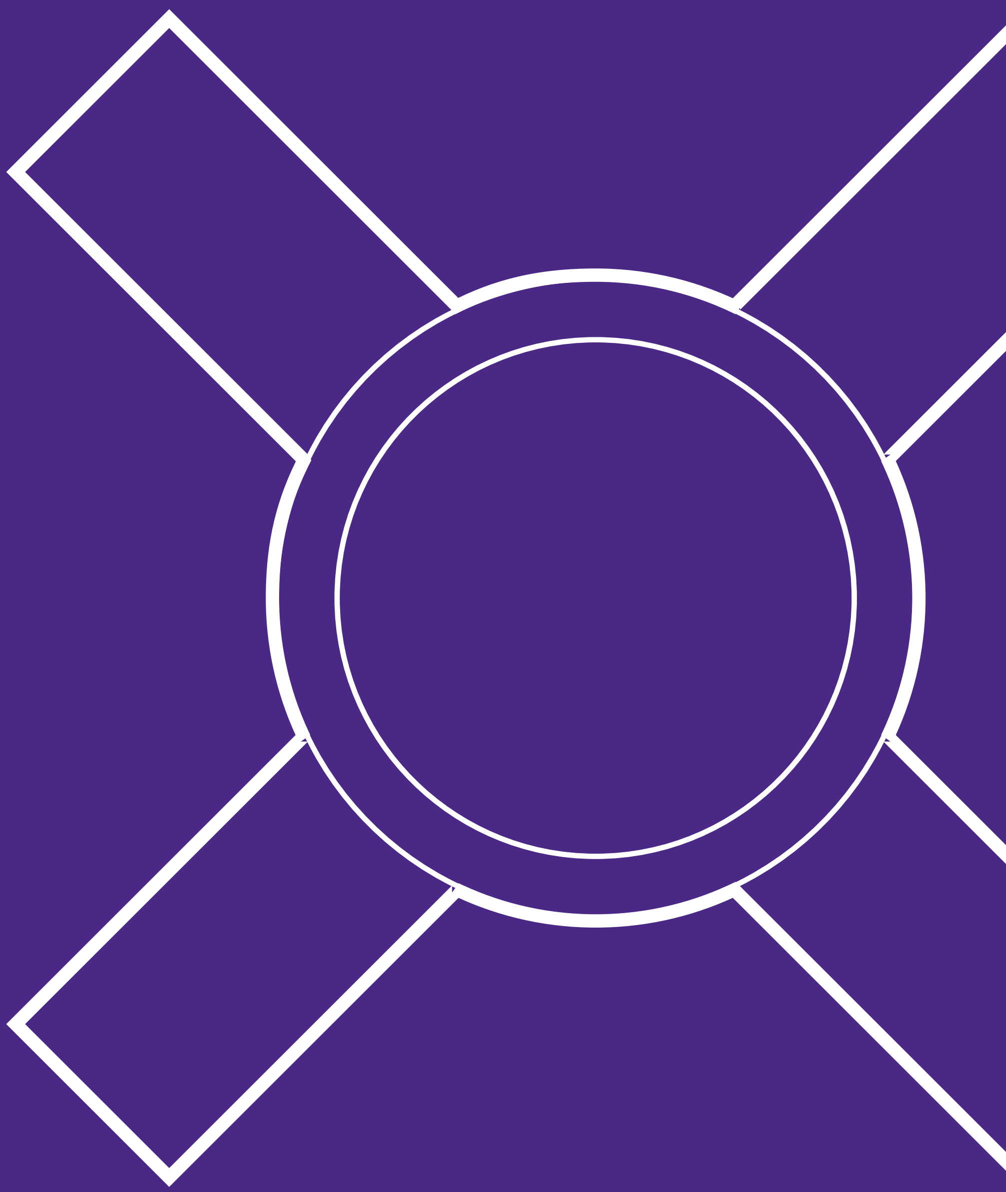
Corus Tubes

Design of SHS welded joints

Structural & Conveyance Business



Design of SHS welded joints

Contents

1. Introduction

In construction with structural hollow sections the members are generally welded directly to each other and, as a result, member sizing has a direct effect on both the joint capacity and the cost of fabrication. In order to obtain a technically secure, economic and architecturally pleasing structure, both the architect and design engineer must, from the very beginning, be aware of the effects that their design decisions will have on the joint capacity, the fabrication, assembly and the erection of the structure.

Structural hollow sections have a higher strength to weight ratio than open section profiles such as I-, H- and L- sections. They also require a much smaller weight of protection material, whether this is a fire protection or corrosion coating, because of their lower external area.

A properly designed steel construction using structural hollow sections will nearly always be lighter in terms of material weight than a similar construction made with open section profiles and, although structural hollow sections are more expensive than open section profiles on a per tonne basis, the overall weight saving of steel and protective coatings will very often result in a much more cost effective construction.

This publication has been produced to show how the joint capacity of staticaly loaded joints can be calculated and how it can be affected by both the geometric layout and the sizing of the members.

Considerable international research into the behaviour of structural hollow section (SHS) welded joints for lattice type constructions has enabled comprehensive design recommendations to be developed which embrace the large majority of manufactured structural hollow sections.

These design recommendations have been developed by CIDECT (Comité International pour la Développement et l'Étude de la Construction Tubulaire) and the IIW (International Institute of Welding) and, as a result, have gained considerable international recognition and acceptance. They have been used in a series of CIDECT Design Guides [1,2] and are now incorporated into Eurocode 3 : Annex K.[3]

The joint capacity formulae, reproduced in section 5, were developed and are presented in a limit states form and are therefore fully compatible with the requirements of BS 5950 : Part 1 [4] and Eurocode 3.

A software program [5], called CIDJOINT, has been developed by CIDECT for the design of most of the joints described in this design publication. The CIDJOINT design program requires MS-Windows version 3.x (or higher).

The design recomendations can be used with Corus Tubes Celsius® hot finished hollow sections to EN 10219 [6, 7], cold formed Hybox® 355 hollow sections to EN 10219 [8, 9] and cold formed Strongbox® 235 hollow sections to Corus Tubes specification TS30 [10]

1.1 Production specification

Corus Tubes produces four types of hollow section: Celsius® 275, Celsius® 355, Hybox® 355 and Strongbox® 235.
Celsius® hot finished structural hollow sections are produced by the Corus Tubes Structural & Conveyance Business. They are availble in two grades Celsius® 275 and Celsius® 355, which fully comply with EN 10210 S275J2H and EN 10210 S355J2H respectively. All Celsius® hot finished structural hollow sections have an improved corner profile of 2T maximum. For full details see Corus Tubes publication CTO6.
Hybox® 355 and Strongbox® 235 cold formed hollow sections are produced by Corus Tubes Cold Form Business. Hybox® 355 fully complies with EN 10219 S355J2H. Strongbox® 235 is in accordance with the Corus Tubes publication CTO5. The chemical composition and mechanical properties of these products, are given below.

Chemical composition

	Cold formed hollow sections		**Hot finished hollow sections**	
	Strongbox® 235	**Hybox® 355**	**Celsius® 275**	**Celsius® 355**
Specification	**TS 30** [1]	**EN 10219 355J2H**	**EN 10210 275J2H**	**EN 10210 355J2H**
C % max	0.17	0.22	0.20	0.22
Si % max	-	0.55	-	0.55
Mn % max	1.40	1.60	1.50	1.60
P % max	0.045	0.035	0.035	0.035
S % max	0.045	0.035	0.035	0.035
Ni % max	0.009	-	-	-
CEV % t ≤16mm	0.35	0.45	0.41	0.45

[1] Corus Tubes specification TS 30, generally in accordance with EN 10219 235JRH.

Mechanical properties

	Cold formed hollow sections		**Hot finished hollow sections**	
	Strongbox® 235	**Hybox® 355**	**Celsius® 275**	**Celsius® 355**
Specification	**TS 30** [1]	**EN 10219 355J2H**	**EN 10210 275J2H**	**EN 10210 355J2H**
Tensile strength R_m N/mm²				
t < 3mm	340 min	510-680	430-580	510-680
3 < t ≤ 40mm		490-630	410-560	490-630
Yeild strength R_{eh} min N/mm²				
t ≤ 16mm	235	355	275	355
t > 16mm	-	-	-	345
Min Elongation %				
L_o=5.65 $\sqrt{S_0}$ t ≤ 40mm	24[2][3]	20[2][3]	22	22
Impact properties				
Min Ave energy (J)	-	27 @ -20ºC	27 @ -20ºC	27 @ -20ºC
10 x 10 specimen				

[1] Corus Tubes specification TS 30, generally in accordance with EN 10219 235JRH excluding upper tensile limit and mass tolerance.

[2] 17% min for sizes 60 x 60, 80 x 40 and 76.1mm and below.

[3] Valve to be agreed for t< 3mm

Note: For Strongbox® 235, reduced section properties and thickness applies.
All thicknesses used in the design formulae and calculations are nominal, except for Strongbox® 235 which should use $0.9t_{nom}$ or (t_{nom}-0.5mm) whichever is the larger.

2 Scope

This publication has been written mainly for plane frame girder joints under predominantly static axial and/or moment loading conditions, however, some advice on non-planar frame joints is also given.

Note: In calculations this publication uses the convention that tensile forces and stresses are positive (+) and compressive ones are negative (-).

2.1 Joint geometry

The main types of joint configuration covered in this publication are shown in figure 1, however, other types of connections to structural hollow section main members, such as fin plates and cross plates, are also discussed.

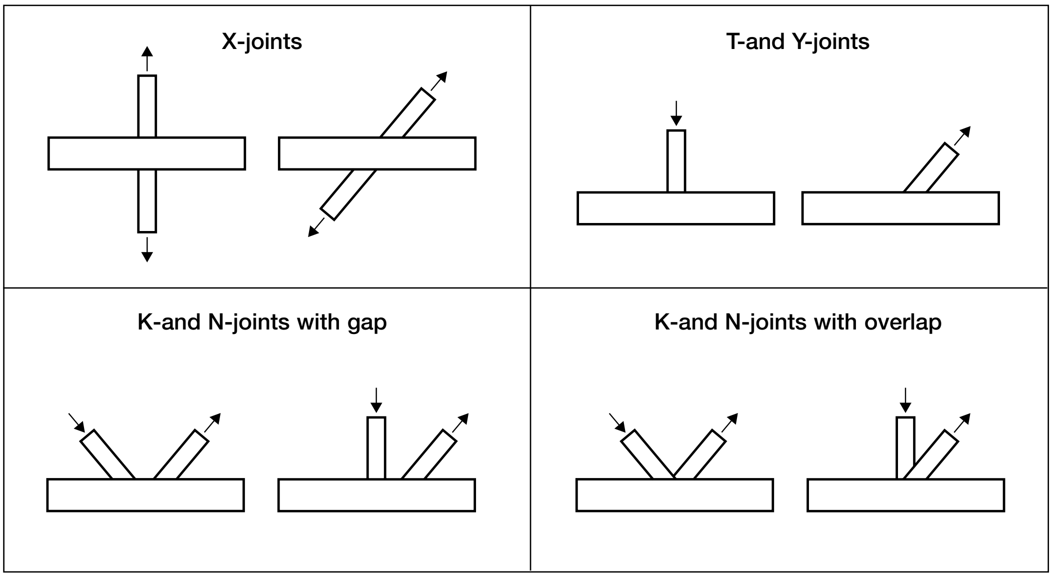


Figure 1 : Joint geometries

The angle between the chord and a bracing or between two bracings should be between 30º and 90º inclusive. If the angle is less than 30º then :

1. the designer must ensure that a structurally adequate weld can be made in the acute angle.

and 2. any joint capacity calculation should be made using an angle of 30º instead of the actual angle.

When K- or N-joints with overlapping bracings are being used, the overlap must be made with the first bracing running through to the chord and the second bracing either sitting on both the chord and the first bracing (partial overlap) or sitting fully on the first bracing (fully overlapped) as shown in (figure 2a). The joint should never be made by cutting the toes from each bracing and butting them up together (figure 2b), because this is both more difficult to fit together satisfactorily and, more importantly, can result in joint capacities up to 20% lower than those calculated by the joint design formulae given in section 5. A modified version of the type of joint shown in (figure 2b) can, however, be used provided that a plate of sufficient thickness is inserted between the two bracings - see section 4.6.3 on RHS chord overlap joint reinforcement.

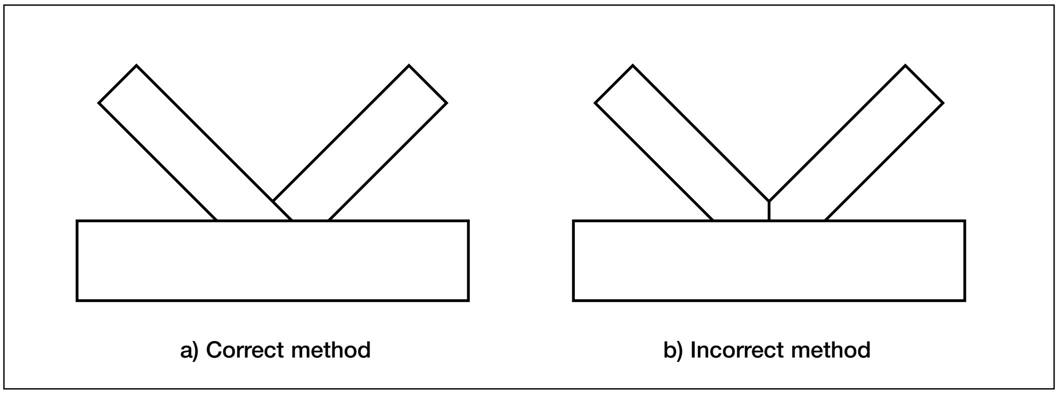


Figure 2 : Method of overlapping bracings

2.1.1 Validity ranges

In section 5 validity ranges are given for various geometric parameters of the joints. These validity ranges have been set to ensure that the modes of failure of the joints fall within the experimentally proven limits of the design formulae. If joints fall outside of these limits other failure modes, not covered by the formulae, may become critical. As an example, no check is required for chord shear in the gap between the bracings of CHS K- and N-joints, but this failure mode could become critical outside the validity limits given.

