F_{Ed,ser}$ in serviceability limit state, expressed in kN, shall satisfy the following:

$$F_{Sd,ser} \leq 0{,}062 f_y^2 \frac{r\ell}{E}$$

where f_y is the lower yield strength of the cylinder or of the plates, in N/mm^2, and r is the radius of the cylinder in mm.

In the expressions above, if the yield strength of the steel is equal to or greater than 500 N/mm^2, it shall take a reduced value $f_{y,red} = 0.12f_y + 440$ (in N/mm^2). If the yield strength is not known, but the Vickers hardness HV as per ISO 4964 is known, the following value may be taken $f_{y,red} = 0{,}273HV + 440$ (in N/mm^2).

For cylindrical surfaces that have undergone a surface hardening treatment or had a special coating applied, the yield strength or the hardness of the surface layer may be used if its thickness is greater than:

$$25r\frac{f_y}{E} \text{ or } 55r\frac{HV}{E}$$

respectively. In these expressions, r should be in mm and f_y and E in N/mm^2.

In general, it shall not be necessary to check the ultimate limit state of spherical or cylindrical joints.

Joints and rolls are commonly made of high-strength Martensitic chrome steels, quenched and tempered, in accordance with UNE-EN 10088-1.The mandatory certificate to be provided by the Manufacturer, in addition to the mechanical characteristics, shall give details of the manufacturing method, heat treatment, hardness and, if agreed, the toughness of the material.

Brittle fracture safety shall be checked if the steel used has an yield strength greater than 1000 N/mm^2 or an HV hardness of more than 450.

CHAPTER XV. STRUCTURAL MEMBERS

Section 67. Beams

This section includes verification of any prismatic member that satisfies the ratio L/h > 5 and is subject to axial forces, simple bending or torsion or any combination of these.

This section covers full-web beams and light-web beams. Truss beams are covered in Section 72 on triangular structures.

67.1. Full-web beams

Full-web beams are rolled or welded beams with an opaque web whose cross-section is uniform or variable lengthways.

Full-web beams should undergo the relevant checks in serviceability limit state (Chapter X) and ultimate limit state (Chapter IX).

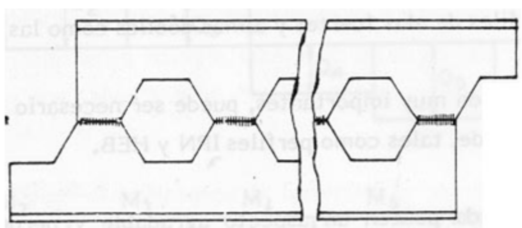
67.2. Light-web beams

Light-web beams are beams whose web has several equal voids evenly arranged along it.

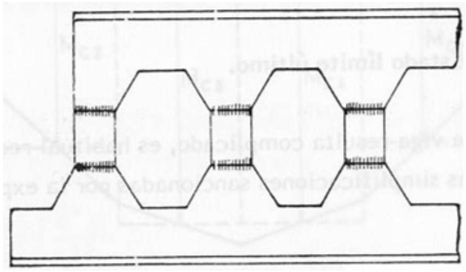
67.2.1. Types of light-web beams

The following types of light-web beams may be used:

a) Hollow-core beams (hexagonal or circular cells)



b) Elevated hollow-core beams (octagonal cells)



67.2.1.1. Ultimate limit state checks

The lightest section should be checked in bending and shear, as should the section in which the interaction of shear force and bending moment is least favourable, causing the maximum longitudinal direct stress. Furthermore, the joint area between both parts of the hollow-core beam must be able to resist the shear force to which it is subjected.

67.2.1.2. Serviceability limit state checks

To check the deformation limit state, both deflection components caused by bending f_M and shear f_V should be taken into account.

A conservative value for the bending deflection component f_M may be obtained in a simplified manner by taking the value of the inertia of the section in the lightest zone, where the inertia is lowest I_{min}, as the inertia of the section.

An average value of the inertia I_M of both section types (most lightened and least lightened) may also be used to calculate said deflection f_M.

Shear deflection f_V is obtained using the equivalent shear area of section A_e.

Section 68. Latticestructures

Lattices are flat structures formed by members arranged in two directions perpendicular to one another and loaded perpendicularly to the middle plane of the structure.

There may be a predominant direction, in which the main beams are laid, in which case the structural members arranged perpendicular to this (secondary beams) are used to distribute loads transversely. Forces shall be calculated as with a grid.

A simplified calculation of forces may be used for secondary beams, using a transversal distribution method for the loads applied in order to obtain the loads to be taken into account for the calculation and verification of the main beams.

Once the forces in the beams that make up the lattice have been determined, they shall be checked for the corresponding ultimate and serviceability limit state, in accordance with the provisions of Section 67.

Section 69. Floor slabs

Steel floor slabs are floor slabs whose resistance members are made of steel.

Generally, these floor slabs are unidirectional floor slabs formed by steel joists with non-load-bearing ceramic or concrete beam fill members.

Joists shall be checked like steel beams, in consideration of Section 67.

In the case of industrial building floor slabs that are required to withstand large loads, the beam fill may be made of steel parts, forming a flat lattice in which the beams can provide similar dimensions in both directions. Details of checks are given in Section 68.

In the case of inclined roof slabs with joists (purlins) supported on main beams or trusses, it should be noted that if the web of the purlin is not in a vertical plane, bending inclination occurs in the plane perpendicular to the web, the extent of which depends on the of the cover.

In any case, the purlins shall be checked in accordance with the provisions of Section 67.

Section 70. Columns

This section sets out the checks to be carried out on structural steel members subjected predominantly to axial compression forces. The columns may be simple or combined.

Simple columns are columns that are made up of a single section or by two or more main sections in contact or spliced to each other using steel shells at a distance s between joins, measured from centre to centre, ensuring that $s \leq 15i_{min}$, where i_{min} is the minimum radius of gyration of one of the sections that make up the column in question or, where applicable, the chord, in the most generic manner. Under these conditions, for practical purposes, these columns may be checked as if they were a column made up of a single section.

Combined columns are columns made up by two or more single sections spliced together. Joining members may be batten plates or trusses. Combined columns shall be checked in consideration of their shear deformability, in accordance with the provisions of 70.5 and Section 71.

This section covers:

- Columns of uniform section.
- Columns of variable section.
- Column subjected to continuous variable axial stress.
- Columns subjected to concentrated loads along their directrix.
- Columns of built-up section.

70.1. Columns of uniform section

This section covers columns whose cross-section is constant throughout its directrix, regardless of its shape.

The cross-sections may be open or closed and, depending on their symmetrical characteristics, they may have double symmetry, single symmetry or point symmetry.

The provisions of section 35.1 shall be taken into account when checking the column in compression.

In the presence of compression and bending forces, the provisions of sections 35.2 and 35.3 shall be taken into account.

If the centre of gravity and the shear centre of the section are not the same, the possibility of buckling by bending and torsion must be taken into account, and the provisions of Section 35.1.4 shall apply.

If the shear centre is the same as the centre of gravity, and the cross-section has low torsional stiffness, the possibility of buckling by pure torsion must be taken into account, if the polar radius of gyration is greater than the radius of gyration in torsion, in which case the provisions of 35.1.4 shall apply.

70.2. Columns of variable section

This section covers columns made up of a single section or by several sections or plates spliced together continuously, whose cross-section varies slightly along the directrix.

These columns may be checked for buckling by determining the value of an equivalent radius of gyration, obtained as a function of the variation in inertia and the way the cross-section varies along the directrix.

70.3. Column subjected to continuous variable axial force

In the case of columns of constant section subject to a variable axial force along the directrix, a buckling length may be used to check buckling such that its ratio β is determined as a function of the maximum and minimum values of the axial force and the way said axial force varies along the directrix.

70.4. Columns subjected to concentrated loads along their directrix

Columns of constant section with point loads applied along the directrix may be checked for buckling by using a buckling ratio β that is a function of the relative application position of the concentrated load or loads and the binding conditions of the column.

In the typical case of application of n point loads along the directrix, the buckling ratio β may be obtained using the following expression:

$$\beta = \sqrt{\sum_{1}^{n} \left(\alpha_i \beta_i^2 \right)}$$

in which:

$$\alpha_i = \frac{P_i}{\sum_1^n P_i}$$

The ratio β_i is the buckling ratio corresponding to the load P_i, as if it were acting independently.

70.5. Columns of combined section

Columns of combined section are columns made up of two or more sections, spliced together by means of sections or plates to ensure the combined resistive effort of the main sections.

The joins may be made by means of Lattices with uprights and diagonals, forming a triangular combined column.

If the joins are made only across uprights and these are plates, this forms a battened combined column.

If the members of the join are arranged with the gap between them of less than $15i_{min}$, where i_{min} is the minimum radius of gyration of one of the main sections, the column is considered, for checking purposes, to be made up of a single section, with the mechanical characteristics of the group of main sections.

The provisions of Section 71 shall be used to check combined columns.

Section 71. Combined members

71.1. General

Combined members are structural members (columns, braces, etc.) made up of two or more single sections, parallel to the directrix, joined discontinuously and modularly by means of a lattice structure (diagonals or brace members) or by means of members perpendicular to the directrix (batten plates) with a view to ensuring a combined resistive effort from all of the sections that make up the built-up structural member.

To ensure the transmission of forces, batten plates should also be placed at the ends of the combined member and joined to its end plates (base and head).

The joint members of the main sections, whether diagonals or uprights, or batten plates, shall divide the member into equal portions of length a, where the number of portions in a single structural member is at least three.

The length a of each portion into which the member is divided must not be larger than 50 i_{min}, where i_{min} is the minimum radius of gyration of one of the main sections (or main chords).

The resistance of joint members, batten plates and/or brace members must be checked for the stresses indicated in 71.2.3

Joint members, batten plates and/or brace members are joined to the main sections using bolts or welding, and the resistance of these joints must be checked for the stresses indicated in 71.2.3

The joint member systems of the lattice on opposite sides of a combined member with two planes of triangulation should have the same layout, i.e. one system must be the shadow of the other.

If they are used as diagonal joint members, the angle that they form with the main sections shall typically be between 30° and 60°.

If parallel batten plate planes are used, they must be arranged facing (one batten plate plane must be the shadow of the other).

Furthermore, if loads are applied to intermediate points of the member in question, batten plates shall also be placed in these points.

The checks set out in the sections below are based on the assumption that the combined member has articulated ends that prevent lateral movement. Furthermore, the centre of the combined member shall be considered to have an imperfection of value e_0 = L/500.

71.2. Checking built-up members for buckling

71.2.1. General

When checking built-up members in compression, a distinction shall be made between triangular built-up members and battened built-up members.