However, in general, if just one of these validity limits is slightly violated, and all of the joint's other geometric parameters are well inside the limits, then we would suggest that the actual joint capacity should be reduced to about 0.85 times the capacity calculated using the design formulae.

2.1.2 Joint symbols

A list of all the symbols used in this publication is given in section 7, however the main geometric symbols for the joint are shown below in figure 3.

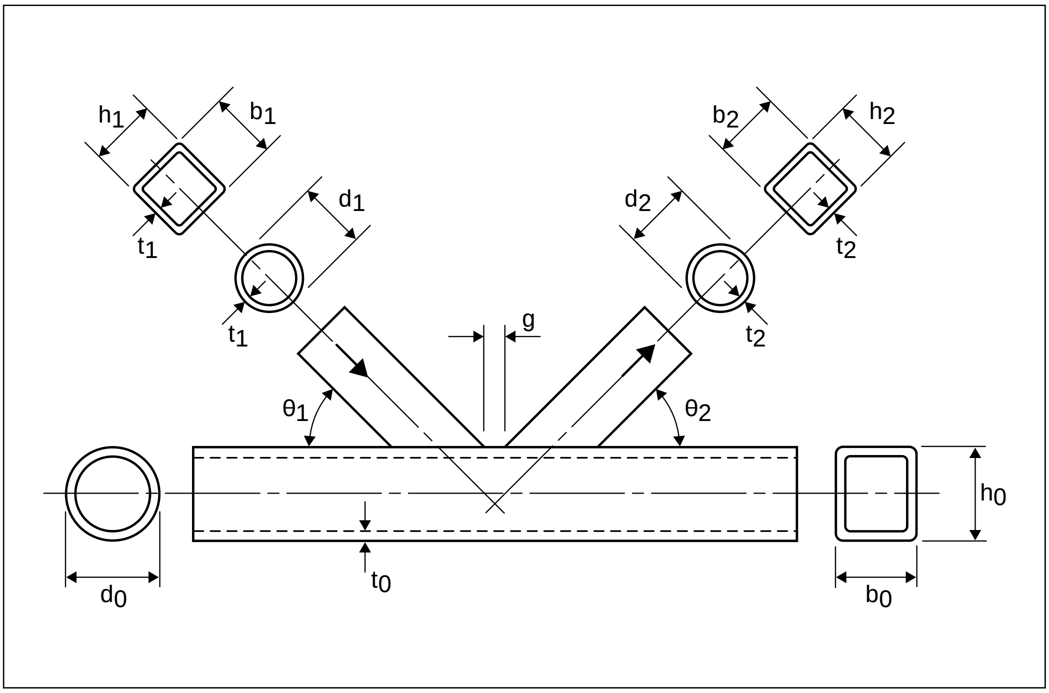


Figure 3 : Joint geometric symbols

2.2 Material

The design formulae, given in section 5, have only been verified experimentally for SHS material with a maximum nominal yield strength of 355N/mm^2. Care should be taken if materials with higher nominal yield strengths than this are used, since it is possible that, in some circumstances, deformations could become excessive and critical to the integrity of the structure.

All dimensions used in the design formulae and parameter limits are nominal, except for Strongbox® 235 thicknesses which should use $0.9t_{nom}$ or (t_{nom} -0.5mm) whichever is the larger.

2.3 Multiplanar joints

Multiplanar joints, such as those found in triangular and box girders, can be designed using the same design formulae as for planar joints, but with the multiplanar factor, μ, given in table 1, applied to the calculated chord face deformation capacity. The factors shown in table 1 have been determined for angles between the planes of 60º to 90º.

Additionally the chord must be checked for the combined shear from the two sets of bracings.

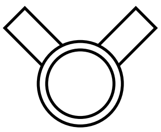
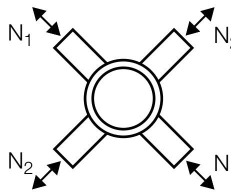
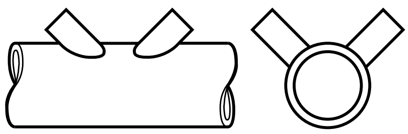
Joint type		CHS chords	RHS chords
TT-joint		$\mu = 1.0$	$\mu = 0.9$
XX-joint		$\mu = 1 + 0.33(N_{2,App}/N_{1,App})$	$\mu = 0.9(1 + 0.33 (N_{2,App}/N_{1,App}))$
		taking account of the sign (+ or -) and with $\lvert N_{2,App} \rvert \leq \lvert N_{1,App} \rvert$	
KK-joint		$\mu = 0.9$	$\mu = 0.9$

Table 1 : Multiplanar factors

To determine if a joint should be considered to be a multiplanar joint or a planar joint refer to figure 4

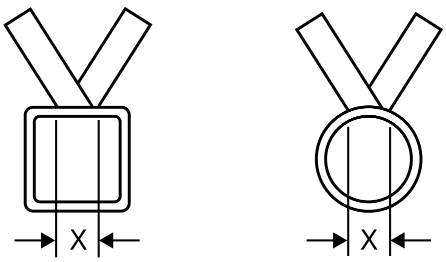
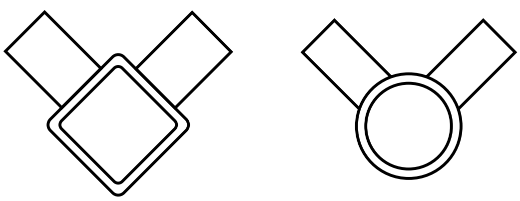
	Design as a plane frame joint and resolve bracing axial capacity into the two planes. RHS bracing - replace b_i with X CHS bracings - replace d_i with lesser of d_o or an equivalent CHS bracing having the same perimeter as the combined bracing footprint perimeter.
	Design as a planar joint and multiply by the relevent multiplaner factor from table 1

Figure 4 : Multiplanar joints

2.4 Load and moment interaction

If primary bending moments as well as axial loads are present in the bracings at a connection then the interaction effects of one on the other must be taken into account. Annex K of Eurocode No. 3 gives the following interaction formulae

For CHS chord joints the interaction formula is :-

$$\frac{N_{i,App}}{N_i} + \left[\frac{M_{ip,i,App}}{M_{ip,i}}\right]^2 + \frac{M_{op,i,App}}{M_{op,i}} \leq 1.0$$

For RHS chord joints the interaction formula is :-

$$\frac{N_{i,App}}{N_i} + \frac{M_{ip,i,App}}{M_{ip,i}} + \frac{M_{op,i,App}}{M_{op,i}} \leq 1.0$$

For I -and H- chord joints:-
use RHS interaction formula above.

3 General design guidance

3.1 Structural analysis

Lattice structures have traditionally been designed on the basis of pin-jointed frames with their members in tension or compression and the loads noding (meeting at a common point) at the centre of each joint. The usual practice is to arrange the joint so that the centre line of the bracing members intersect on the centre line of the chord member, as shown in figure 5.

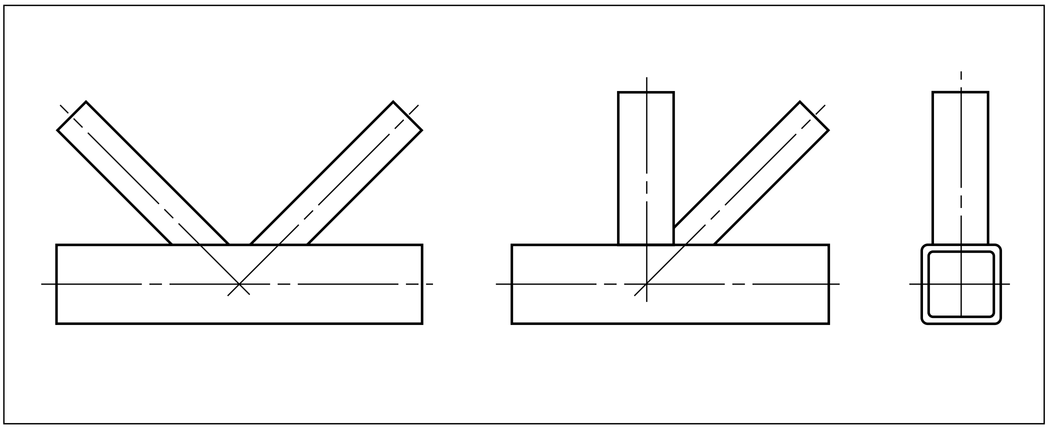


Figure 5 : Noding joints

The member sizes are determined in the normal way to carry the design loads and the welds at the joint to transfer the loads in the members. However, a lattice girder constructed using structural hollow sections is almost always welded, with one element welded directly to the next, e.g. bracing to chord. This means that the sizing of the members has a direct effect on the actual capacity of the joint being made. It is therefore imperative, if a structurally efficient and cost effective design is to result, that the member sizes and thicknesses are selected in such a way that they do not compromise the capacity of the joint. This is explained further in section 4.

While the assumption of centre line noding and pinned connections enables a good approximation of the axial forces in the members to be obtained, clearly in a real girder with continuous chords and welded connections, bending moments will be introduced into the chord members due to the inherent stiffness of the joints. In addition, in order to achieve the desired gap or overlap conditions between the bracings it may be necessary to depart from the noding conditions.

Many of the tests that have been carried out on welded joints, to derive the joint design recommendations, have incorporated noding eccentricities (see figure 6), some as large as $\pm d_0/2$ or $\pm h_0/2$.

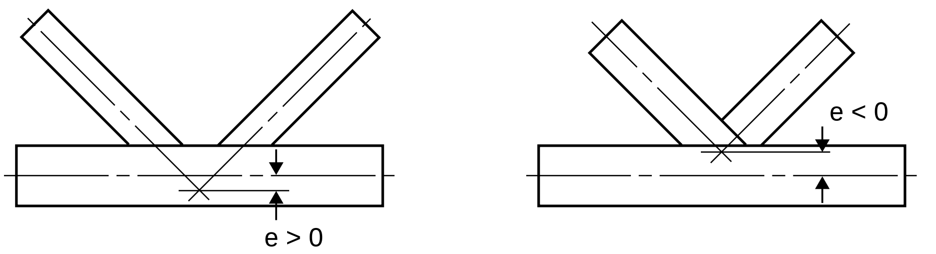


Figure 6 : Definition of joint eccentricity

The effects of moments due to the joint stiffness, for joints within the parameter limits given in section 5, and noding eccentricities, within the limits given below, are automatically taken into account in the joint design formulae given in section 5. It is good practice, however, to keep noding eccentricities to a minimum, particularly if bracings node outside the chord centre line (positive eccentricity, figure 6 a). The joint design formulae in section 5 should be used for eccentricities within the limits given below.

$$-0.55\ (d_0 \text{ or } h_0) \leq e \leq +0.25\ (d_0 \text{ or } h_0)$$

The effect of eccentricities outside these limits should be checked with reference to section 2.4 with the moments due to the eccentricity being taken into account. In most instances, the chords will be very much stiffer than the bracings and any moment, generated by the eccentricities, can be considered as being equally distributed to each side of the chord.

3.2 Welding

Only the main points regarding welding of structural hollow section lattice type joints are given here. More detailed information on welding methods, end preparation, weld strengths, weld types, weld design, etc. is given in reference 13.

When a bracing member is under load, a non-uniform stress distribution is set up in the bracing close to the joint, see figure 7, and therefore, the welds connecting the bracing to the chord must be designed to have sufficient resistance to allow for this non-uniformity of stress.

The weld should normally be made around the whole perimeter of the bracing by means of a butt weld, a fillet weld or a combination of the two. However, in partially overlapped bracing joints the hidden part of the connection need not be welded provided that the bracing load components perpendicular to the chord axis do not differ by more than 20%. In the case of 100% overlap joints the toe of the overlapped bracing must be welded to the chord. In order to acheive this, the overlap may be increased to a maximum of 110% to allow the toe of the overlapped bracing to be welded satisfactorily to the chord.

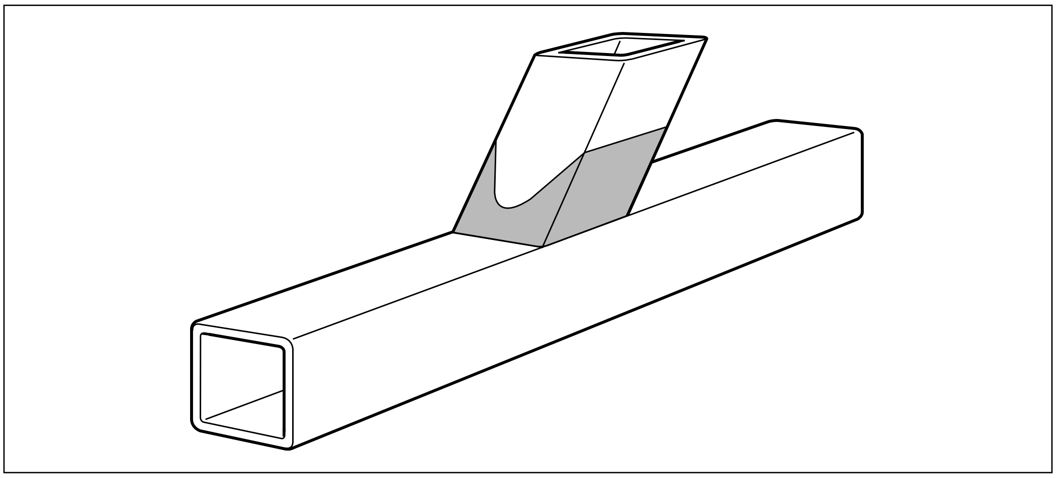


Figure 7 : Typical localised stress distribution at a joint

For bracing members in a lattice construction, the design resistance of a fillet weld should not normally be less than the design resistance of the member. This requirement will be satisfied if the throat size (a) is at least equal to or larger than the values shown in table 2, provided that electrodes of an equivalent strength grade to the steel, in terms of both yield and tensile strength, are used, see also figure 8.