Moreover, the existence of two possible buckling planes shall be taken into account, requiring different checks. The material axis of inertia is the main axis that passes through the centre of mass of the portions of all of the sections that make up the member. Principal axes that do not satisfy this condition are called free axes of inertia.

71.2.2. Buckling checks in a plane perpendicular to the material axis of inertia

When a built-up member is checked for buckling in a plane perpendicular to the material axis of inertia (buckling by bending around the material axis of inertia), the check shall be carried out as if it were a simple member with the cross-section characteristics corresponding to the built-up member.

71.2.3. Buckling checks in a plane perpendicular to the free axis of inertia

The single section portion (usually the chord) between two consecutive joins must be checked assuming a design axial force $N_{cor,Ed}$ the value of which depends on the join type.

The design value of the axial compression force on the chord $N_{cor,Ed}$ in the case of two identical chords is:

$$N_{cor,Ed} = 0{,}5N_{Ed} + \frac{M_{Ed} h_0 A_{cor}}{2I_{ef}}$$

where M_{Ed} is the design value of the maximum bending moment in the centre of the built-up member, taking account of second-order effects.

$$M_{Ed} = \frac{N_{Ed} e_0 + M'_{Ed}}{1 - \frac{N_{Ed}}{N_{cr}} - \frac{N_{Ed}}{S_v}}$$

in which:

$N_{cr} = \frac{\pi^2 E I_{ef}}{L^3}$ is the effective elastic critical axial force of the built-up member.

N_{Ed} is the design value of compression axial force in the middle of the built-up member.

M'_{Ed} is the design value of the maximum bending moment to the built-up member, without second-order effects.

h_0 is the distance between the centroids of the chords of the built-up member.

A_{cor} is the area of the cross-section of a chord.

e_0 is the geometric imperfection taken as L/500.

I_{ef} is the effective inertia of the built-up member, the value of which can be obtained from section 71.2.3.1 (triangular members) and section 71.2.3.2 (battened members) below.

S_v is the shear stiffness of the lacing used for the join or battened panel; said stiffness depends on the type of join used and its value can be seen in section 71.2.3.1 (triangular members) and section 71.2.3.2 (battened members) .

The join members of the triangulations (triangulated members) or the battened panels (determination of bending moments and shear forces in the chords and batten plates of the battened members) must be checked for the end panel, assuming application of a shear force with the following value:

$$V_{Ed} = \pi \frac{M_{Ed}}{L}$$

where M_{Ed} and L have the same meaning as in previous paragraphs.

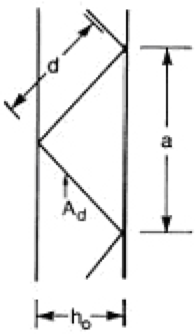
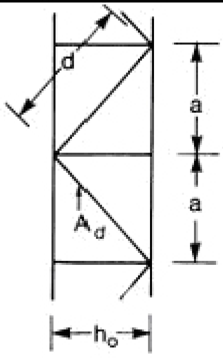
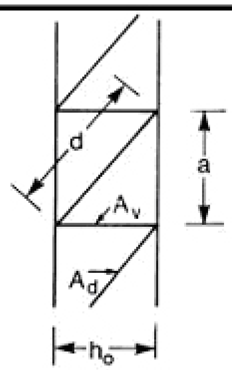
For chords subject to compression, the design value of the axial compression force $N_{cor,Ed}$ obtained as explained above, must demonstrate that:

$$N_{cor,Ed} \leq N_{b,Rd}$$

where $N_{b,Rd}$ is the design buckling resistance of the compression chord, using the buckling length given in Figure 71.2.3.1.b.

71.2.3.1. Laced members

Figure 71.2.3.1.a sets out the shear stiffness values S_v for specific join geometries typically used in laced members. If the joined type used is not shown, the value of S_v may be obtained on the basis that it is the shear stiffness of the join or the shear force required to produce a unit shear deformation, which is the same thing.

Triangulation system			
S_V	$\frac{nEA_d a h_0^2}{2d^3}$	$\frac{nEA_d a h_0^2}{d^3}$	$\frac{nEA_d a h_0^2}{d^3\left[1+\frac{A_d h_0^3}{A_V d^3}\right]}$

n is the number of planes of lacing
A_d and A_V refers to the cross-sectional area of the bracings respectively.

Figure 71.2.3.1.aShear stiffness S_v of lacing of built-up members

The effective inertia I_{ef} of the laced member may be taken as:

$$I_{ef} = 0.5\, h_0^2 A_{cor}$$

To determine the design buckling resistance $N_{b,Rd}$ of the compressed chord, the buckling length given in Figure 71.2.3.1.b shall be used.

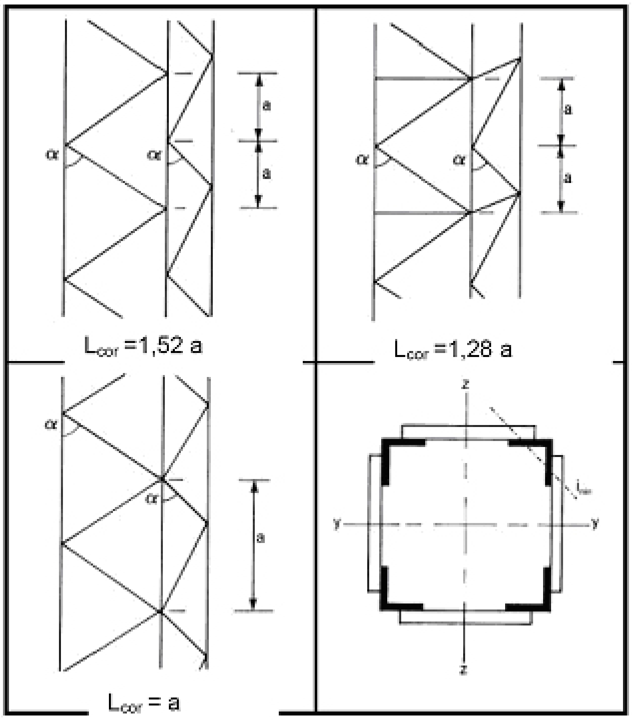


Figure 71.2.3.1.b Lacing on four sides and buckling length of chords L_{cor}

71.2.3.2. Battened members

Chords and batten plates, and the joints between them and the chords, should be checked under the stresses induced in the end panel and in span centre of the battened member. For the purpose of simplification, the maximum design axial force of the chord $N_{cor,Ed}$ may be combined simultaneously with the maximum shear force V_{Ed} (see Figure 71.2.3.2).

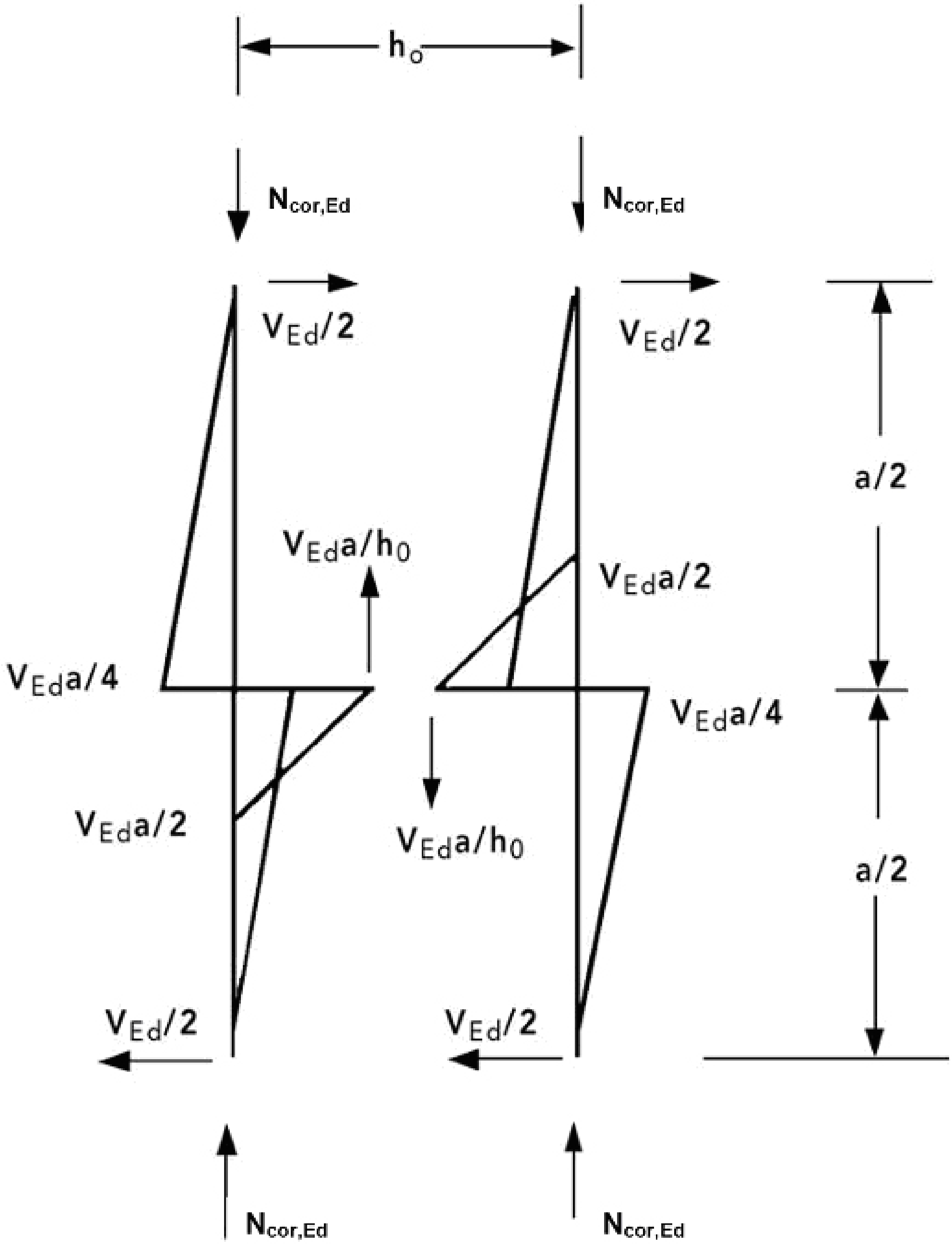


Figure 71.2.3.2 Moments and forces in an end panel of a battened built-up

The buckling length of the chord is the distance a between batten plates.