The requirements of table 2 may be waived where a smaller weld size can be justified with regard to both resistance and deformational / rotational capacity, taking account of the possibility that only part of the weld's length may be effective.

Steel grade	Minimum throat size, a mm	Electrode grade EN 499
Celsius® 275	0.94 x t*	E35 2 xxxx
Celsius® 355	1.09 x t*	E42 2 xxxx
Strongbox® 235	0.91 x t*	E35 2 xxxx
Hybox® 355	1.09 x t*	E42 2 xxxx

* see figure 8

Table 2 : Prequalified Weld Throat Size

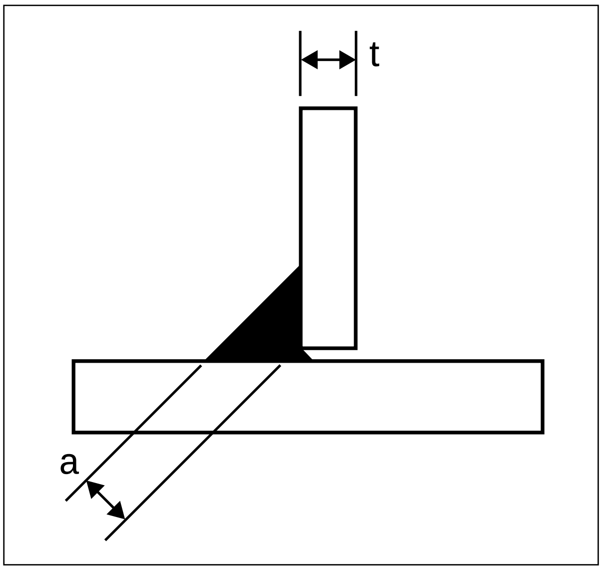


Figure 8 : Weld throat thickness

The weld at the toe of an inclined bracing is very important, see figure 9. Because of the non-uniform stress distribution around the bracing at the chord face, the toe area tends to be more highly stressed than the remainder of its periphery. As a result it is recommended that the toe of the bracing should be bevelled and a butt weld should always be used if the bracing angle, θ, is less than 60º. If the angle is 60º or greater then the weld type used for the remainder of the weld should be used, i.e. either a fillet or a butt weld.

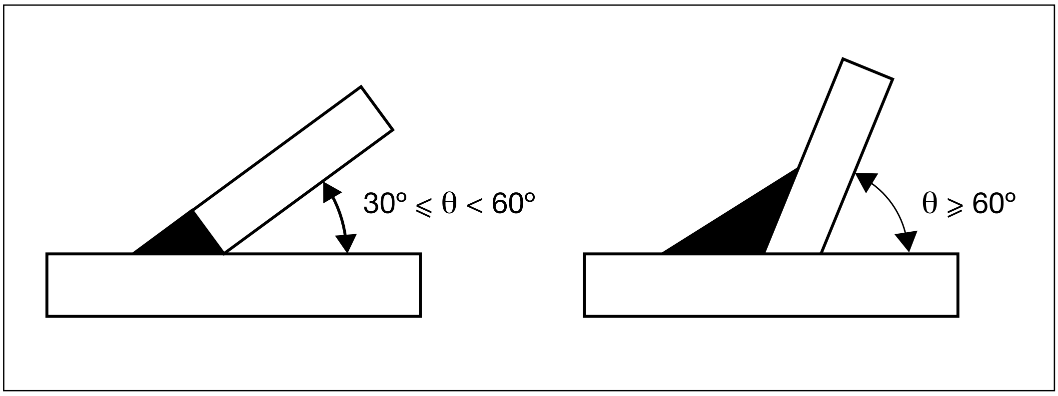


Figure 9 : Weld detail at bracing toe

3.2.1 Welding in cold formed RHS corners

EN1993-1-1: Annex K: Table A4 restricts welding of bracing members to chord members within 5t of the corner region of cold formed square or rectangular hollow section chord members unless the steel is a fully killed (A1≥0.02%) type.

Both Corus Tubes Strongbox® 235 and Hybox® 355 meet the fully killed requirements and can be welded in the corner region unless the thickness is greater than12mm when the 5t restriction applies.

3.3 Fabrication

In a lattice type construction the end preparation and welding of the bracings is generally the largest part of the fabrication costs and the chords the smallest. For example, in a typical 30m span girder, whilst the chords would probably be made from three lengths of material with straight cuts and two end-to-end butt welds, the bracings would number some twenty to twenty-five and each would require bevel cutting or profiling, if using a CHS chord, and welding at each end.

As a general rule the number of bracing members should be as small as possible and this can usually best be achieved by using K- type bracings rather than N-type bracings. Hollow sections are much more efficient in compression than open sections, such as angles or channels, and as a result the requirement to make compression bracings as short as possible does not occur and a K-type bracing layout becomes much more efficient.

The ends of each bracing in a girder with circular hollow section chords have to be profile shaped to fit around the curvature of the chord member, see figure 10, unless the bracing is very much smaller than the chord. Also for joints with CHS bracings and chords and with overlapping bracings the overlapping bracing has to be profile shaped to fit to both the chord and the other bracing.

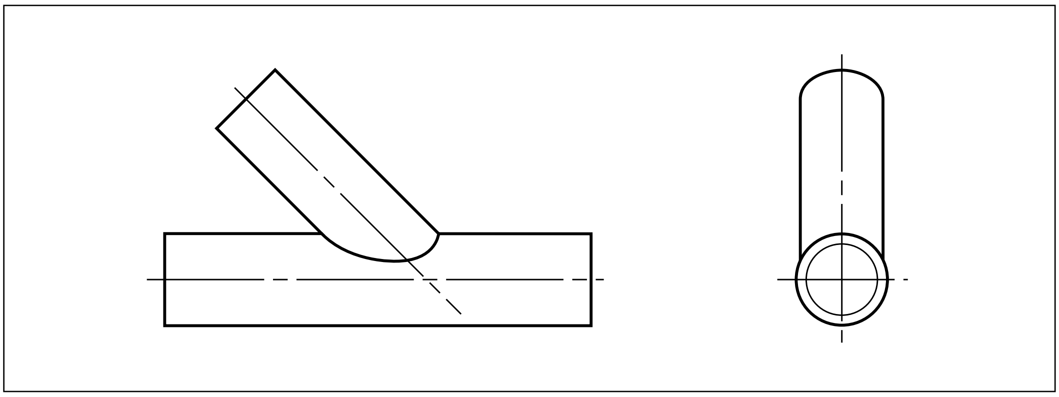


Figure 10 : Connections to a circular chord

For joints with RHS chords and either RHS or CHS bracings, unless the bracings partially overlap, only a single straight cut is required at the ends of the bracings.

As well as the end preparation of the bracings, the ease with which the members of a girder, or other construction, can be put into position and welded will effect the overall costs. Generally it is much easier, and therefore cheaper, to assemble and weld a girder with a gap between the bracings than a similar one with the bracings overlapping. This is because with gap joints you have a much slacker tolerance on fit up and the actual location of the panel points can easily be maintained by slight adjustments as each bracing is fitted; this is not possible for joints with overlapping bracings, especially partial overlapping ones, and unless extra care is taken it can result in accumulated errors in the panel point locations.

More detailed information on fabrication, assembly and erection is given in reference 14

4 Parameters affecting joint capacity

4.1 General

The effect that the various geometric parameters of the joint have on its load capacity is dependant upon the joint type (single bracing, two bracings with a gap or an overlap) and the type of loading on the joint (tension, compression, moment). Depending on these various conditions a number of different failure modes, see section 4.2, are possible.

Design is always a compromise between various conflicting requirements and the following notes highlight some of the points that need to be considered in arriving at an efficient design.

1) The joint

a) The joint capacity will always be higher if the thinner member at a joint sits on and is welded to the thicker member rather than the other way around.

b) Joints with overlapping bracings will generally have a higher capacity than joints with a gap between the bracings, all other things being equal.

c) The joint capacity, for all joint and load types (except fully overlapped joints), will be increased if small thick chords rather than larger and thinner chords are used.

d) Joints with a gap between the bracings have a higher capacity if the bracing to chord width ratio is as high as possible. This requires large thin bracings and small thick chords.

e) Joints with partially overlapping bracings have a higher capacity if both the chord and the overlapped bracing are as small and thick as possible.

f) Joints with fully overlapping bracings have a higher capacity if the overlapped bracing is as small and thick as possible. In this case the chord has no effect on the joint capacity.

g) On a size for size basis, joints with CHS chords will have a higher capacity than joints with RHS chords

2) The overall girder requirements

a) The overall girder behaviour, e.g. lateral stability, is increased if the chord members are large and thin. This also increases the compression chord strut capacity, due to its larger radius of gyration.

b) Consideration must also be given to the discussion on fabrication in section 3.3.

4.2 Joint failure modes

Joints in structural hollow sections can fail in a number of different failure modes depending on the joint type, the geometric parameters of the joint and the type of loading. These various types of failure are described in figures 11 to 16.

If the relevant geometric parameter limits given in section 5 are adhered to then the number of failure modes is limited to those defined there; however, if this is not the case then other failure modes may become critical.

Chord face deformation, figure 11, is the most common failure mode for joints with a single bracing, and for K- and N-joints with a gap between the bracings if the bracing to chord width ratio (ß) is less than 0.85.

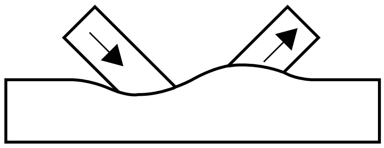
Mode	Description
Chord face deformation	

Figure 11 : Chord face deformation

Chord side wall buckling, figure 12, usually only occurs when the ß ratio is greater than about 0.85, especially for joints with a single bracing. The failure mode also includes chord side wall yielding if the bracing carries a tensile load.

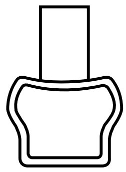
Mode	Description
Chord sidewall buckling	

Figure 12 : Chord side wall buckling

Chord shear, figure 13, does not often become critical, it is most likely to become so if rectangular chords with the width (b_0) greater than the depth (h_0) are being used. If the validity ranges given in section 5 are met then chord shear does not occur with CHS chords.

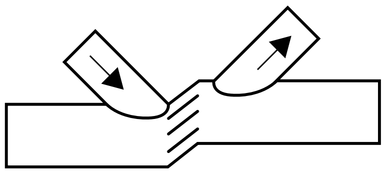
Mode	Description
Chord shear	

Figure 13 : Chord shear

Chord punching shear, figure 14, is not usually critical but can occur when the chord width to thickness ratio (2γ) is small.

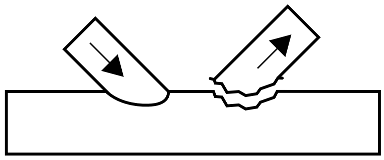
Mode	Description
Chord punching shear	

Figure 14 : Chord punching shear

Bracing effective width failures, figure 15, are generally associated with RHS chord gap joints which have large ß ratios and thin chords. It is also the predominant failure mode for RHS chord joints with overlapping RHS bracings.

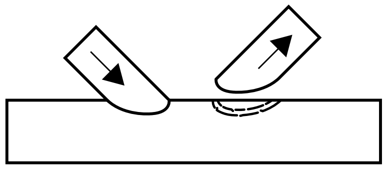
Mode	Description
Bracing effective width	

Figure 15 : Bracing effective width

Localised buckling of the chord or bracings, figure 16, is due to the non-uniform stress distribution at the joint, and will not occur if the validity ranges given in section 5 are met.

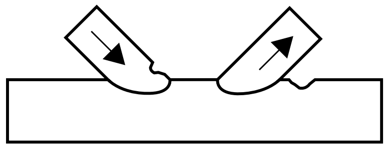
Mode	Description
Chord or bracing localised buckling	

Figure 16 : Localised buckling of the chord or bracings

4.3 Joints with a single bracing

The statements given in table 3 will only be true provided that the joint capacity does not exceed the capacity of the members. In all cases the capacity is defined as a load along the axis of the bracing.

Joint parameter		Parameter value	Effect on capacity
Chord width to thickness ratio	b_o / t_o or d_o / t_o	reduced	increased
Bracing to chord width ratio	d_1 / d_0 or b_1 / b_0	increased	increased (1)
Bracing angle	θ	reduced	increased
Bracing to chord strength factor	$\frac{f_{y1}\, t_1}{f_{y0}\, t_0}$	reduced	increased

Note : (1) - provided that RHS chord side wall buckling does not become critical, when ß > 0.85

Table 3 : Effect of parameter changes on the capacity of T-, Y- and X-joints

4.4 Joint with a gap between bracing

The statements given in table 4 will only be true provided that the joint capacity does not exceed the capacity of the members. In all cases the capacity is defined as a load along the axis of the bracing.

Joint parameter		Parameter value	Effect on capacity
Chord width to thickness ratio	b_0 / t_0 or d_0 / t_0	reduced	increased
Bracing to chord width ratio	d_1 / d_0 or b_1 / b_0	increased	increased (1)
Bracing angle	θ	reduced	increased
Bracing to chord strength factor	$\frac{f_{y1}\, t_1}{f_{y0}\, t_0}$	reduced	increased
Gap between bracings	g	reduced	increased (2)

Note : (1) - provided that RHS chord side wall buckling does not become critical, when ß > 0.85
(2) - only true for CHS chord joints

Table 4 : Effect of parameter changes on the capacity of K- or N-joints with gap

4.5 Joints with overlapped bracings

The statements given in table 5 will only be true provided that the joint capacity does not exceed the capacity of the members. In all cases the capacity is defined as a load along the axis of the bracing.

Joint parameter		Parameter value	Effect on capacity
Chord width to thickness ratio	b_0 / t_0 or d_0 / t_0	reduced	increased
Overlapped bracing width to thickness ratio	b_j / t_j	reduced	increased (1)
Bracing to chord width ratio	d_1 / d_0 or b_1 / b_0	increased	increased (2)
Bracing angle	θ	reduced	increased (3)
Overlapped bracing to chord strength factor	$\frac{f_{yj}\, t_j}{f_{y0}\, t_0}$	reduced	increased
Bracing to bracing strength factor	$\frac{f_{y1}\, t_1}{f_{yj}\, t_j}$	reduced	increased
Overlap of bracings	O_v	increased	increased

Note : (1) - only true for RHS joints
(2) - provided that RHS chord side wall buckling does not become critical, when ß > 0.85
(3) - only true for CHS chord joints

Table 5 : Effect of parameter changes on the capacity of K- or N-joints with overlap

4.6 Joint reinforcement

If a joint does not have the design capacity required, and it is not possible to change either the joint geometry or the member sizes, it may be possible to increase the design capacity with the use of appropriate reinforcement. Adding reinforcement to a joint should only be carried out after careful consideration. It is relatively expensive from a fabrication point of view and can be obtrusive from an aesthetics view point.

The type of reinforcement required depends upon the criterion causing the lowest capacity. Methods for reinforcing both CHS and RHS chord joints are given below. An alternative to the methods shown is to insert a length of chord material, of the required thickness, at the joint location, the length of which should be at least the same as the length, h_r, given in the following methods.

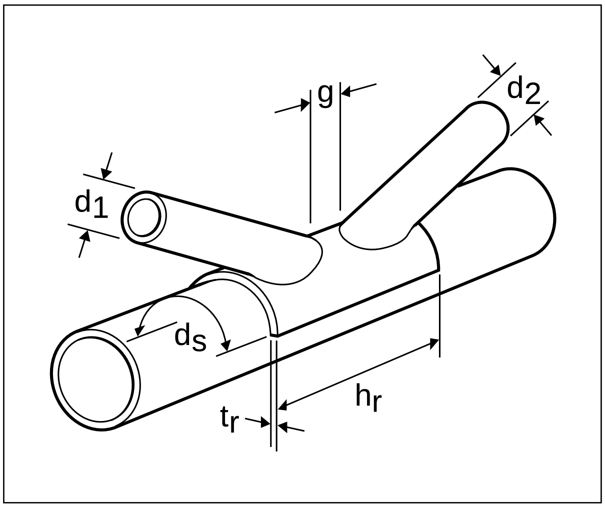
The required thickness of the reinforcement, t_r, should be calculated by re-arranging the relevant formula given in section 5 to calculate the required chord thickness, t_0, this is then the thickness of the reinforcement required. In the case of CHS chord saddle and RHS chord face reinforcement only the reinforcement thickness, and not the combined thickness of the chord and reinforcement, should be used to determine the capacity of the reinforced joint. For RHS chord side wall reinforcement the combined thickness may be used for the shear capacity, but for chord side wall buckling the chord side wall and reinforcement should be considered as two separate plates and their capacities added together.

The plate used for the reinforcement should be the same steel grade as the chord material. For CHS saddle and RHS chord face reinforcement the plate should have good through thickness properties with no laminations. The weld used to connect the reinforcement to the hollow section chord member should be made around the total periphery of the plate.

When plates are welded all round to the chord face, as is the case for the reinforcement plates shown in sections 4.6.1 and 4.6.2, special care and precautions should be taken if the structure is subsequently to be galvanised.

4.6.1 Reinforcement of CHS chord joints

The only external reinforcement method used with a CHS chord is saddle reinforcement, where either a curved plate or part of a thicker CHS is used. The size and type of reinforcement is shown in figure 17. The dimensions of the saddle should be as shown below.



$d_s = \pi\, d_0 / 2$

$h_r \geq 1.5\, [d_1 / \sin\theta_1 + g + d_2 / \sin\theta_2]$
for K- or N-gap joints

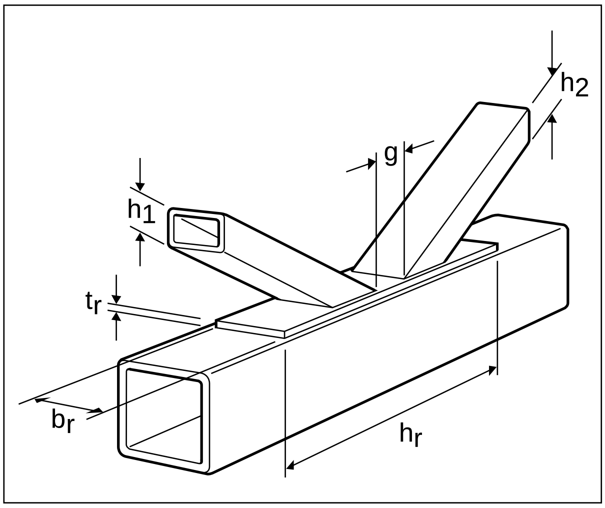
$h_r \geq 1.5\, d_1 / \sin\theta_1$
for T-, X- or Y-joints

t_r = required reinforcement thickness

Figure 17 : CHS chord saddle reinforcement

4.6.2 Reinforcement of RHS chord gap joints

A gap joint with RHS chords can be reinforced in several ways depending upon the critical design criterion. If the critical criterion is chord face deformation or chord punching shear or bracing effective width then reinforcing the face of the chord to which the bracings are attached is appropriate (see figure 18). However, if the critical criterion is either chord side wall buckling or chord shear then plates welded to the side walls of the chord should be used (see figure 19). The required dimensions of the reinforcing plates are shown below.



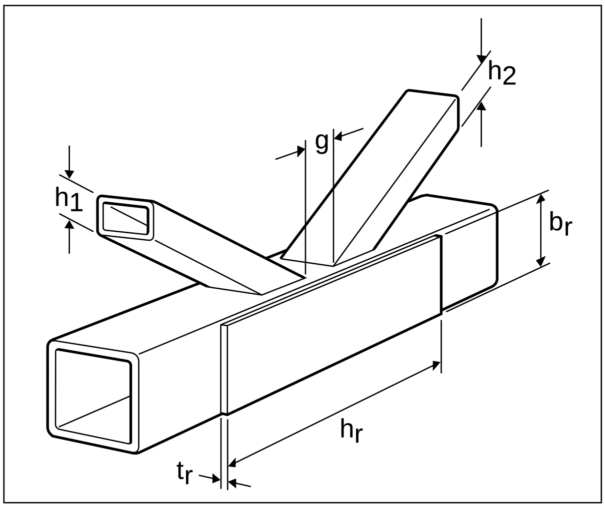
$h_r \geq 1.5\ [h_1 / \sin\theta_1 + g + h_2 / \sin\theta_2]$
for K- or N-gap joints

$h_r \geq h_1 / \sin\theta_1 + \sqrt{(b_r(b_r - b_1))}$
and $\geq 1.5\ h_1 / \sin\theta_1$
for T-, X- or Y-joints

$b_r \geq b_0 - 2t_0$

t_r = required reinforcement thickness

Figure 18 : RHS chord face reinforcement



$h_r \geq 1.5\ [h_1 / \sin\theta_1 + g + h_2 / \sin\theta_2]$
for K- or N-gap joints

$h_r \geq 1.5\ h_1 / \sin\theta_1$
for T-, X- or Y-joints

$b_r \geq h_0 - 2t_0$

t_r = required reinforcement thickness

Figure 19 : RHS chord side wall reinforcement

4.6.3 Reinforcement of RHS chord overlap joints

An overlap joint with RHS chords can be reinforced by using a transverse plate as shown in figure 20. The plate width b_r should generally be wider than the bracings to allow a fillet weld with a throat thickness equal to the bracing thickness to be used.

This should be treated as a 50 to 80% overlap joint with t_r being used instead of the overlapped bracing thickness t_j in the calculation of b_{eov} (see section 5.2). This type of reinforcement can be used in conjunction with the chord face reinforcement , shown in figure 18, if necessary.

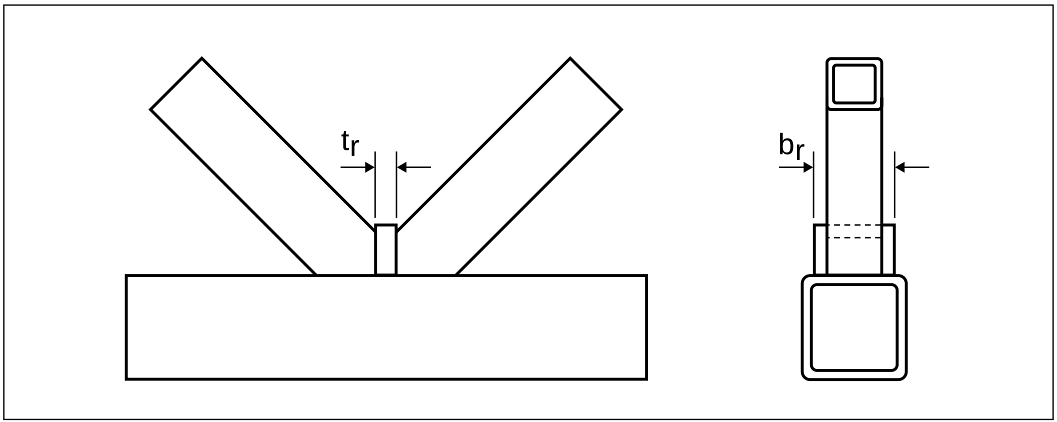


Figure 20 : RHS chord transverse plate reinforcement

5. Joint design formulae

When more than one failure criteria formula is given the value of the lowest resulting capacity should be used. In all cases any applied factored moment should be taken as that acting at the chord face and not that at the chord centre line.

5.1 CHS chord joints

All dimensions used in the design formulae and parameter limits are nominal, except for Strongbox® 235 thicknesses which should use $0{,}9t_{nom}$ or (t_{nom} -0.5mm) which ever is the larger.

5.1.1 CHS chord joint parameter limits

Joint type	Bracing type	d_0/t_0	d_i/t_i	$(b_i, h_i$ or d_i or $t_i)/d_0$	Gap/lap	Brace angle
T-,K- and N-joints	CHS	≤50	≤50	$d_i/d_0 \geq 0.2$	gap $\geq t_1+t_2$ lap ≥ 25%	30° ≤ θ ≤ 90°
X-joints		≤40			-	
T-joints	Transverse plate	≤50	-	$b_1/d_0 \geq 0.4$	-	θ ≈ 90°
X-joints		≤40	-		-	
T-joints	Longitudinal plate	≤50	-	$h_1/d_0 \leq 4.0$* $t_1/d_0 \leq 0.2$	-	
X-joints		≤40	-		-	
T-joints	RHS and I- or H- section	≤50	-	$b_1/d_0 \geq 0.4$ $h_1/d_0 \leq 4.0$*	-	30° ≤ θ ≤ 90°
X-joints		≤40	-		-	

Table 6 : CHS Joint Parameter limits

* can be physically > 4, but for calculation purposes should not be taken as > 4 for plate or > 2 for RHS bracing.