The shear stiffness of a battened built-up member should be taken as follow.

$$S_V = \frac{24EI_{cor}}{a^2\left[1+\frac{2I_{cor}}{nI_p}\frac{h_0}{a}\right]} \le \frac{2\pi^2 EI_{cor}}{a^2}$$

The effective second moment of area of battened built-up member may be taken as follow:

$$I_{ef}=0.5\,h_0^2\,A_{cor}+2\,\mu I_{cor}$$

in which:

I_{cor} is in plane second moment of area of one chord.

I_p is in plane second moment of area of one of batten.

n is the number of planes of lacing.

μ is the efficiency factor from Table 71.2.3.2.

Table 71.2.3.2. Efficiency factor μ

Criterion	Efficiency factor μ
$\lambda \geq 150$	0
$75<\lambda< 150$	$\mu=2-\frac{\lambda}{75}$
$\lambda \leq 75$	1.0
where $=\frac{L}{i_0}$; $i_0=\sqrt{\frac{I_1}{2A_{cor}}}$; $I_1=0.5\,h_0^2 A_{cor} + 2I_{cor}$	

Section 72. Triangular structures

72.1. General

Triangular structures, commonly known as lattices, are frequently used in steel construction, both as lightweight members (lightweight lattices) for covering warehouses or spaces with medium or large spans, and as heavier members, such as in bridges. If the triangular structure is gabled and is normally used to column the cover of any sort of warehouse, this structure is known as a truss. If the laced structure has parallel upper and lower chords, the structure is referred to as triangular girder or lattice girder; this type of structure is used in floor slabs in medium-or large-span buildings, in cover warehouses and in bridges.

The geometry of lacing may be very varied, as may the different profiles or sections of members that constitute the bars of the structure, which may be flat or spatial.

Depending on the type of join between nodes of triangular structures, they may be classified as hinged-node structures or stiff-node structures. This classification may influence the design process to be followed to determine stresses.

The scope of this section may be very wide, although the importance of compression members in the design and the checking of this type of structure is highlighted.

72.1.1. Determination of stresses

Stiff-node structures subjected to predominantly static loads in which the triangulation is regular and the angles formed between members are not very acute ($\alpha \leq 30°$) may be deemed hinged at their ends for the purpose of determining stresses, assuming that the potential stresses induced by the stiffness of the nodes are negligible.

The above simplification shall not apply if there are highly stiff members or the structure is statically indeterminate. In this case, it shall be necessary to carry out a rigorous calculation to determine the stresses in the members.

72.2. Checking members

The procedure provided for any prismatic member, as specified in Chapter IX, shall be used to check the members of lacing.

The buckling lengths given in the sections below shall be used to check members subjected to compression or to compression and bending.

72.3. Buckling length of members in the plane of the structure

For chord members generally the buckling length of braces may be taken as their real length.

If the joints between uprights and diagonals to the chords of the truss or girder form a suitable fixing, i.e. they exercise a given rotation restraint in the plane of the structure, the buckling length of such brace members may be taken as 0.9 L, where L is the real length between their nodes. This may not apply to diagonals or uprights dimensioned with angle.

A joint is deemed to have a suitable abutment if it is welded or, if it is bolted, if at least two bolts are used.

Performing a rigorous check on brace members in compression that are dimensioned with angle and held by a single face to the main bars or members (for example: lattice angle in built-up members held to the rounds) is complicated. The geometry of the angle may cause buckling by bending and torsion, and they are subjected to eccentric compression. Furthermore, it is not easy to precisely determine the buckling length to be taken into consideration. These effects may be taken into account in a simplified manner assuming an effective slenderness ratio λ_{ef} with the following value:

For buckling about v-v axis:

$$\lambda_{ef,v} = 0.35 + 0.70\ \lambda_v$$

For buckling about y-y, axis parallel to the flanges:

$$\lambda_{ef,y} = 0.40 + 0.70\ \lambda_y$$

For buckling about z-z axis:

$$\lambda_{ef,z} = 0.40 + 0.70\ \lambda_z$$

72.4. Buckling length of members out-of-plane members

A distinction should be made depending on whether there is transverse bracing perpendicular to the plane of the structure of the compressed chord.

72.4.1. Compressed chord with transverse bracing

When checking the compressed chord, the buckling length shall be the distance between two consecutive bracings.

If there is an intermediate node between bracings, such that the portion of the compressed chord is subject to two compression forces, one in each portion, N_1 and N_2, where $N_1 > N_2$, the buckling length shall be obtained by multiplying the distance between bracings by a coefficient β with the following value:

$0.75 + 0.25\, N_2 / N_1$

If there are various intermediate nodes between bracings, such that along the portion of the compressed chord in question there is a variation of the axial force applied, the buckling length may be calculated in accordance with section 70.4 (Columns subjected to concentrated loads along their directrix).

72.4.2. Compressed chord without transverse bracing

If there is no bracing on the compressed chord, the buckling length may be taken as the length of the entire chord. As there will be intermediate nodes causing a variation in the axial force along the chord, the buckling length may be estimated in consideration of the provisions of section 70.4.

For a more precise estimation of the buckling length of the compressed chord, the transversal stiffness of the brace members and their joint conditions must be taken into account.

72.4.3 Brace members

Members may be calculated with a buckling length equal to the real length of the part.

In the case of lattice uprights with K triangulation in which both portions of the upright are subject to axial forces N_1 and N_2, where $N_1 > N_2$, the upright shall be checked for buckling under the maximum axial compression force N_1, assuming a buckling coefficient β with the following value:

$0.75 + 0.25\, N_2 / N_1 \geq 0.5$

If a diagonal of length d subject to an axial compression force N crosses another diagonal of length d_t subject to an axial tension force N_t, being considered the crossing as a joint conditions, it shall be checked for buckling assuming the following coefficient β:

$$\sqrt{1 - 0{,}75\,\frac{N_t \cdot d}{N \cdot d_t}} \geq 0{,}5$$

72.5. Joints

Joints of members or parts in a node may be welded or bolted. Joints may be butt joints or made using blocks gusset plates.

In the case of spatial triangular structures of hollow section (space frame) the joints may be made using special parts to which the members in the node are bolted.

The joints shall be checked in accordance with the provisions of Chapter XIV. In particular, the provisions of Section 64 and Annex 9, as well as the provisions of Chapter XI on checking tubular joints for fatigue, shall apply when checking joints between members of tubular section.

Section 73. Profiled steel sheets

73.1. General

This section is intended to set out specific rules for designing and calculating profiles steel sheets made up of cold-formed plates and profiles. On account of their reduced thickness and preparation, the characteristics of this type of part are different to hot-rolled plates and profiles such as:

a) Partial modification of the yield strength.

b) Greater influence of instability phenomena.

c) Greater influence of dimensional tolerances.

d) Potential variation of transverse dimensions.

e) Specific joining methods.

f) Frequent use of test-based design methods.

g) Greater influence of corrosion protection.

h) Effect of provisional maintenance and construction loads.

This section covers the most important aspects, making the appropriate references to the other sections of this Code.

This section does not apply to cold-formed hollow sections manufactured in accordance with product standard UNE-EN 10219, which are covered by the remaining sections of this Code.

73.2. Scope

This section applies to plates and sections cold formed from galvanised and ungalvanised steel and manufactured in accordance with the standards given in the attached lists, which specify the basic yield strength and the ultimate tensile strength in N/mm^2, which must be used in calculations.

UNE-EN 10025-2 Hot-rolled products of structural steels. Part 2: Technical delivery conditions for non-alloy structural steels

S 235	235	360
S 275	275	430
S 355	355	490

UNE-EN 10025-3 Hot-rolled products of structural steels. Part 3: Normalized/normalized rolled weldable fine-grain structural steels.

S 275 N / NL	275	370
S 355 N / NL	355	470
S 420 N / NL	420	520
S 460 N / NL	460	540

UNE-EN 10025-4 Hot-rolled products of structural steels. Part 4: Thermomechanical rolled weldable fine-grain structural steels

S 275 M / ML	275	360
S 355 M / ML	355	450
S 420 M / ML	420	500
S 460 M / ML	460	530

UNE-EN 10346. Continuously hot-dip coated steel flat products.

S 220 GD+Z	220	300
S 250 GD+Z	250	330
S 280 GD+Z	280	360
S 320 GD+Z	320	390
S 350 GD+Z	350	420

UNE-EN 10149-2. Hot-rolled flat products made of high yield strength steels for cold forming. Part 2: Thermomechanically rolled steels.

S 315 MC	315	390
S 355 MC	355	430
S 420 MC	420	480
S 460 MC	460	520

UNE-EN 10149-3. Hot-rolled flat products made of high yield strength steels for cold forming. Part 3: Normalized or normalized rolled steels.

S 260 NC	260	370
S 315 NC	315	430
S 355 NC	355	470
S 420 NC	420	530

UNE-EN 10268. Cold-rolled steel flat products with high yield strength for cold forming.

HC 260 LA	240	340
HC 300 LA	280	370
HC 340 LA	320	400
HC 380 LA	360	430
HC 420 LA	400	460

UNE-EN 10346. Continuously hot-dip zinc coated low carbon steels strip and sheet for cold forming.

HX 260 LAD	265	350
HX 300 LAD	300	380
HX 340 LAD	340	410
HX 380 LAD	380	440
HX 420 LAD	420	470

UNE-EN 10346. Continuously hot-dip zinc-aluminium (ZA) coated steel strip and sheet. Technical delivery conditions.

S 220 GD+ZA	220	300
S 250 GD+ZA	250	330
S 280 GD+ZA	280	360
S 320 GD+ZA	320	390
S 350 GD+ZA	350	420

UNE-EN 10346. Continuously hot-dip zinc coated low carbon steels strip and sheet for cold forming.

DX 51 D+Z	140	270
DX 52 D+Z	140	270
DX 53 D+Z	140	270

If steel with an ultimate tensile strength greater than 550 N/mm^2 is used, the resistance and ductility of the joints must be demonstrated by testing.

The application limits in terms of design thicknesses are determined by the range used in the tests providing reliable results. Unless a design based on this method is used, the limit thicknesses shall be 0.45 mm and 15 mm.

A different thickness limitation may be imposed depending on the joining methods used.

73.3. Design thickness

The design thickness should consider the significant influence that may be exerted by the protective coating and the delivery tolerances. If this is equal to or less than 5 %, the design thickness shall be obtained by deducting the thickness of the zinc coating t_{mc} from the nominal thickness only.

$$t_{cor}=t_{nom}-t_{mc}$$

If the thickness tolerance is greater than 5 %, the above value must be corrected.

$$t_{cor}=(t_{nom}-t_{mc})(100-tol)/95$$

73.4. Modification of yield strength

Due to the process of cold deformation, in the corners and folds, there is an increase of the mechanical characteristics of the steel. This advantage may be used in some cases using an average yield strength f_{ya}:

$$f_{ya} = f_{yb} + (f_u - f_{yb})knt^2 / A_g$$

with the upper limit:

$$f_{ya} < (f_u + f_{yb}) / 2$$

in which:

A_g is the gross area of the section.

k is the experimental coefficient, 5 for roll forming and 7 for other folding methods.

n is the number of folds in the section of 90°.