5.1.2 CHS chord joint functions

The following functions are used during the calculation of CHS chord joint capacities

Chord end load function, $f(n_p)$ - see figure 21

$$f(n_p) = 1 + 0.3\sigma_p/f_{y0} - 0.3\,(\sigma_p/f_{y0})^2 \text{ but not greater than 1.0,}$$

σ_p = the least compressive factored applied stress in the chord due to axial loads and moments adjacent to the joint and is negative for compression

σ_p/f_{y0} is the chord stress ratio shown in figure 21

Gap/lap function, f(g) - see figure 22

$$f(g) = \gamma^{0.2}\left[1 + \frac{0.024\gamma^{1.2}}{1 + \exp(0.5\,g/t_0 - 1.33)}\right]$$

Gap (g) is positive for a gap joint and negative for an overlap joint

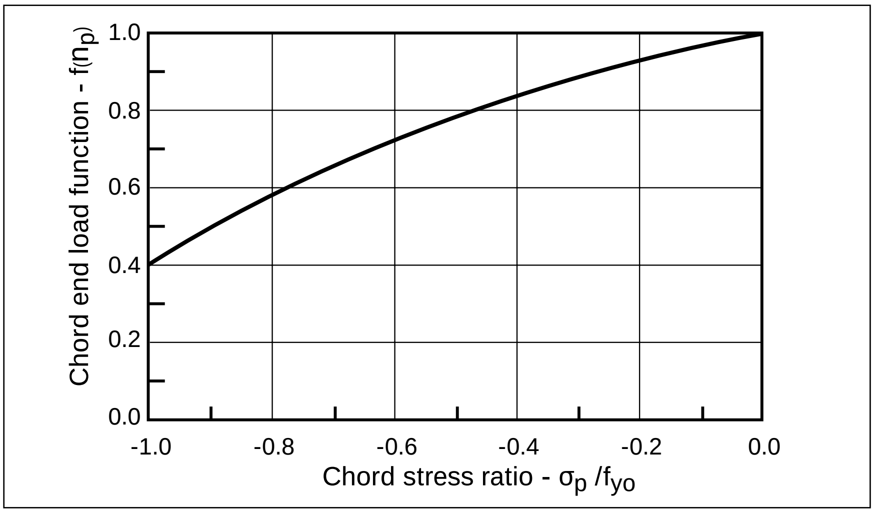


Figure 21 : CHS joint - Chord end load function

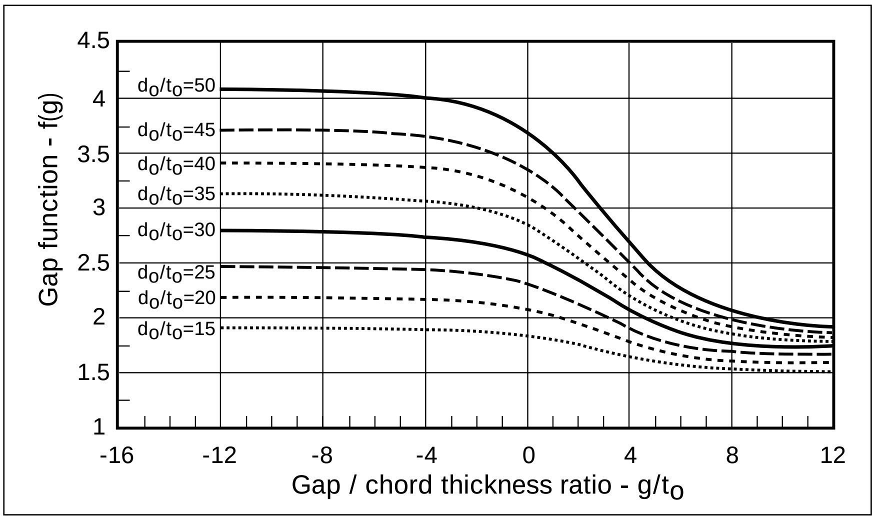


Figure 22 : CHS joint - Gap/lap function

5.1.3 CHS chords and CHS bracings with axial loads

T- and Y-joints

Chord face deformation, $$N_1 = \frac{f_{y0}\, t_0^2}{\sin\theta_1} (2.8 + 14.2\, \beta^2)\, \gamma^{0.2}\, f(n_p)$$

X-joints

Chord face deformation, $$N_1 = \frac{f_{y0}\, t_0^2}{\sin\theta_1} \frac{5.2}{(1 - 0.81\, \beta)} f(n_p)$$

K- and N-joints

Chord face deformation, $$N_1 = \frac{f_{y0}\, t_0^2}{\sin\theta_1} (1.8 + 10.2\, d_1/d_0)\, f(g)\, f(n_p)$$
(compression brace)

Chord face deformation, $$N_2 = \frac{\sin\theta_1}{\sin\theta_2} \times N_1$$
(tension brace)

For all these joint types, except those with overlapping bracings, the joint must also be checked for chord punching shear failure when $d_i \leq d_0 - 2t_0$

Chord punching shear, $$N_i = \frac{f_{y0}\, t_0\, \pi\, d_i}{\sqrt{3}} \quad \frac{1 + \sin\theta_i}{2 \sin^2\theta_i}$$

5.1.4 CHS chords and CHS bracings with moments

T-, Y-, X-joints and K- and N-joints with gap

Chord face deformation criterion - this should be checked for all geometric joint configurations

In-plane moments, $$M_{ip,i} = 4.85 \frac{f_{y0}\, t_0^2\, d_i}{\sin\theta_i} \quad ß \sqrt{\gamma} \; f(n_p)$$

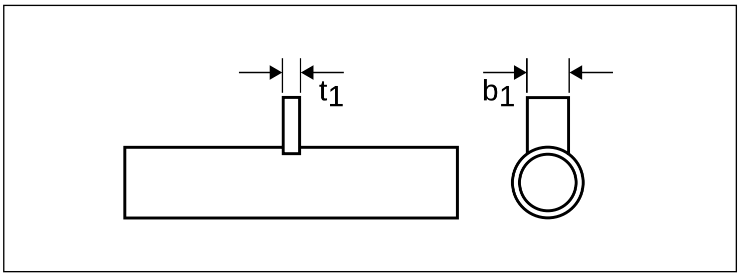
Out-of-plane moments, $$M_{op,i} = \frac{f_{y0}\, t_0^2\, d_i}{\sin\theta_i} \quad \frac{2.7}{1 - 0.81\, ß} \quad f(n_p)$$

Punching shear criterion - this must also be checked for these joint types when $d_i \leq d_0 - 2\, t_0$

In-plane moments, $$M_{ip,i} = \frac{f_{y0}\, t_0\, d_i^2}{\sqrt{3}} \quad \frac{1 + 3 \sin\theta_i}{4 \sin^2\theta_i}$$

Out-of-plane moments, $$M_{op,i} = \frac{f_{y0}\, t_0\, d_i^2}{\sqrt{3}} \quad \frac{3 + \sin\theta_i}{4 \sin^2\theta_i}$$

5.1.5 CHS chords with transverse gusset plates



T-joints axial load chord face deformation

$$N_1 = f_{y0}\, t_0^2\, (4 + 20\, ß^2)\, f(n_p)$$

X-joints axial load chord face deformation

$$N_1 = \frac{5\, f_{y0}\, t_0^2}{(1 - 0.81\, ß)} \quad f(n_p)$$

T- and X-joints out-of-plane moment chord face deformation

$$M_{op,1} = 0.5\ b_1\ N_1$$

T- and X-joints in-plane moment chord face deformation

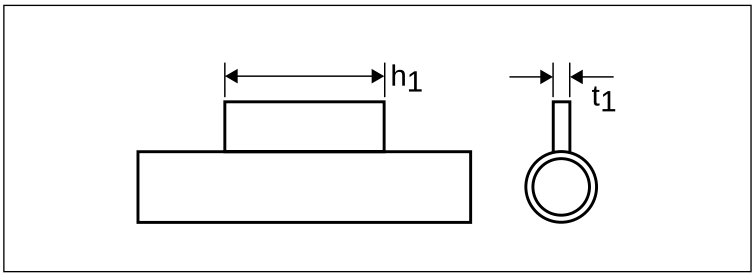
$$M_{ip,1} = t_1\ N_1$$

T- and X-joint chord punching shear

In all cases the following check must be made to ensure that any factored applied axial loads and moments do not exceed the chord punching shear capacity.

$$N_{app} + 6\ M_{app}/\ b_1 \leq 2\ f_{y0}\ t_0\ b_1/\sqrt{3}$$

5.1.6 CHS chords with longitudinal gusset plates



T- and X-joints axial load chord face deformation

$$N_1 = 5\ f_{y0}\ t_0^2\ (1+ 0.25\ h_1/d_0)\ f(n_p)$$

T- and X-joints out-of-plane moment chord face deformation

$$M_{op,1} = 0.5\ t_1\ N_1$$

T- and X-joints in-plane moment chord face deformation

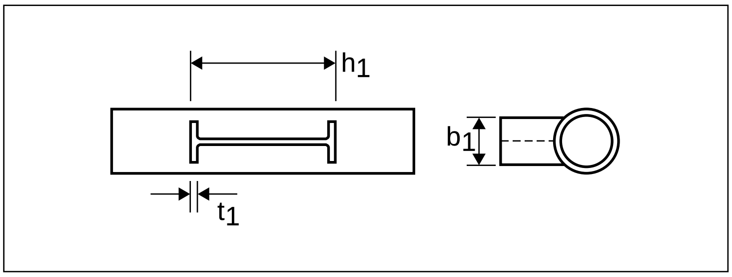
$$M_{ip,1} = h_1\ N_1$$

T- and X-joint chord punching shear

In all cases the following check must be made to ensure that any factored applied axial loads and moments do not exceed the chord punching shear capacity.

$$N_{app} + 6\ M_{app}/h_1 \leq 2\ f_{y0}\ t_0\ h_1/\sqrt{3}$$

5.1.7 CHS chords and I -, H - or RHS bracings



T-joints chord face deformation

$$N_1 = f_{y0}\, t_0^2\, (4 + 20\, ß^2)\, (1 + 0.25\, h_1/d_0)\, f(n_p)$$

$$M_{ip,1} = h_1\, N_1 / (1 + 0.25\, h_1/d_0) \quad \text{for I- and H- bracings}$$

$$M_{ip,1} = h_1 N_1 \quad \text{for RHS bracings}$$

$$M_{op,1} = 0.5\, b_1 N_1$$

X-joints chord face deformation

$$N_1 = \frac{5\, f_{y0}\, t_0^2}{(1 - 0.81\, ß)}\, (1 + 0.25\, h_1/d_0)\, f(n_p)$$

$$M_{ip,1} = h_1\, N_1 / (1 + 0.25\, h_1/d_0) \quad \text{for I- and H- bracings}$$

$$M_{ip,1} = h_1\, N_1 \quad \text{for RHS bracings}$$

$$M_{op,1} = 0.5\, b_1 N_1$$

T- and X-joint chord punching shear

In all cases the following check must be made to ensure that any factored applied axial loads and moments do not exceed the chord punching shear capacity.

For I- and H-sections $(N_{app} / A_1 + M_{app} / W_{el.1})\, t_1 \leq 2\, f_{y0}\, t_0 / \sqrt{3}$

For RHS sections $(N_{app} / A_1 + M_{app} / W_{el.1})\, t_1 \leq f_{y0}\, t_0 / \sqrt{3}$

5.2 RHS chord joints

All dimensions used in the design formulae and parameter limits are nominal, except for Strongbox® 235 thicknesses which should use $0.9t_{nom}$ or (t_{nom} -0.5mm) whichever is the larger.

5.2.1 RHS chord joint parameter limits

<table>
<tr><th rowspan="2">Joint type</th><th rowspan="2">Bracing type</th><th rowspan="2">$\frac{(b_o \text{ or } h_o)}{t_o}$</th><th colspan="2">$(b_i$ or h_i or $d_i) / t_i$</th><th rowspan="2">$(d_i$ or $b_i)/ b_o$</th><th rowspan="2">Gap / lap</th></tr>
<tr><th>Compression</th><th>Tension</th></tr>
<tr><td>T- and X-joints</td><td rowspan="3">RHS</td><td rowspan="2">≤ 35</td><td rowspan="2">≤ 35
and
$\leq 34.5\sqrt{(275/f_{yi})}$</td><td rowspan="3">≤ 35</td><td>≥ 0.25</td><td>-</td></tr>
<tr><td>K- and N-gap joints</td><td>≥ 0.35 and ≥ 0.1 + 0.01 b_0/t_0</td><td>gap ≥ t_1+t_2
and≥ $0.5(b_0-(b_1+b_2)/2)$
but ≤ $1.5(b_0-(b_1+b_2)/2)$</td></tr>
<tr><td>K- and N-lap joints</td><td>≤ 40</td><td>$\leq 30.4\sqrt{(275/f_{yi})}$</td><td>≥ 0.25</td><td>25% ≤ lap ≤ 100%</td></tr>
<tr><td>All types</td><td>CHS</td><td>As above</td><td>$\leq 41.5\sqrt{(275/f_{yi})}$</td><td>≤ 50</td><td>≥ 0.4 and ≤ 0.8</td><td>As above</td></tr>
<tr><td rowspan="2">T- and X-joints</td><td>Transverse plate</td><td>≤ 30</td><td>-</td><td>-</td><td>≥ 0.5</td><td>-</td></tr>
<tr><td>Longitudinal plate</td><td>≤ 30</td><td>-</td><td>-</td><td>$t_1/b_0 \leq 0.2$
$h_1/b_0 \leq 4.0$*</td><td>-</td></tr>
</table>

Table 7 : RHS joint Parameter limits

Note : in gap joints, if the gap is greater than $1.5(b_0-b_i)$, then it should be treated as two separateT- or Y-joints and the chord checked for shear between the braces

* can be physically > 4, but should for calculation purposes not be taken as > 4 for plater or H-section bracings.

The angle between the chord and either an RHS or a CHS bracing and between braces should be between 30º and 90º inclusive. Longitudinal plates should be at about 90º to the chord face.

5.2.2 RHS chord joint functions

The following functions are used during the calculation of RHS chord joint capacities.