The average yield strength f_{ya} may be used to check the sections in tension. When checking for concentrated loads, buckling and local buckling by shear, the basic yield strength f_{ya} must be used, and it must also be used in formulas relating to the interaction forces.

If the material is subjected to subsequent annealing or a heat treatment in which the temperature exceeds 580 °C for more than one hour, the basic yield strength f_{yb} must be used.

73.5. Terms and dimensions

The parts are of uniform section formed by flat members and small-radius curved fillets. The description considers the stability offered by a member in compression along the axis of the part:

a) Unstiffened member: flat member joined on one edge only to another flat member.

b) Stiffened member: Flat member joined on both edges to other members or stiffeners.

c) Multi-stiffened member: Stiffened member that also has intermediate stiffeners.

d) Sub-member: portion of a multi-stiffened member between stiffeners or edges.

To increase the capacity of members in compression, intermediate and edge longitudinal stiffeners are installed to increase the critical local buckling stress. These stiffeners may be straight or lipped, or may be made up of several folds. This section does not cover transverse stiffeners.

Section characteristics (area, second moments of area, radius of gyration, etc.) may be determined using conventional material resistance methods. These characteristics may be obtained more simply by using the linear method, in which the material of the section is deemed concentrated along the centreline of the section such that all of the members are replaced by straight or curved members, introducing the design thickness once the calculation corresponding to this centreline has been made. In this case, the inertia of flat members in relation to the axis parallel to itself shall be disregarded. This method makes it possible to precisely determine the length of each flat member to calculate its non-dimensional slenderness and its consequent effective width. Curved members shall not be subject to reduction.

If the internal radius of a fold is less than five times the thickness and one tenth of the length of the contiguous flat member, it may be assumed that the section is formed by square edges, without fillets, and the length of the flat members may be taken as the corresponding projection to the midpoints of the corners. This method is slightly conservative as it attributes a greater-than-real length to the flat members.

The length of each flat member is referred to as the straight width b_p.

Figure 73.5 Straight width b_p

73.6. Maximum width to thickness ratios

Application of the expressions in this section is limited to the width to thickness ratios shown in the attached figure, which shows the scope of the tests on which the calculation formulas are based.

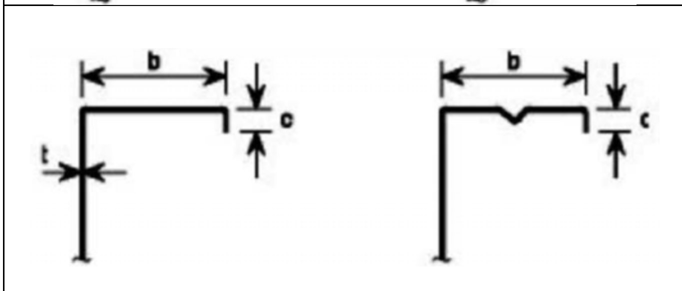
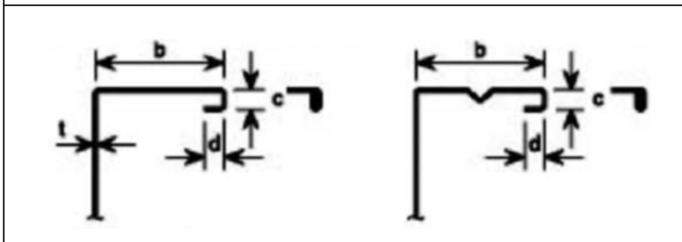
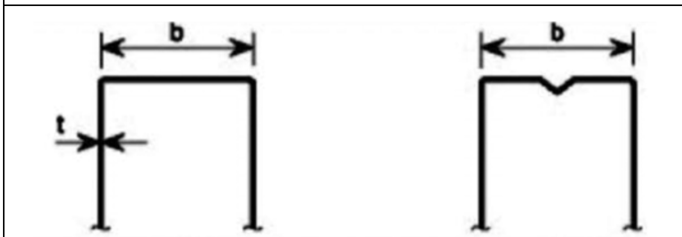
	b/t≤50
	b/t≤60 c/t≤50
	b/t≤90 c/t≤60 d/t≤50
	b/t≤500
	45°≤Φ≤90°

Figure 73.6 Maximum width to thickness ratios for application of the calculation expressions of this section

Stiffeners must provide sufficient stiffness to prevent their own local buckling, which means limiting their free length. On the other hand, they must be larger than a minimum size in order to stiffen the contiguous member. The sizes of stiffners should be within the following ranges 0.2≤c/b≤0.6

0.1≤d/b≤0.3

73.7. Flange curling

There is a tendency for members wide (in relation to depth) flanges subjected to bending to curve towards the neutral fibre. This distortion may be disregarded if its magnitude does not exceed 5 % of the depth.

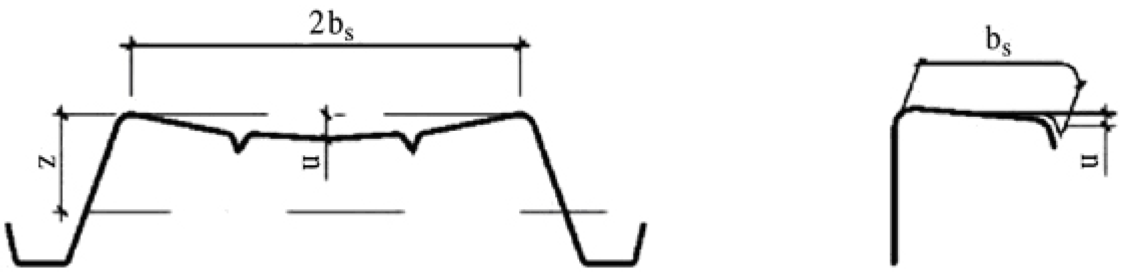


Figure 73.7 Flange curling (u)

The following expression may be used to calculate curling:

$$u = 2\frac{\sigma_a^2 b_s^4}{E^2 t^2 z}$$

where:

b_s is one half the distance between webs in box hat sections or, width in Z or C sections.

t is flange .

z is the distance of flange under consideration from the neutral fibre.

σ_a is the mean stress in the flange, calculated with gross area

73.8. Non-uniform distribution of stresses in flanges

The deformation restraint caused by shear stresses generated in the flanges of beams with low span/width ratios disturbs the uniform distribution of direct stresses in them. This phenomenon is described as “shear lag” in Section 21, which sets out the formulas used to evaluate its effects.

In order to simplify incorporation of the stress increase in the case of concentrated or distributed loads, the effective flange width, in tension or in compression, is given in the table below:

Table 73.8 Effective width for shear lag

L/ bs	30	25	20	18	16	14	12	10	8	6
Reduction	1.00	0.96	0.91	0.89	0.86	0.82	0.78	0.73	0.67	0.55

73.9. Direct stress local buckling

73.9.1. Introduction

The effect of compression stresses may reduce stiffness and resistance capacity by causing local buckling, both local and as section distortion. The effect of local buckling due to direct stresses may be taken into account by using the effective width in compressed flat members in accordance with the rules set out below.

The performance of the flat members that make up a section is determined by the post-critical resistance, which involves a redistribution of direct stresses equivalent to a maximum uniform stress $\sigma_{com,Ed}$ applied to a effective width ρb_p, the product of performance ρ and straight width b_p.

73.9.2. Sheet slenderness

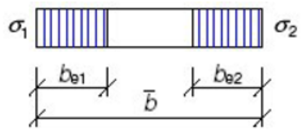
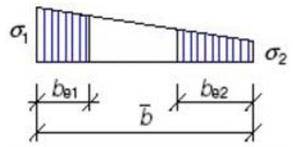
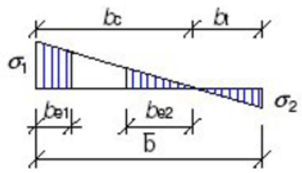
The use of effective widths determines certain new geometric characteristics that depend on the acting forces. Consequently, in this type of structure there is dependence between the stresses and the geometry if the magnitude of the compression stresses exceeds certain limits.

To characterise a member, the relative sheet slenderness $\bar{\lambda}_p$, calculated for a compression stress equal to the yield strength f_{yb}, which depends on its ideal critical local buckling stress σ_{cr}, is determined.

$$\bar{\lambda}_p = \sqrt{\frac{f_{yb}}{\sigma_{cr}}} = \frac{b_p}{t}\sqrt{\frac{12(1-\upsilon^2)f_{yb}}{\pi^2 E k_\sigma}} \cong 1{,}052\frac{b_p}{t}\sqrt{\frac{f_{yb}}{E k_\sigma}} \cong \frac{b_p/t}{28{,}4\varepsilon\sqrt{k_\sigma}}$$

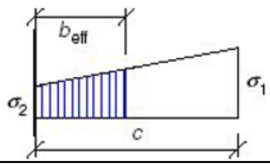
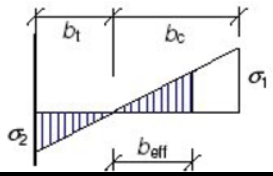
The local buckling coefficient k_σ can be obtained from Tables 73.9.2.a and 73.9.2.b.

Table 73.9.2.a. Compressed inner panels. Effective width

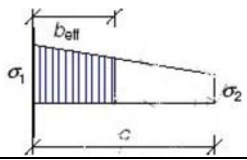
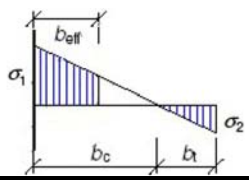
Distribution of stresses (positive compression)	Effective width (b_{ef})
	<u>Ψ=1:</u> $b_{ef}=\rho \bar{b}$ $b_{e1}=0.5b_{ef}$ $b_{e2}=0.5b_{ef}$
	<u>1>ψ≥0:</u> $b_{ef}=\rho \bar{b}$ $b_{e1}=\frac{2}{5-\psi}b_{ef}$ $b_{e2}=b_{ef}-b_{e1}$
	<u>ψ<0:</u> $b_{ef}=\rho b_c= \rho \bar{b}/(1-\psi)$ $b_{e1}=0.4b_{ef}$ $b_{e2}=0.6b_{ef}$

$\psi=\sigma_2 / \sigma_1$	1	1>Ψ>0	0	0>Ψ>-1	-1	-1>Ψ>-3
Local buckling coefficient k_σ	4.0	8.2/(1.05+Ψ)	7.81	$7.81-6.29\psi+9.78\psi^2$	23.9	$5.98(1-\psi)^2$

Table 73.9.2.b. Compressed panels with one free edge. Effective width

Distribution of stresses (positive compression)	Effective width (b_{ef})
	<u>1>ψ≥0:</u> $b_{ef}=\rho c$
	<u>ψ<0:</u> $b_{ef}=\rho b_c= \rho c/(1-\psi)$

$\psi=\sigma_2 / \sigma_1$	1	0	-1	
Local buckling coefficient k_σ	0.43	0.57	0.85	$0.57-0.21\psi+0.07\psi^2$

Distribution of stresses (positive compression)	Effective width (b_{ef})
	<u>1>ψ≥0:</u> $b_{ef}=\rho c$
	<u>ψ<0:</u> $b_{ef}=\rho b_c= \rho c/(1-\psi)$

$\psi=\sigma_2 / \sigma_1$	1	1>ψ>0	0	0>ψ>-1	-1
Local buckling coefficient k_σ	0.43	0.578/(ψ+0,34)	1.70	$1.7-5\psi+17.1\psi^2$	23.8

If the maximum stress $\sigma_{com,Ed}$ is less than f_{yb}/γ_{M0}, the effective plate slenderness will be used:

$$\overline{\lambda}_{p,red} = \overline{\lambda}_p \sqrt{\frac{\sigma_{com,Ed}}{f_{yb}/\gamma_{M0}}}$$

For the serviceability state, where the maximum compression stress is $\sigma_{com,Ed,ser}$, the following slenderness shall be used:

$$\overline{\lambda}_{p,ser} = \overline{\lambda}_p \sqrt{\frac{\sigma_{com,Ed,ser}}{f_{yb}}}$$

In these formulas, which make it possible to determine the local buckling coefficient and consequently the relative sheet slenderness (see section 20.7), the stress ratio ψ must be realistic, i.e. it must correspond to the definitive distribution of stresses occurring in the set of members that make up the section once the effective widths have been reduced. However, in flanges this ratio may be kept the same as in the initial section without reduction; iteration should be used in webs.

73.9.3. Effective width

In stiffened members, the ratio ρ in the most general case is as follows:

For $\overline{\lambda}_{p,red} \leq 0{,}5 + \sqrt{0{,}085 - 0{,}055\psi}$ ρ = 1.00

For $\overline{\lambda}_{p,red} \leq 0{,}5 + \sqrt{0{,}085 - 0{,}055\psi}$

$$\rho = \frac{1 - 0{,}055(3+\psi)/\overline{\lambda}_{p,red}}{\overline{\lambda}_{p,red}} + 0{,}18\frac{(\overline{\lambda}_p - \overline{\lambda}_{p,red})}{(\overline{\lambda}_p - 0{,}6)}$$

In unstiffened members:

For $\overline{\lambda}_{p,red} \leq 0.748$ ρ = 1.00

For $\overline{\lambda}_{p,red} > 0.748$ $\rho = \frac{1 - 0{,}188/\overline{\lambda}_{p,red}}{\overline{\lambda}_{p,red}} + 0{,}18\frac{(\overline{\lambda}_p - \overline{\lambda}_{p,red})}{(\overline{\lambda}_p - 0{,}6)}$

When determining geometric characteristics in serviceability limit state, the effective plate slenderness $\lambda_{p,red}$ in these formulas shall be replaced by $\lambda_{p,ser}$.

A reasonably conservative simplification involves calculating the geometric characteristics corresponding to axial and bending extreme cases with maximum stress equal to the yield strength f_{yb}. This simplification may be used when checking ultimate limit state with axial-bending interaction. The following values are obtained:

A_{ef}, effective area calculated for uniform stress f_{yb} by axial force.

W_{ef}, modulus calculated for the maximum compression stress f_{yb} by bending.

The effect of changing the centroid, which can be obtained by determining A_{ef}, should be considered, as in the typical sections indicated in the attached figures:

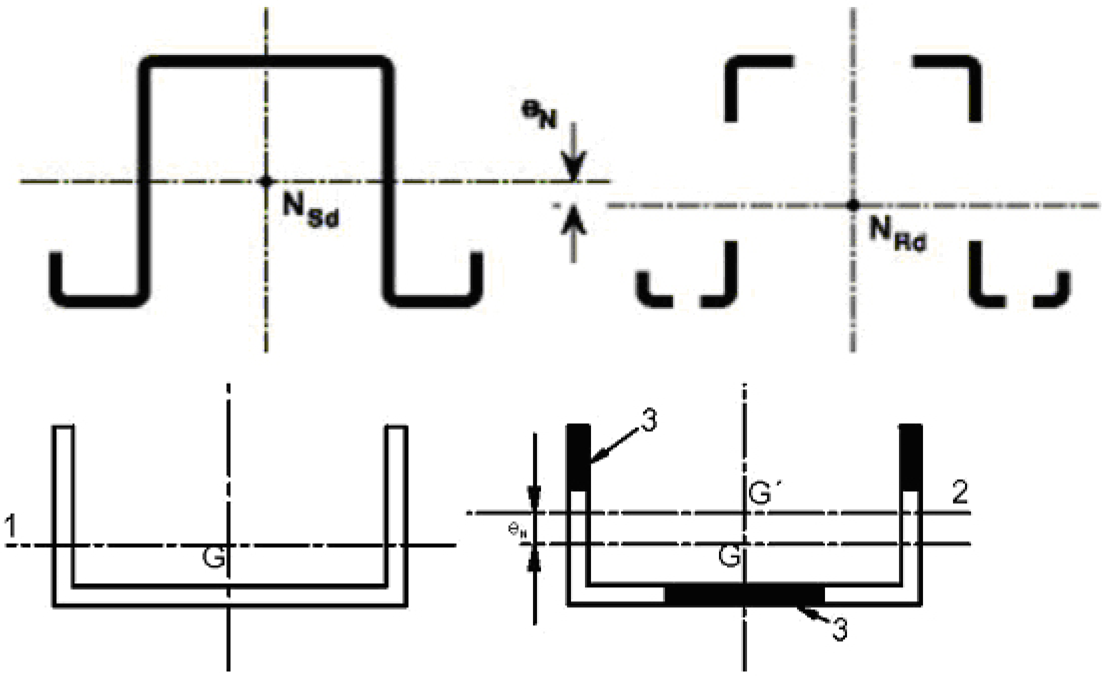


Figure 73.9.3 Effective cross-section under compression. Variation of the position of the barycentric axis of the section

73.10. Shear stress local buckling

The effect of shear stress local buckling shall be taken into account by limiting the shear capacity of the webs of the sections by means of shear stress local buckling resistance f_{bv}.

This resistance is based on the simple post-critical method and does not require the presence of transverse stiffeners. The formula for the shear capacity of a web is:

$$V_{b,Rd} = \frac{\frac{h_w}{sen\Phi} t f_{bv}}{\gamma_{M0}}$$

in which:

h_w is the web height between the midlines end of the flanges , .

φ is the slope of the web to the flanges.

t is the design thickness.

The value of f_{bv} depends on the shear slenderness of the web $\bar{\lambda}_w$ according to Table 73.10 below, in which the second column covers the existence of devices preventing local distortion, such as purlin brackets.

Table 73.10 hear I buckling strength f $_{b,v}V_{b,Rd}$

Relative slenderness of web	Web without stiffening at the column	Web with stiffening at the column
$\bar{\lambda}_w \leq 0.83$	0.58 f_{yb}	0.58 f_{yb}
0.83< $\bar{\lambda}_w$ <1.40	0.48 $f_{yb}/\bar{\lambda}_w$	0.48 $f_{yb}/\bar{\lambda}_w$
$\bar{\lambda}_w$ ≥40.1	0.67 $f_{yb}/\bar{\lambda}_w^{\,2}$	0.48 $f_{yb}/\bar{\lambda}_w$

The relative shear slenderness of the web $\bar{\lambda}_w$ shall be obtained, from the following:

For webs without longitudinal stiffening:

$$\bar{\lambda}_w = 0,346 \frac{S_w}{t}\sqrt{\frac{f_{yb}}{E}}$$

equivalent to the general expression, assuming that shear stress at the start of plastification is the same as the Von Mises criteria 0.58 f_{yb} and that the shear local buckling coefficient k_τ is 5.34, as there is no transverse stiffening,

$$\bar{\lambda}_w = \sqrt{0,58 f_{yb} / \tau_{cr}}$$

For webs with longitudinal stiffening that satisfy the following condition $\bar{\lambda}_w \geq 0,346 \frac{S_p}{t}\sqrt{\frac{f_{yb}}{E}}$,:

$$\bar{\lambda}_w = 0,346 \frac{S_d}{t}\sqrt{\frac{5,34 f_{yb}}{k_\tau E}}$$

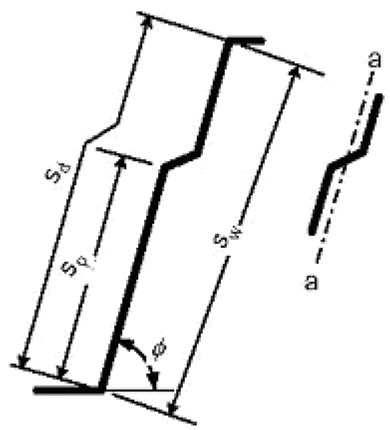


Figure 73.10 Geometric data for calculating relative slenderness of the web, with intermediate longitudinal stiffening. Longitudinally stiffened web.

With $k_\tau = 5{,}34 + \frac{2{,}10}{t}\left(\frac{\sum I_S}{S_d}\right)^{\frac{1}{3}}$

in which:

I_s is the second moment of area of the individual longitudinal stiffener about the axis a-a (Figure 73.10.

s_d is the total developed m slant height of the web.

s_p is the slant height of the largest portion of the web.

s_w is the slant height of the web.

73.11.Ultimate limit states

73.11.1. Section resistance

Profiled steel sheets may be checked using the applicable criteria in Chapter IX, covering interaction of stresses, with the geometric characteristics of the effective section for the maximum compression stress $\sigma_{com,Ed}$. If the section is class 3 in bending in the plane corresponding to its principal axis, section 73.11.2 may be applied. The following stress limitation criteria may also be used:

$$\sigma_{to,Ed} \leq f_{ya}/\gamma_{M0}$$

$$\tau_{tot,Ed} \leq \frac{f_{ya}/\sqrt{3}}{\gamma_{M0}}$$

$$\sqrt{\left(\sigma_{tot,Ed}^2 + 3\tau_{tot,Ed}^2\right)} \leq 1{,}1\frac{f_{ya}}{\gamma_{M0}}$$

in which:

$\sigma_{tot,Ed}$ is the sum of direct stresses.