Chord end load function, f(n), f(m)

For all joints except those with a longitudinal gusset plate - see figure 23

$$f(n) = 1.3 + \frac{0.4\,\sigma_0}{f_{y0}\,\beta} \quad \text{but not greater than 1.0,}$$

For joints with a longitudinal gusset plate only - see figure 24

$$f(m) = 1.3(1 + \sigma_0 / f_{y0}) \quad \text{but not greater than 1.0,}$$

σ_0 = the most compressive factored applied stress in the chord due to axial loads and moments adjacent to the joint and is negative for compression

σ_0 / f_{y0} is the chord stress ratio shown in figures 23 and 24

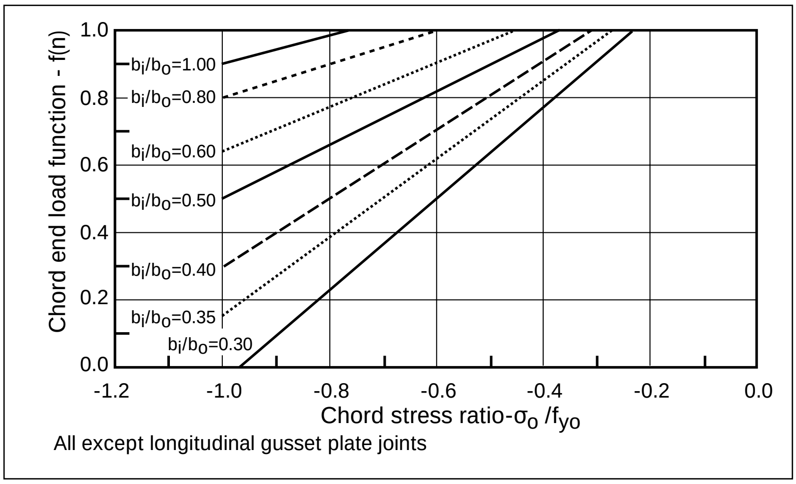


Figure 23 : RHS joint - Chord end load function (All except longitudinal gusset plate joints)

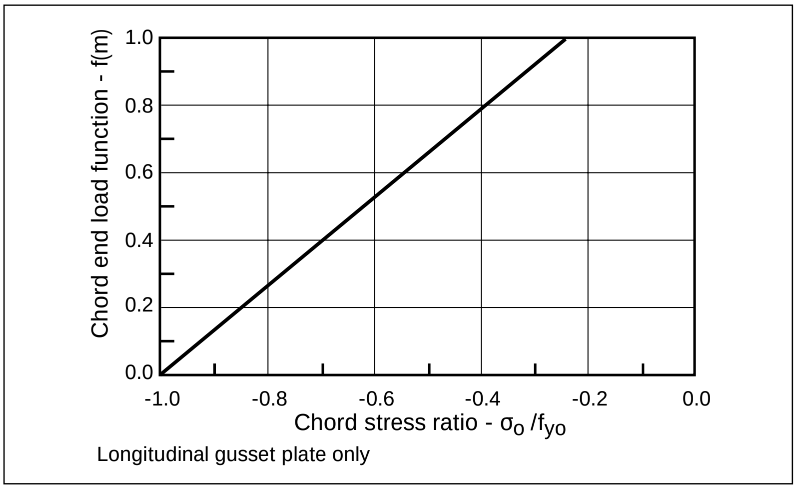


Figure 24 : RHS joint - Chord end load function (Longitudinal gusset plates only)

Bracing effective width functions

Normal effective width, $b_{eff} = \dfrac{10}{b_0/t_0} \dfrac{f_{y0}\, t_0}{f_{yi}\, t_i}\ b_i \quad \text{but} \le b_i$

Punching shear effective width, $b_{ep} = \dfrac{10}{b_0/t_0}\ b_i \quad \text{but} \le b_i$

Overlap effective width, $b_{eov} = \dfrac{10}{b_j/t_j} \dfrac{f_{yj}\, t_j}{f_{yi}\, t_i}\ b_i \quad \text{but} \le b_i$

(Suffix 'j' indicates the overlapped bracing)

Chord design strength for T-, Y- and X-joints, $f(f_b)$

For tension in the bracing $f(f_b) = f_{y0}$

For compression in the bracing $f(f_b) = f_c$ for T- and Y-joints

$f(f_b) = 0.8\ f_c \sin\theta_i$ for X-joints

With f_c obtained from BS5950: Part 1 Table 24 for strut curve c or Eurocode 3 Clause 5.5.1 for a slenderness ratio, λ, of $3.46\ (h_0/t_0 - 2) / \sqrt{(\sin\theta_i)}$

Chord shear area, A_v

The chord shear area, A_v, in uniplanar K- and N-joints with a gap is dependant upon the type of bracings and the size of the gap

$$A_v = (2\,h_0 + \alpha\,b_0)\,t_0$$

$$\text{with } \alpha = \left[\frac{1}{1 + \dfrac{4\,g^2}{3\,t_0^2}}\right]^{0.5} \quad \text{for RHS bracings}$$

and $\alpha = 0$ for CHS bracings

In multiplanar girders the shear area, Av, given below should be used for the two shear planes respectively, irrespective of the type of bracing.

$$A_v = 2(h_0 - t_0)\,t_0 \text{ or } 2(b_0 - t_0)\,t_0$$

5.2.3 RHS chords and RHS bracings with axial loads

A number of failure modes can be critical for RHS chord joints. In this section the design formulae for all possible modes of failure, within the parameter limits, are given. The actual capacity of the joint should always be taken as the lowest of these capacities.

T-, Y- and X-joints

Chord face deformation, ($ß \leq 0.85$ only)

$$N_1 = \frac{f_{y0}\,t_0^2}{(1 - ß)\sin\theta_1}\left[\frac{2h_1}{b_0\sin\theta_1} + 4\sqrt{(1 - ß)}\right] f(n)$$

Chord shear, (X-joints with $\theta < 90°$ only)

$$N_1 = \frac{f_{y0}\,A_v}{\sqrt{3}\sin\theta_1} \quad \text{where } \alpha = 0 \text{ in } A_v$$

Chord side wall buckling, ($ß = 1.0$)

$$N_1 = \frac{f(f_b)\,t_0}{\sin\theta_1}\left[\frac{2h_1}{\sin\theta_1} + 10\,t_0\right]$$

Chord punching shear, ($0.85 \leq ß \leq (1 - 2t_0/b_0)$ only)

$$N_1 = \frac{f_{y0}\,t_0}{\sqrt{3}\sin\theta_1}\left[\frac{2h_1}{\sin\theta_1} + 2\,b_{ep}\right]$$

Bracing effective width, $N_1 = f_{y1}\,t_1\,[\,2h_1 - 4t_1 + 2b_{eff}\,]$ ($ß \geq 0.85$ only)

For $0.85 \leq ß \leq 1$ use linear interpolation between the capacity for chord face deformation at $ß = 0.85$ and the governing value for chord side wall failure (chord side wall buckling or chord shear) at $ß = 1.0$.

K- and N-gap joints

Chord face deformation, $N_i = \dfrac{6.3\, f_{y0}\, t_0^2}{\sin\theta_i} \dfrac{\sqrt{b_0}}{\sqrt{t_0}} \left[\dfrac{b_1 + h_1 + b_2 + h_2}{4b_0}\right] f(n)$

Chord shear between bracings, $N_i = \dfrac{f_{y0}\, A_v}{\sqrt{3}\, \sin\theta_i}$

Bracing effective width, $N_i = f_{yi}\, t_i\, [\, 2\, h_i - 4\, t_i + b_i + b_{eff}\,]$

Chord punching shear, $N_i = \dfrac{f_{y0}\, t_0}{\sqrt{3}\, \sin\theta_i} \left[\dfrac{2\, h_i}{\sin\theta_i} + b_i + b_{ep}\right]$
($\beta \le (1 - 2t_0\, /b_0)$ only)

The chord axial load resistance in the gap between the bracings (N_{0gap}) should also be checked if the factored shear load in the gap (V_{App}) is greater than 0.5 times the shear capacity (Vp).

$$N_{0gap} = f_{y0}[A_0 - A_v(2V_{App}/V_p - 1)^2]$$

K- and N-overlap joints

Only the overlapping member i need be checked. The efficiency of the overlapped member j should be taken as equal to that of the overlapping member.
i.e. $N_j = N_i\, (A_j\, f_{yj})/(A_i\, f_{yi})$

$b_i/b_j \ge 0.75$

$25\% \le O_v < 50\%$
Bracing effective width, $N_i = f_{yi}\, t_i\, [(O_v\, /\, 50)\, (2\, h_i - 4\, t_i) + b_{eff} + b_{eov}]$

$50\% \le O_v < 80\%$
Bracing effective width, $N_i = f_{yi}\, t_i\, [2\, h_i - 4\, t_i + b_{eff} + b_{eov}]$

$O_v \ge 80\%$
Bracing effective width, $N_i = f_{yi}\, t_i\, [2\, h_i - 4\, t_i + b_i + b_{eov}]$

5.2.4 RHS chords and CHS bracings with axial loads

For all the joints described in section 5.2.3, if the bracings are CHS replace the bracing dimensions, b_i and h_i, with d_i and multiply the resulting capacity by $\pi/4$ (except for chord shear).

5.2.5 RHS chords and RHS bracings with moments

Treat K- and N-gap joints as individual T- or Y-joints

5.2.5.1 T- and X-joints with in-plane moments

Chord face deformation, $M_{ip,1} = f_{y0}\, t_0^2\, h_1 \left[\dfrac{1 - \beta}{2 h_1/b_0} + \dfrac{2}{\sqrt{(1 - \beta)}} + \dfrac{h_1/b_0}{1 - \beta}\right] f(n)$
($\beta \le 0.85$ only)

Chord side wall crushing, $M_{ip,1} = 0.5\, f_{yk}\, t_0\, (h_1 + 5\, t_0)^2$
($0.85 \le \beta \le 1.0$ only) with $f_{yk} = f_{y0}$ for T-joints and 0.8 f_{y0} for X-joints

Bracing effective width, $M_{ip,1} = f_{y1}\, [W_{pl,1} - (1 - b_{eff}/b_1)\, b_1\, h_1\, t_1]$
($0.85 \le \beta \le 1.0$ only)

5.2.5.2 T- and X-joints with out-of-plane moments

Chord face deformation, $M_{op,1} = f_{y0}\, t_0^2 \left[\dfrac{h_1\,(1 + \beta)}{2\,(1 - \beta)} + \left[\dfrac{2b_0\, b_1 (1 + \beta)}{(1 - \beta)} \right]^{0.5} \right] f(n)$
($\beta \leq 0.85$ only)

Chord side wall crushing, $M_{op,1} = f_{yk}\, t_0\, (h_1 + 5\, t_0)\, (b_0 - t_0)$
($0.85 \leq \beta \leq 1.0$ only) with $f_{yk} = f_{y0}$ for T-joints and $0.8\, f_{y0}$ for X-joints

Bracing effective width, $M_{op,1} = f_{y1}\, [W_{pl,1} - 0.5(1 - b_{eff}/b_1)^2\, b_1^2\, t_1]$
($0.85 \leq \beta \leq 1.0$ only)

Chord distortional failure (lozenging), $M_{op,1} = 2f_{y0}\, t_0\, [h_1\, t_0 + (b_0\, h_0\, t_0\, (b_0 + h_0))^{0.5}]$
(T joints only)

5.2.6 RHS chords with gusset plates or I - or H -section bracings

Transverse gusset plate

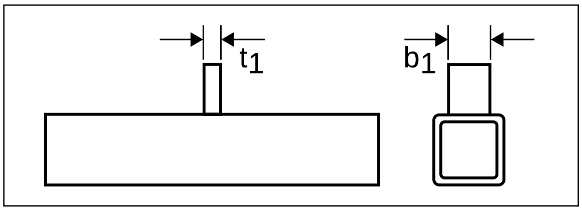


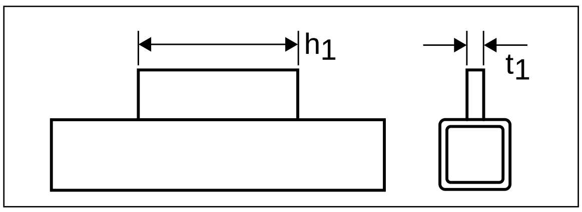
Plate effective width, $N_1 = f_{y1}\, t_1\, b_{eff}$

Chord side wall crushing, $N_1 = f_{y0}\, t_0\, (2\, t_1 + 10\, t_0)$
($b_1 \geq b_0 - 2\, t_0$ only)

Chord punching shear, $N_1 = \dfrac{f_{y0}\, t_0}{\sqrt{3}}\, (2\, t_1 + 2b_{ep})$
($b_1 \leq b_0 - 2\, t_0$ only)

In-plane moment = $M_{ip,1} = 0.5\, N_1\, t_1$
Out of plane moment = $M_{op,1} = 0.5\, N_1\, b_1$

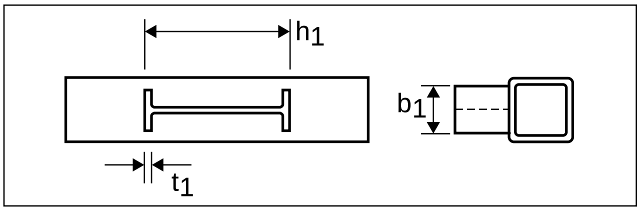
Longitudinal gusset plate



Chord face deformation, $N_1 = \dfrac{f_{y0}\, t_0^2}{1 - t_1/b_0}\, [2\, h_1/b_0 + 4\sqrt{(1 - t_1/b_0)}]\, f(m)$

In-plane moment, $M_{ip,1} = 0.5\, h_1\, N_1$
Out of plane moment, $M_{op,1} = 0.5\, N_1\, t_1$

I- or H-section bracings



Base axial load capacity, N1, upon two transverse plates, similar to it's flanges, as specified in 5.2.6 above, ie.

Plate effective width, $N_1 = 2\ f_{y1}\ t_1\ b_{eff}$

Chord side wall crushing, $N_1 = 2\ f_{y0}\ t_0\ (2\ t_1 + 10\ t_0)$
($b_1 \geq b_0 - 2\ t_0$ only)

Chord punching shear, $N_1 = \dfrac{2\ f_{y0}\ t_0}{\sqrt{3}}(2\ t_1 + 2\ b_{ep})$
($b_1 \leq b_0 - 2\ t_0$ only)

In-plane moment, $M_{ip,1} = 0.5\ (h_1 - t_1)\ N_1$

Out of plane moment, $M_{op,1} = 0.5\ N_1\ b_1$

5.3 Special joints in RHS

All dimensions used in the design formulae and parameter limits are nominal, except for Strongbox® 235 thicknesses which should use $0.9t_{nom}$ or (t_{nom} -0.5mm) whichever is the larger.

5.3.1 Welded knee joints

All members should be full plastic design sections. Loads should be predominantly moments with the factored applied axial load no greater than 20% of the member tension capacity.