$\tau_{tot,Ed}$ is the sum of shear stresses.

Both stresses are calculated in the least favourable axis in consideration of all of the stresses acting on the section in question:

$\sigma_{tot,Ed} = \sigma_{N,Ed} + \sigma_{My,Ed} + \sigma_{Mz,Ed} + \sigma_{w,Ed}$

$\tau_{tot,Ed} = \tau_{Vy,Ed} + \tau_{Vz,Ed} + \tau_{t,Ed} + \tau_{w,Ed}$

where:

$\sigma_{N,Ed}$ is the direct stress due to axial load, using the effective section.

$\sigma_{My,Ed}$ is the direct stress due to bending $M_{y,Ed}$, using the effective section.

$\sigma_{Mz,Ed}$ is the direct stress due to bending $M_{z,Ed}$, using the effective section.

$\sigma_{w,Ed}$	is the direct warping-torsion stress, using the gross section.
$\tau_{Vy,Ed}$	is the shear stress due to shear $V_{y,Ed}$, using the gross section.
$\tau_{Vz,Ed}$	is the shear stress due to shear $V_{z,Ed}$, using the gross section.
$\tau_{t,Ed}$	is the shear stress due to uniform torsion, with the gross section.
$\tau_{w,Ed}$	is the shear stress due to warping torsion, with the gross section.

Unless demonstrated by testing, the plastic redistribution of bending forces between different portions of apart shall not be admitted.

If bending causes the start of plastification in the axis in tension before the axis in compression, the plastic reserve of the zone in tension may be used, without any limitation of the deformation ε, establishing equilibrium with the zone in compression, in which f_{yb} is reached in the edge axis.

Figure 73.11.1 Plastic distribution stresses in the portion in tension

73.11.2. Resistance of class 3 sections

In resistance checks, an improved bending moment resistance $M_{c,Rd}$ may be used if the greater slenderness λ of the members of the section is such that the section belongs to a class 3,.

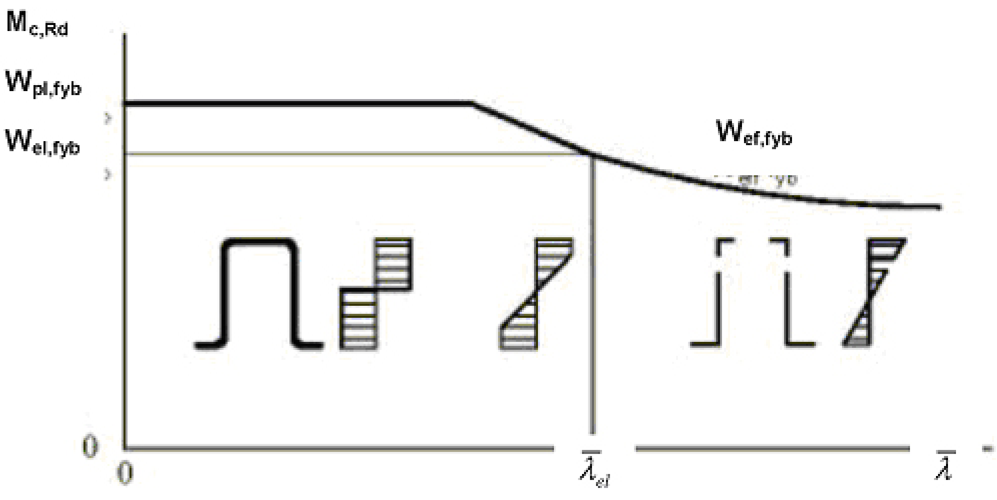


Figure 73.11.2 bending moment resistance as function of slenderness

$$M_{c,Rd} = f_{yb} [W_{el} + (W_{pl} – W_{el})4(1-\bar{\lambda} / \bar{\lambda}_{el})]/ \gamma_{M0}$$

In this expression, $\bar{\lambda}$ is the slenderness of the member with the largest $\bar{\lambda} / \bar{\lambda}_{el}$ ratio, which should be calculated with the stress corresponding to f_{yb} i.e. equivalent to $\bar{\lambda}_p$.

The slenderness $\bar{\lambda}_{el}$ used for this interpolation is as follows:

$$\bar{\lambda}_{el} = 0.5 + \sqrt{0,25 - 0,055(3+\psi)}$$

for flat members with two stiffened edges. For cantilevered members with an unstiffened free edge, $\bar{\lambda}_{el}$ =0.673.For members stiffened on their free edge or at an intermediate point, $\bar{\lambda}_{el}$ =0.65, including in this case the compressed flanges of purlins.

73.11.3. Buckling resistance

The members of profiled steel sheets shall be checked for buckling using the corresponding formulas in Section 35, depending on their classification. For class 4 sections, the geometric characteristics A_{ef} and W_{ef}, calculated in accordance with the above guidelines, shall be used.

Given the reduced thickness of this type of section, torsional stability tends to be limited, on account of which torsional buckling and torsional-flexural buckling must be checked in sections particularly susceptible to this phenomenon, such as sections whose centroid is not the same as the shear centre, as shown in Figure 73.11.3.

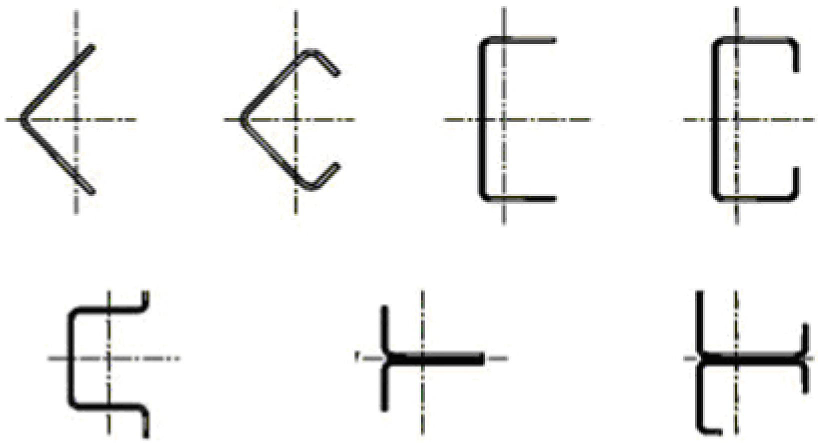


Figure 73.11.3 Monosymmetric cross sections susceptible to torsional-flexural buckling

Section 73.11.4 sets out the buckling curves to be used, depending on the type of cross-section. The non-dimensional slenderness of the member shall be obtained in consideration of the lower elastic critical axial force N_{cr}.

$$\bar{\lambda} = \sqrt{\frac{A_{ef} f_y}{N_{cr}}}$$

in which: $N_{cr} = \min (N_{cr,F}, N_{cr,T}, N_{cr,TF})$

where:

$N_{cr,F}$ is the elastic critical axial buckling force by bending.

$N_{cr,T}$ is the elastic critical axial buckling force by torsion.

$N_{cr,TF}$ is the elastic critical axial buckling force by bending and torsion.

Provisions of sections 35.1.3 and 35.1.4 shall be used to obtain these values.

In front or cover structures in which the purlins are attached to corrugated sheet using self tapping screws on one of their flanges, a continuous cross-bracing may be deemed to exist if the following is true:

$$S \leq S_{ch}$$

where S is the required shear stiffness of the purlin, which can be determined using the following expression:

$$s = \left(EI_w \frac{\pi^2}{L^2} + GI_t + EI_z \frac{\pi^2}{L^2} 0,25h^2 \right) \frac{70}{h^2}$$ h is purlin height

and S_{ch} is the stiffness provided by the sheet of thickness t, the height of the rib h_w, the gap between the purlins s and b_{fal} is the total uninterrupted length of the skirt (i.e. a multiple of the gap s) which may be determined using the following expression:

$$S_{ch} = 1000\sqrt{t^3}\,(50 + 10\sqrt[3]{b_{fal}})\,\frac{s}{h_w}$$ (in N), (all dimensions in mm)

If the fixings are made in alternate ribs, the stiffness of the sheet shall be reduced by one fifth.

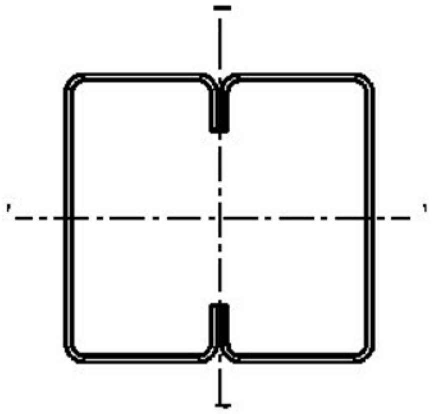
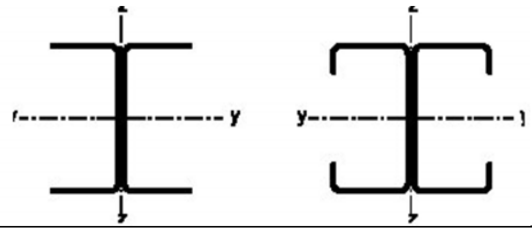
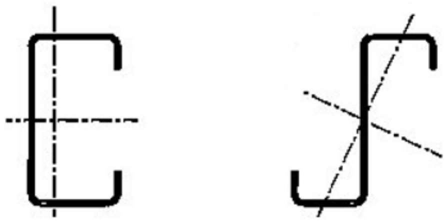
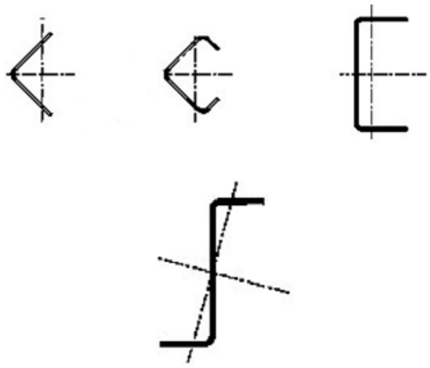
Members subjected to flexural compression may be checked for instability using the following interaction formula:

$$\left(\frac{N_{Ed}}{N_{b,Rd}} \right)^{0,8} + \left(\frac{M_{Ed}}{M_{b,Rd}} \right)^{0,8} \leq 1$$

73.11.4. Buckling curves

Table 73.11.4 sets out the buckling curves to be used for the different types of section in question.

Table 73.11.4 Appropriate buckling curves for various types of cross-sections

Type of cross-section		Buckling about axis	Buckling curve
	If f_{yb} is used	Any	b
	If f_{ya} is used *)	Any	c
		y-y z-z	a b
		Any	b
	or other cross-section	Any	c
*) The average yield strength f_{ya} should not be used unless A_{ef} (effective area)=A_g (gross area)			

73.12. Serviceability limit states

73.12.1. General

When checking serviceability limit state, the general requirements shall apply with the specific additional details set out in this section.

The geometric properties of the effective reduced section shall be obtained in accordance with 73.9.3.Given that these vary with the stress from the value $\overline{\lambda}_{p,ser} \geq 0.673$, the members consequently have variable geometry and inertia, and for the purpose of simplicity a fictitious second moment of area obtained using the expression:

$$I_{fic} = I_g - \frac{\sigma_g}{\sigma}(I_g - I_{\sigma,ef})$$

where:

I_g is second moment of area of the gross section.

σ_g is the serviceability maximum compression stress in service calculated with I_g.

σ is the maximum compression stress $\sigma_{com,E,ser}$ in the span.

$I_{ef,\sigma}$ is second moment of area of the effective cross section corresponding to σ.

73.12.2. Plastic deformation

In structures that admit global analysis based on tests, a plastic redistribution may occur in serviceability state, which must be taken into account. For this purpose, the verification of internal columns of continuous beams with bending and reaction must not exceed 0.9 the design resistance value.

73.12.3. Deflections

The deflections in purlins and enclosures must be limited so as not to adversely affect the water tightness, insulation or appearance of the building.

This shall be determined using the linear calculation with real geometric properties, which may be obtained using the simplification given in 73.12.1In continuous systems of purlins with overlaps or bushings, provision must be made for the increase in deformation due to slipping of bolts in joints.

A value of l/200 of the span length may be used as the deflection limit for variable short-term overloading.

73.13. Joints

73.13.1. General

This section covers joints in cold-formed steel beams and plates, galvanised or otherwise, as well as their splices and reciprocal connections to form built-up members.

The general principles are as set out in the sections of Chapter XIV, the scope being extended here in to thicknesses of less than 4 mm.

If steel with an ultimate tensile strength greater than 550 N/mm^2 is used, the resistance and ductility of the joints must be demonstrated by testing.

The joining means covered in this Code, for which calculation formulas are provided, are mechanical fixings (self-tapping bolts and conventional bolts) and welding (spot, resistance or fusion, and electric arc).Other joining means may be used with test-based resistance capacity values complying with the requirements of EN 1993-1-3.

Given that in this type of structure local stability is the key in the design, it must be guaranteed that the joints between members subjected to compression are made through the effective part of the section, taking into account potential local eccentricities.

The resistance capacity of joints and splices in tension must be at least half that of the net section. In the case of seismic-resistant design, the ductility requirement requires it to be 20 % greater than that of the net section.

In members subjected to compression, regardless of the magnitude of the force, the joint must be able to resist their buckling capacity. This ensures that, in the event of failure, the member will fail before the joint or splice.

73.13.2. Forces in joints and splices

A method equivalent to determining second-order forces, which are added to those acting on a joint or splice, consists in taking into account the effect of an additional bending force ΔM_{Ed} and an additional shear force ΔV_{Ed}, obtained using the following formulas:

$$\Delta M_{Ed} = N_{Ed}\left(\frac{1}{\chi}-1\right)\frac{W_{ef}}{A_{ef}}$$

$$\Delta V_{Ed} = \frac{\pi}{L} N_{Ed}\left(\frac{1}{\chi}-1\right)\frac{W_{ef}}{A_{ef}}$$

in which:

A_{ef}, W_{ef} are the values of the effective section of the member.

L is length.

χ is the buckling reduction coefficient in the least favourable buckling plane.

N_{Ed} is the design axial force.

73.13.3. Joints with mechanical fixing

Joints using self-tapping screws shall comply with standard UNE-EN ISO 10666 “Drilling Screws with Tapping Screw Threads. Mechanical and Functional Properties” and the remaining specific standards applicable to their specific geometry (hexagonal, countersunk, roundhead, etc. in UNE-EN ISO 15480 to 15483). Conventional bolts shall comply with the requirements of Section 29.

The arrangement of fixing members should be so as to enable assembly and subsequent maintenance, and must observe the reciprocal and edge distances indicated in the calculation formulas.

A general principle of these joints is that the fail should be ductile, i.e. the shear capacity of the screw shall be greater than that of any other failure mode.

73.13.3.1. Self-tapping screws

This type of joint is used frequently to attach cover and front sheets to purlins, and to connect said sheets to each other along their longitudinal edges and transverse overlaps (perpendicular to the direction of the fretwork).They are also used to connect simple bars together to form built-up parts.

The range of diameters tends to vary between 3 and 8 mm. Both self-tapping and self-drilling screws must comply with the assembly instructions given by the manufacturer concerning placement. The tightening torque applied must be slightly higher than that necessary to form the thread and less than the torsion breaking torque of the screw head. This threading torque must be less than two-thirds of the thread stripping torque or head-shear torque.

The distances to free edge e_1 and reciprocal distances between fixings in both directions p_1 and p_2 must be greater than three nominal diameters. In the transverse direction, the minimum distance e_2 to the edge is one and a half times the diameter.

A common load case is wind suction imposing tensile loads on fixing between sheets and purlins. For application of the formulas cited below to be valid, the thinnest sheet must have a thickness of between 0.5 and 1.5 mm, and the thickness of the thickest sheet must be greater than 0.9 mm in all cases.

The screw head must always be in contact with the thinnest sheet. They must be provided with a washer stiff enough to mobilise the resistance capacity of the joint for two typical types of failure:

a) By extraction or ripping of the threaded part, for example in purlin fixing in which the thread is pulled out

b) By perforation or punching of the sheet in contact with the screw head, for example lifting with ripping of the sheet fixed to the purlins around the screw head or washer (pull through)

The extraction capacity $F_{o,Rd}$ depends on the thread pitch in relation to the thickness of the sheet affected t_{sup}.

For $\frac{t_{\sup}}{s} < 1$ $$F_{o,Rd} = 0,45\, d t_{\sup} f_{u,\sup} / \gamma$$

For $\frac{t_{\sup}}{s} \geq 1$ $$F_{o,Rd} = 0,65\, d t_{\sup} f_{u,\sup} / \gamma_{M2}$$

The perforation capacity depends on the diameter of the screw head or the washer d_w, the thickness t of the sheet affected and the nature of the load.

For static actions: $$F_{p,Rd} = d_w t f_u / \gamma_{M2}$$

For actions that include wind: $$F_{p,Rd} = 0{,}5 d_w t f_u / \gamma_{M2}$$

The layout of covers must be as uniform and regular as possible, although on the edges of eaves and walls it is advisable to double the number of fixings along one tenth of the dimension in question to provide for the effects of a local wind amplification. Another best practice is to provide transverse cover sheet overlaps on a double purlin or with an oversized upper flange.

The placement of fixings shall only be valid for structural purposes if they are located in the bottom of the trough. Crest fixings may only be used for the purpose of watertightness. If non-centred fixings are used, at half-width of the trough, the perforation capacity must be reduced by 10 %, and if two fixings are placed in each trough, one of them shall be reduced by 30 % (see Figure 73.13.3.1).

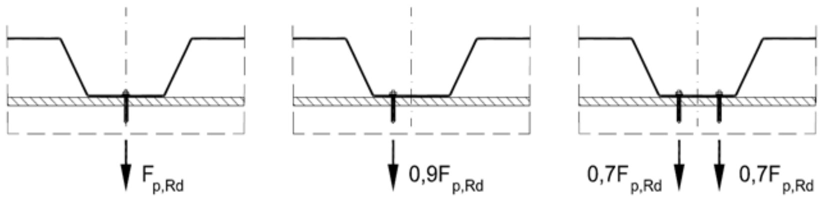


Figure 73 13.3.1 Reduction of pull through resistance due to position of fasteners

The tensile strength of the screw $F_{t,Rd}$ shall be guaranteed by the Manufacturer and must exceed the extraction resistance $F_{o,Rd}$ and perforation resistance $F_{p,Rd}$. It is advisable for extraction resistance to exceed perforation resistance to prevent abrupt tearing if the cover is subjected to upward loads.

For shear stresses in the plane of the sheet, the flattening resistance of the fixing against the sheet depends on the thickness ratio:

a) If both thicknesses are equal:

$$F_{b,Rd} = \alpha f_u d t / \gamma_{M2} \qquad \text{in which } \alpha = 3{,}2(t/d)^{0{,}5} \le 2{,}1$$

b) If the larger thickness t_1 is such that $t_1 \geq 2.5t$, where t is less than 1 mm, the above case shall apply.

c) If the larger thickness t_1 is such that $t_1 \geq 2.5t$, but $t \geq 1$ mm, the following value shall be taken directly α=2.1

d) For intermediate cases, where $t < t_1 < 2.5t$, α shall be obtained by interpolation.

The shear strength of the screw $F_{v.Rd}$ guaranteed by the Manufacturer must be 20 % greater than the flattening strength $F_{b,Rd}$.

The sum of individual resistances of screws must also be 20 % higher than net section resistance $F_{n,Rd}$.

$$F_{n,Rd} = A_{net} f_u / \gamma_{M2}$$

In flats and strips in tension, it is recommended that the above value not be less than the elastic capacity of the gross section to ensure plastification of the bar before the joints fail.

$$F_y = Af_y / \gamma_{M0}$$

In the event of combined tension and shear forces, the following expression shall apply:

$$\left(\frac{F_{t,Ed}}{\min(F_{p,Rd}, F_{0,Rd})} \right) + \left(\frac{F_{v,Rd}}{\min(F_{b,Rd}, F_{n,Rd})} \right) \leq 1$$

73.13.3.2. Conventional bolts

Bolted joints are frequently used to fix front or cover purlins to brackets and to connect purlins forming continuous-beam systems. They are also used to join the extremities of purlins near to the columns on lintels, using inclined struts or props in the lower flange of the lintels to prevent lateral buckling therein.

The thickness of the thinnest sheet should be less than 3 mm and equal to or greater than 0.75 mm; the thickness of the thickest sheet t_1 has no impact.

The reciprocal distances between screws must be larger than three diameters both in the direction of the force and in transverse direction. One diameter is required for the front edge e_1, and one and a half diameters for the side edges e_2.