Unreinforced knee joints (see figure 25)

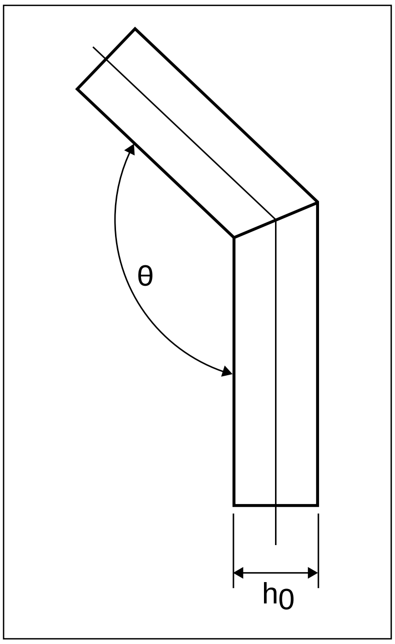


Figure 25 : Unreinforced knee joint

$$\frac{N_{app}}{Af_y} + \frac{M_{app}}{W_{pl}\ f_y} \leq \kappa$$

$$\theta \leq 90^\circ \text{ then } \kappa = \kappa_{90} = \frac{3\sqrt{(b_0/h_0)}}{(b_0/t_0)^{0.8}} + \frac{1}{1 + 2\ b_0/h_0}$$

$\theta > 90^\circ$ then $\kappa = \kappa_\theta = 1 - (\sqrt{2}\cos(\theta/2))\ (1 - \kappa_{90})$
κ_{90} and κ_θ are shown graphically in figs 26 and 27 respectively.

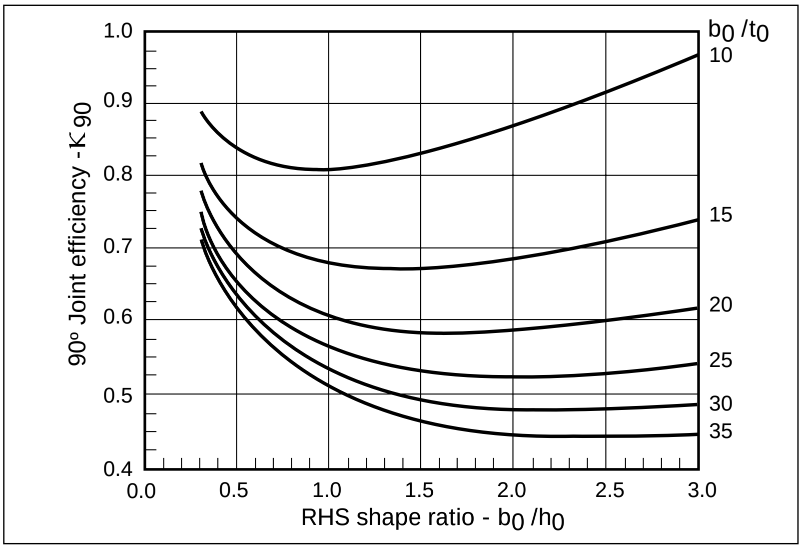


Figure 26 : Knee joint efficiency for $\theta \leq 90º$

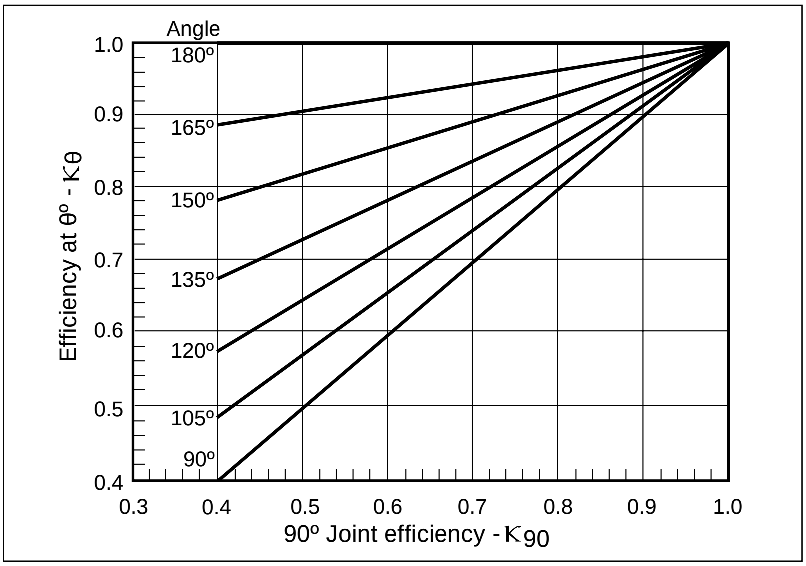


Figure 27 : Knee joint efficiency for $\theta > 90º$

Reinforced knee joints

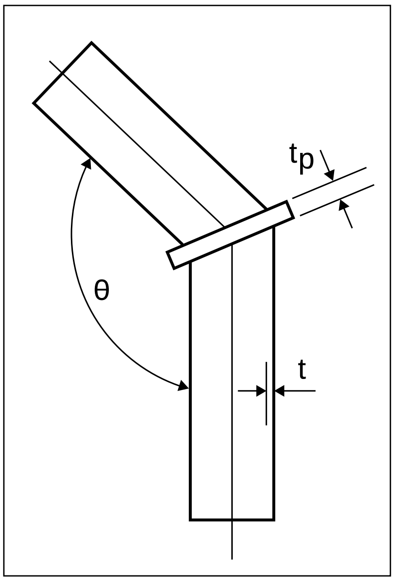


Figure 28 : Reinforced knee joint

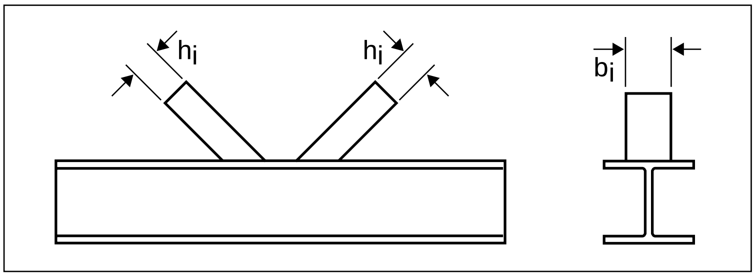
Knee joints can easily be reinforced by using a plate as shown in figure 28

If $t_p \geq 1.5$ t and $\geq$ 10mm
then the joint will be 100% efficient and

$$\frac{N_{app}}{A\,f_y} + \frac{M_{app}}{W_{pl}\,f_y} \leq 1.0$$

5.4 I - or H-section chord joints

All dimensions used in the design formulae and parameter limits are nominal, except for Strongbox® 235 thicknesses which should use $0.9t_{nom}$ or (t_{nom} -0.5mm) whichever is the larger.



5.4.1 I- or H-section chord joint parameter limits

<table>
<tr><th rowspan="2">Joint type</th><th rowspan="2">b_f / t_f</th><th rowspan="2">d_w / t_w</th><th colspan="2">$(b_i$ or h_i or $d_i) / t_i$</th><th rowspan="2">`Gap /lap</th><th rowspan="2">b_i / b_j</th></tr>
<tr><th>Compression</th><th>Tension</th></tr>
<tr><td>X-joints</td><td rowspan="4">≤
$20.7\sqrt{(275/f_{y0})}$</td><td>≤
$33.2\sqrt{(275/f_{y0})}$</td><td rowspan="2">b_i / t_i and
$h_i / t_i \le$
$30.4\sqrt{(275/f_{yi})}$</td><td rowspan="2">b_i / t_i
and h_i / t_i
≤ 35</td><td>-</td><td>-</td></tr>
<tr><td>T- and Y- joints</td><td rowspan="3">≤
$41.5\sqrt{(275/f_{y0})}$</td><td>-</td><td>-</td></tr>
<tr><td>K- and N- gap joints</td><td rowspan="2">$d_i / t_i \le$
$41.5\sqrt{(275/f_{yi})}$</td><td rowspan="2">d_i / t_i
≤ 50</td><td>gap ≥ $t_1 + t_2$
and ≤ $1.5(b_f\text{-}b_i)$</td><td>-</td></tr>
<tr><td>K- and N- lap joints</td><td>25% ≤ lap
≤ 100%</td><td>≥ 0.75</td></tr>
</table>

Table 8 : Joint Parameter limits

Note : 1) in gap joints, if the gap is greater than $1.5(b_f - b_i)$, then it should be treated as two separate T - or Y-joints (check for chord shear in the gap).

2) the web depth d_w should not be greater than 400mm.

5.4.2 I - or H - section chord joint functions

Bracing effective width functions

Normal effective width, $b_{eff} = t_w + 2\,r + 7\,t_f\,f_{y0} / f_{yi}$ but $\le b_i + h_i - 2t_i$ for RHS bracings,
$\le \pi\, d_i /2$ for CHS bracings

Web effective length, $b_w = h_i / \sin(\theta_i) + 5(t_f + r)$ but $\le 2\,t_i + 10(t_f + r)$

Overlap effective width, $b_{eov} = \dfrac{10}{b_j / t_j} \; \dfrac{f_{yj}\,t_j}{f_{yi}\,t_i} \; b_i$ but $\le b_i$

(Suffix 'j' indicates the overlapped bracing)

Chord shear area, A_V

The chord shear area, A_V, in K- and N-joints with a gap is dependant upon the type of bracings and the size of the gap

$$A_V = A_0 - (2 - \alpha)\, b_f\, t_f + (t_w + 2r)\, t_f$$

$$\text{with } \alpha = \left[\frac{1}{1 + \frac{4 g^2}{3 t_f^2}}\right]^{0.5} \quad \text{for RHS bracings}$$

and $\alpha = 0$ for CHS bracings

5.4.3 I - or H - section chords and RHS bracings with axial loads

T-, Y- and X-joints

Chord web yielding, $N_1 = f_{y0}\, t_w\, b_w / \sin(\theta_i)$

Bracing effective width, $N_1 = 2\, f_{y1}\, t_1\, b_{eff}$

K- and N-gap joints

Chord web yielding, $N_i = f_{y0}\, t_w\, b_w / \sin(\theta_i)$

$$\text{Chord shear, } N_i = \frac{f_{y0} A_V}{\sqrt{3} \sin(\theta_i)}$$

The bracing effective width failure criterion, below, does not need to be checked provided that :

$$g / t_f \le 20 - 28\,\beta \quad : \quad \beta \le 1.0 - 0.015\, b_f / t_f \quad : \quad 0.75 \le d_1 / d_2 \text{ or } b_1 / b_2 \le 1.33$$

Bracing effective width, $N_i = 2\, f_{yi}\, t_i\, b_{eff}$

The chord axial load resistance in the gap between the bracings (N_{0gap}) should also be checked if the factored shear load in the gap (V_{App}) is greater than 0.5 times the shear capacity (V_p).

$$N_{0gap} = f_{y0}[A_0 - A_V(2V_{App}/V_p - 1)^2]$$

K- and N-overlap joints

Only the overlapping member i need be checked. The efficiency of the overlapped member j should be taken as equal to that of the overlapping member.
i.e. $N_j = N_i\, (A_j\, f_{yj})/(A_i\, f_{yi})$

$25\% \le O_V < 50\%$
Bracing effective width, $N_i = f_{yi}\, t_i\, [(O_V / 50)(h_i - 2\, t_i) + b_{eff} + b_{eov}]$

$50\% \le O_V < 80\%$
Bracing effective width, $N_i = f_{yi}\, t_i\, [h_i - 2\, t_i + b_{eff} + b_{eov}]$

$O_V \ge 80\%$
Bracing effective width, $N_i = f_{yi}\, t_i\, [2\, h_i - 4\, t_i + b_i + b_{eov}]$

5.4.4 I - or H - section chords and RHS bracings with in-plane moments

T-, Y- and X-joints

Chord web yielding, $M_{ip,1} = 0.5\ f_{y0}\ t_w\ b_w\ h_1$

Bracing effective width, $M_{ip,1} = f_{y1}\ t_1\ b_{eff}\ (h_1 - t_1)$

K- and N-gap joints

Treat these as two separate T- or Y-joints.

5.4.5 I - or H - section chords and CHS bracings

For joints with CHS bracings use the above formulae but replace h_i and b_i with d_i and multiply the resulting capacities by $\pi/4$ (except chord shear).

6. Design examples

The example given here is for a simply supported, K-braced girder and is designed firstly for RHS and secondly for CHS members. For each joint being checked the joint parameters and the joint capacities for all possible failure modes must be calculated. The lowest capacity is then taken as the joint's actual capacity.

Note - this process can be undertaken quickly by the use of appropriate computer design software, for example [5].