These joints usually include a clearance between the nominal diameter d and the diameter of the hole d_0, to facilitate assembly. Screws from metric M6 may be used, although the most advantageous diameter is M-16, with a hole d_0 = 18 mm for C and Z purlins.

The quality of the screw material may be 4.6 to 10.9, but the pre-loading effect of high-strength screws may not be taken into account.

Screw calculations in tension and shear are determined by the provisions of Chapter XIV. No pull-through is foreseeable when using standard sets of nuts and washers.

Shear resistance must be checked to prevent sheet flattening or net section failure. This latter suffers a comparatively more unfavourable reduction than in self-tapping screws.

The flattening strength of the sheet is:

$$F_{b,Rd} = 2,5\ \alpha_b k_t f_u d t / \gamma_{M2}$$

in which:

$$\alpha_b = \min\left(\frac{e_1}{3d}, 1 \right)$$

$k_t = (0,8t + 1,5)/2,5$ for $0.75 \le t \le 1.25$

$k_t=1$ for $t > 1.25$

The resistance in the net section is:

$$F_{n,Rd} = \left(1 + 3r\left(\frac{d_0}{u} - 0,3\right)\right)\frac{A_{net} f_u}{\gamma_{M2}}$$

in which:

r = number of screws in the section/total number of screws.

$u = \min(e_2, p_2)$

where the variables set out in the formulas have the same meaning as in Chapter XIV.

73.13.4. Spot welds

Joints between members, galvanised or otherwise, up to 4 mm thick, may be made by spot welds, provided that the thinner connected parts is not more than 3 mm thick.

Spot welds may be either fusion or resistance welding. Unless pre-production tests are carried out, the design diameter of a spot weld shall be:

For fusion welding	$d_s=0.5t+5$	(in mm)
For resistance welding	$d_s=5t^{0.5}$	(in mm)

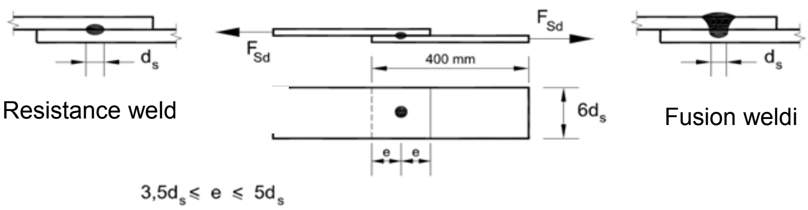


Figure 73.13.4 Test specimen for shear tests of spot welds

The distance between the last spot-weld and the front edge e_1 shall be between two and six times the design diameter d_s. The distance between a row of spots and the parallel edge e_2 shall be less than four times the design diameter d_s.

The reciprocal distances between spots must be larger than three diameters and less than 8 for p_1 and six diameters for p_2.

This type of joint is only suitable for transmitting forces in the plane of the sheets. The shear capacity of each spot $F_{v,Rd}$ must be 25 % greater than the tearing or flattening capacity and the sum of them must be 25 % greater than the net section resistance capacity $F_{n,Rd}$.

There are four failure types:

a) Shear: $F_{v,Rd} = (\pi / 4) d_s^2 f_u / \gamma_{M2}$

b) Net section: $F_{net,Rd} = A_{net} f_u / \gamma_{M2}$

c) End failure: $F_{tb,Rd} = 1{,}4 e_1 f_u / \gamma_{M2}$

d) Flattening and tearing if $t \le t_1 \le 2.5\,t$: $F_{net,Rd} = 2{,}7\sqrt{t}\, d_s f_u / \gamma_{M2}$

If $t_1 > 2.5t$, the same formula shall apply with the following upper limits:

$$F_{tb,Rd} < 0{,}7 d_s^2 f_u / \gamma_{M2} \qquad F_{tb,Rd} < 3{,}1 d_s t f_u / \gamma_{M2}$$

73.13.5. Lap welds

Members up to 4 mm thick may be joined using the electric arc method in accordance with the specifications of this section. The throat thickness should be chosen such that the resistance of the joint is determined by the thickness of the thinnest sheet. If a throat thickness equal to the thickness of the thinnest sheet is chosen, the above condition is deemed satisfied automatically.

73.13.5.1. Fillet welds

The design resistance of a fillet welded connection, parallel to the direction of the force, depends on the gap between them, coinciding with the width b of the welded flat or strip.

If the length of the chord is such that $L_{w,s} \le b$:

$$F_{w,Rd} = t L_{w,s}\left(0{,}9 - 0{,}45\frac{L_{w.s}}{b}\right)\frac{f_u}{\gamma_{M2}}$$

If the length of the chord is: $L_{w,s} > b$:

$$F_{w,Rd} = 0{,}45\, t b f_u / \gamma_{M2}$$

For an end fillet where $L_{w,s} \le b$:

$F_{w,Rd} = t L_{w,e}\left(1 - 0{,}30\frac{L_{w.e}}{b}\right)\frac{f_u}{\gamma_{M2}}$ where $L_{w,e}$ is the length of the end fillet weld

The centroid should be determined in advance in sets of side and frontal chords, and the forces transmitted by the member must be referred to it. The effective lengths of the chords shall match the geometric lengths, including end returns, with no reduction for start and termination of the weld.

Weld chords whose length is less than 8 times the thickness of the thinnest sheet shall not be deemed structural weld chords

73.13.5.2. Spot arc welding

As in fusion or resistance spot welding, covered in 73.13.4, only shear forces may be transmitted in the plane of the sheet. The total thickness of the sheets must not exceed 4 mm and the design interface diameter d_s must not be less than 10 mm.

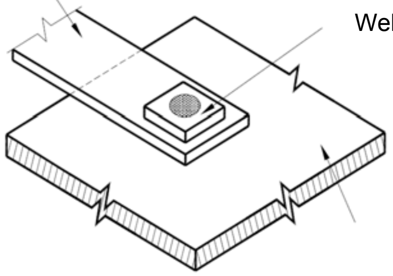


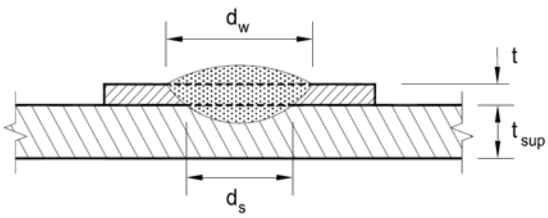
Figure 73.13.5.2.a Arc spot weld with weld washer

The formulas to be applied depend on the contact area between sheets and the peripheral area, along the edge, characterised by the peripheral diameter d_p. Both values are obtained from the visible diameter d_w obtained in the outermost weld washer or sheet.

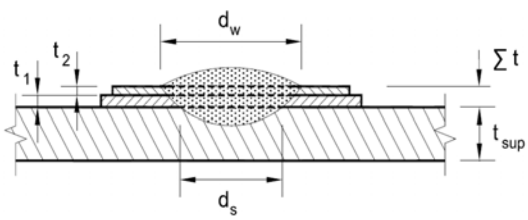
$d_p = d_w - t$ for two sheets

$d_p = d_w - 2\Sigma t$ for multiple connected sheets or parts of total thickness

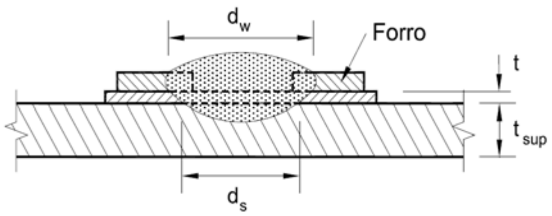
$d_s = 0.7d_w - 1.5\Sigma t > 0.55d_w$



a) Single connected sheet ($\Sigma t = t$)



a) Two connected sheets ($\Sigma t = t_1 + t_2$)



a) Single connected sheet with weld washer

Figure 73.13.5.2.b Arc spot welds. The distance to the end free edges of the last spot weld must be sufficient to prevent local flattening, checking that:

$$e_{\min} \geq 2{,}1 \frac{F_{w,Ed}}{t \dfrac{f_u}{\gamma_{M2}}}$$

where $F_{w,Ed}$ is the design shear resistance of the spot weld, which must be less than:

$$F_{w,Rd} = 0{,}625(\pi / 4) d_s^2 f_{uw} / \gamma_{M2}$$

where f_{uw} is the ultimate tensile strength of the material of the electrode.

The value of $F_{w,Rd}$ is limited by the perimeter resistance given by the following cases:

a) If $\dfrac{d_p}{\sum t} \leq 18\sqrt{\dfrac{420}{f_u}}$ $\qquad F_{w,Rd} = 1{,}5 d_p \sum t f_u / \gamma_{M2}$

b) If $18\sqrt{\dfrac{420}{f_u}} < \dfrac{d_p}{\sum t} \leq 30\sqrt{\dfrac{420}{f_u}}$: $\qquad F_{w,Rd} = 27\sqrt{\dfrac{420}{f_u}} \left(\sum t\right)^2 \dfrac{f_u}{\gamma_{M2}}$

c) If $\dfrac{d_p}{\sum t} \geq 30\sqrt{\dfrac{420}{f_u}}$: $\qquad F_{w,Rd} = 0{,}9 d_p \sum t f_u / \gamma_{M2}$

Elongated slot welds of length L_W have a capacity limited by the lesser of the following formulas:

In sheet contact: $\qquad F_{w,Rd} = \left(\left(\dfrac{\pi}{4}\right) d_s^2 + L_w d_s\right) 0{,}625 \dfrac{f_{uw}}{\gamma_{M2}}$

In the perimeter surface: $F_{w,Rd} = \left(0{,}5 L_w + 1{,}67 d_s\right) \sum t \dfrac{f_u}{\gamma_{M2}}$

Section 74. Meshed fabrics

74.1. Tubular structures

Tubular profiles are manufactured with steels similar to those used for other types of steel profile, on account of which in principle there is no difference between them; the mechanical and resistance properties are obtained using standard parameters. Chapter VI and sections 28.2 and 28.3 cover hot-rolled and cold-formed hollow profiles respectively. With regard to structural analysis, the provisions of Chapter V shall apply, along with the provisions of this chapter regarding the analysis of triangular or lattice structures. Furthermore, the dimensioning and checking of structural members of hollow section shall be carried out in consideration of the ultimate limit states, covered in Chapter IX, and the serviceability limit states, covered in Chapter X.

Moreover, when designing meshes in general or tubular structures, whether spatial or flat, it is important to consider the performance of the nodes from the beginning. Section 64 of this Code sets out the principles and rules for properly dimensioning joints between members of hollow section. Furthermore, Chapter XI specifies construction details of joints in tubular structures liable to be analysed for fatigue.