6.1 Girder layout and member loads

Girder basic details

Span	25m
Number of panels	10
Bracing angles	55°
Depth	1.785m
Span / depth ratio	14
External loading	100kN factored load per panel point excluding ends
Material	Celsius® 275

The structural analysis has been based on the assumption that all member centre lines node, bracings are pinned and chords are continuous. The girder is symmetrical about its centre, so only half is shown here. The girder and member load details are shown in figures 29 and 30

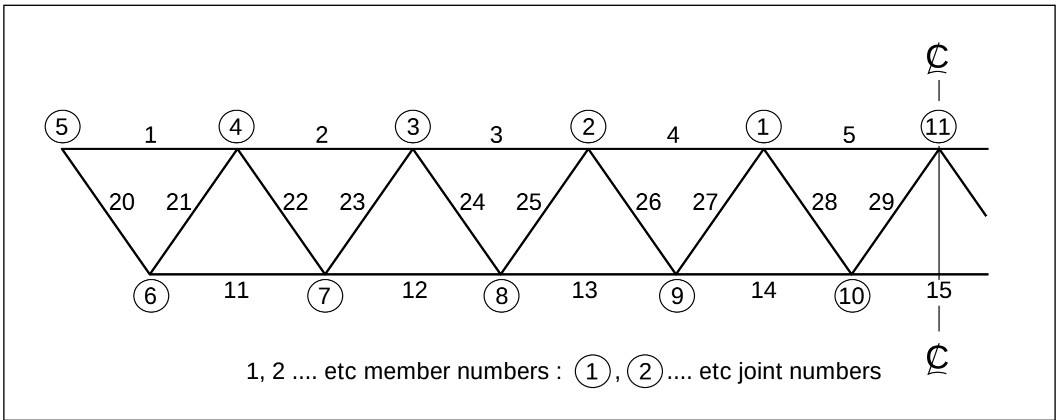


Figure 29: Girder layout, member and node numbering

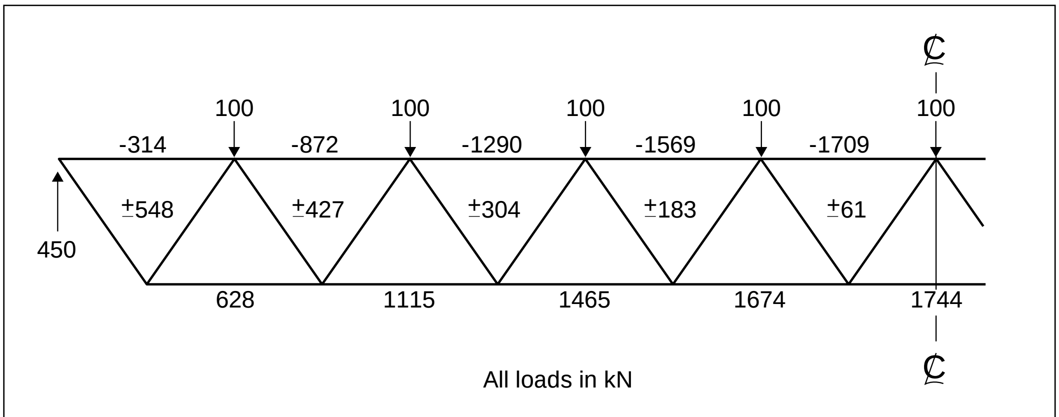


Figure 30: Applied member factored loads

6.2 Design philosophy

The following points should be born in mind when determining the member sizes and thicknesses.

1. Gap joints are more economic to fabricate than overlap joints.
2. For gap joints, smaller thicker chords give higher joint capacities than larger thinner ones.
3. For gap joints, larger thinner bracings give higher joint capacities than smaller thicker ones.
4. It is usually more economic to restrict the number of bracing sizes to about three, rather than to match every bracing to the actual load applied to it. This may not be so true if very large numbers of identical girders are to be produced.
5. The material can be obtained in 12.5m lengths, as a result the chords will be made from the same material throughout their length (other lengths are available).
6. The effective length factors for compression members have been taken as 0.9 for chords and 0.75 for the bracings between chord centres.
7. It is possible that in order to meet the joint parameter limits, it will be necessary to move away from member centre line noding. Any moment generated due to joint eccentricities can be considered to be distributed into the chord only with 50% taken on each side of the joint.

6.3 RHS girder design

6.3.1 RHS Member Selection Options

Top Chord : load -1709kN

Size	Mass	Capacity
180x180x10.0	53.0	-1793
150x150x12.5	53.4	-1767

Bottom Chord : load +1744kN

Size	Mass	Capacity
180x180x10.0	53.0	1857
150x150x12.5	53.4	1870

Bracing 20 : load +548kN

Size	Mass	Capacity
90x90x6.3	16.4	575
80x80x8.0	17.8	625
120x120x5.0	18.0	629

Bracing 21 : load -548kN

Size	Mass	Capacity
80x80x8.0	17.8	-577
120x120x5.0	18.0	-612

Bracing 22 : load +427kN

Size	Mass	Capacity
70x70x6.3	12.5	436
60x60x8.0	12.8	449
90x90x5.0	13.3	464

Bracing 23 : load -427kN

Size	Mass	Capacity
90x90x5.0	13.3	-439
80x80x6.3	14.4	-469
100x100x5.0	14.8	-497

Bracing 24 : load +304kN

Size	Mass	Capacity
90x90x3.6	9.72	340
70x70x5.0	10.1	354

Bracing 25 : load -304kN

Size	Mass	Capacity
90x90x3.6	9.72	-323
70x70x5.0	10.1	-321

Bracing 26 : load +183kN

Size	Mass	Capacity
60x60x3.0	5.34	187
40x40x5.0	5.40	189

Bracing 27 : load -183kN

Size	Mass	Capacity
70x70x3.0	6.28	-201
50x50x5.0	6.97	-190
60x60x4.0	6.97	-212

Bracing 28 : load +61kN

Size	Mass	Capacity
40x40x2.5	2.92	102

Bracing 29 : load -61kN

Size	Mass	Capacity
40x40x2.5	2.92	-67.0

6.3.2 RHS Member Selection

Chord selection

Top and bottom chords will both be 150x150x12.5, since this is smaller and thicker than 180x180x10.0 and is only 0.75% heavier.

Bracing selection

Minimum brace to chord width ratio is 0.35, so bracings must not be smaller than 52.5mm (0.35x150), from the size range available this means 60x60 minimum.

End bracings (20, 21): The lightest section to suit both bracing is 80x80x8, so this is selected.

Bracings 22, 23, 24 and 25: 90x90x5 are suitable for 22 and 23, this will also be used for 24 and 25, so that the inner four bracings can be made as light as possible.

Bracings 26, 27, 28 and 29: The lightest section to suit these is determined by member 27 so 70x70x3 is chosen for all.

6.3.3 RHS Joint Capacity Check

6.3.3.1 RHS Joint parameter check

The table below contains all of the parameter checks required for all of the joints in the girder.

Joint or member	Parameter	Limiting value	Actual value	Remarks
Chords	b_0/t_0	≤ 35	150/12.5 = 12	pass
Bracings	b_i/t_i	≤ 35 for tension ≤ 34.5 for compression	80/8 = 10 90/5 = 18 70/3 = 23.3	all pass
	b_1/b_0	≥ 0.35 and ≥ $0.1+0.01b_0/t_0$ = 0.22	80/150 = 0.53 90/150 = 0.60 70/150 = 0.47	all pass
Joints1, 9 and 10	gap	≥ $t_1 + t_2$ = 6 and ≥ $0.5(b_0-(b_1+b_2)/2)$ = 40 and ≤ $1,5(b_0-(b_1+b_2)/2)$ = 120	19.60	fail - increase to 40mm eccentricity = 14.6
Joint 2	gap	≥ $t_1 + t_2$ = 8 and ≥ $0.5(b_0-(b_1+b_2)/2)$ = 35 and ≤ $1,5(b_0-(b_1+b_2)/2)$ = 105	7.37	fail - increase to 40mm eccentricity = 23.3
Joints 3, 7 and 8	gap	≥ $t_1 + t_2$ = 10 and ≥ $0.5(b_0-(b_1+b_2)/2)$ = 30 and ≤ $1,5(b_0-(b_1+b_2)/2)$ = 90	-4.84 (overlap)	fail - increase to 40mm eccentricity = 32.0
Joint 4	gap	≥ $t_1 + t_2$ = 13 and ≥ $0.5(b_0-(b_1+b_2)/2)$ = 32.5 and ≤ $1,5(b_0-(b_1+b_2)/2)$ = 97.5	1.27	fail - increase to 40mm eccentricity = 27.7
Joint 6	gap	≥ $t_1 + t_2$ = 16 and ≥ $0.5(b_0-(b_1+b_2)/2)$ = 35 and ≤ $1,5(b_0-(b_1+b_2)/2)$ = 105	7.37	fail - increase to 40mm eccentricity = 23.3

In all cases it has been necessary to move away from member centre line noding in order to meet the gap parameter limits. However, the joints at the centre of the girder (1, 2, 9 and 10) have small shear forces and eccentricities and the chords, although they are subject to high axial forces, should be able to accommodate these. At the girder ends, the chords carry relatively small axial loads, and although the shear forces and eccentricities are higher, they should be able to carry the eccentricity moments.

6.3.3.2 RHS Joint capacity check

Generally, it is only necessary to check the capacity of selected joints, e.g. joints with the highest shear loads, joints with the highest chord compression loads or where the bracing or chord sizes change. Also, it should be noted that a tension chord joint will always have as high or a higher capacity than an identical compression chord joint, because the chord end load function is always 1.0 for tension chords, but is 1.0 or less for compression chords. Here, however, as an example, each joint has been checked for completeness.

The results of the joint capacity checks for the normal K-joints (all except 5 and 11) are given in the table below.

Joint number	**Factored applied load, kN**	**Calculated joint capacities, kN for failure modes**				**Joint unity factor**	**Gap mm**	**Ecc. mm**
		Chord face deformation	**Chord shear**	**Chord punching shear**	**Bracing effective width**			
Joint 1	N27 = -183	270.1	821.8	725.0	221.1	0.83	40	14.6
	N28 = 61	270.1	821.8	725.0	221.1	0.28		
Joint 2	N25 = -304	403.9	821.8	932.2	467.5	0.75	40	23.3
	N26 = 183	403.9	821.8	725.0	221.1	0.83		
Joint 3	N23 = -427	572.2	821.8	932.2	467.5	0.91	40	32.0
	N24 = 304	572.2	821.8	932.2	467.5	0.65		
Joint 4	N21 = -548	626.3	821.8	828.6	633.6	0.88	40	27.7
	N22 = 427	626.3	821.8	932.2	467.5	0.91		
Joint 6	N21 = -548	609.9	821.8	828.6	633.6	0.90	40	23.3
	N20 = 548	609.9	821.8	828.6	633.6	0.90		
Joint 7	N23 = -427	686.1	821.8	932.2	467.5	0.91	40	32.0
	N22 = 427	686.1	821.8	932.2	467.5	0.91		
Joint 8	N25 = -304	686.1	821.8	932.2	467.5	0.65	40	32.0
	N24 = 304	686.1	821.8	932.2	467.5	0.65		
Joint 9	N27 = -183	533.7	821.8	725.0	221.1	0.83	40	14.6
	N26 = 183	533.7	821.8	725.0	221.1	0.83		
Joint 10	N29 = -61	533.7	821.8	725.0	221.1	0.28	40	14.6
	N28 = 61	533.7	821.8	725.0	221.1	0.28		

The joints 5 and 11 can be regarded as special joints, and, although checked in a similar way to the others, certain assumptions regarding their behaviour have to be made.

Joint 5 is at the end of the girder and the chord will have an end plate of some type to connect it to the column. It has been shown that provided the plate thickness is the higher of either 10mm or the chord thickness (12.5mm in this case) that the joint will behave as a symmetrical K- or N-joint, rather than a weaker Y-joint. This is because the end plate will restrain the chord cross section from distorting.

Joint 11 should be treated in one of two different ways depending upon the method by which the two lengths of chord material are connected together at the joint.
(a) if the chord/chord connection is a bolted flange site connection, then joint 11 can be treated in a similar way to joint 5
(b) if the chord/chord connection is a butt weld, then joint 11 should be treated as a K-joint with both bracings loaded in compression.

The checks on joints 5 and 11 are given in the table below, in which joint 11a is as for case (a) above and joint 11b as for case (b) above.

Joint number	**Factored applied load, kN**	**Calculated joint capacities, kN for failure modes**				**Joint unity factor**
		Chord face deformation	**Chord shear**	**Chord punching shear**	**Bracing effective width**	
Joint 5	N20 = 548	609.9	821.8	828.6	633.6	0.90
Joint 11a	N29 = -61	270.1	821.8	725.0	221.1	0.28
Joint 11b	N29 = -61	143	-	-	-	0.43
	N30 = -61	143	-	-	-	0.43

Thus all the joints are within all the parameter limits, all the factored loads are below the respective joint capacities and the girder is satisfactory.

6.4 CHS girder design

6.4.1 CHS Member Selection Options

Top Chord : load -1709kN

Size	Mass	Capacity
323.9x6.3	49.3	-1718
219.1x10.0	51.6	-1801

Bottom Chord : load +1744kN

Size	Mass	Capacity
219.1x10.0	51.6	1806
273.0x8.0	52.3	1832

Bracing 20 : load +548kN

Size	Mass	Capacity
139.7x5.0	16.6	582
114.3x6.3	16.8	588

Bracing 21 : load -548kN

Size	Mass	Capacity
139.7x5.0	16.6	567
114.3x6.3	16.8	562

Bracing 22 : load +427kN

Size	Mass	Capacity
114.3x5.0	13.5	472

Bracing 23 : load -427kN

Size	Mass	Capacity
114.3x5.0	13.5	452

Bracing 24 : load +304kN

Size	Mass	Capacity
76.1x5.0	8.77	307
114.3x3.6	9.80	344

Bracing 25 : load -304kN

Size	Mass	Capacity
114.3x3.6	9.80	330
88.9x3.6	10.3	336

Bracing 26 : load +183kN

Size	Mass	Capacity
48.5x5.0	5.34	187
60.3x4.0	5.55	195

Bracing 27 : load -183kN

Size	Mass	Capacity
88.9x3.2	6.76	220
60.3x5.0	6.82	194

Bracing 28 : load +61kN

Size	Mass	Capacity
26.9x3.2	1.87	66
33.7x2.6	1.99	70

Bracing 29 : load -61kN

Size	Mass	Capacity
42.4x3.2	3.09	63
48.3x3.2	3.56	86

6.4.2 CHS Member Selection

Using the same procedure as for the RHS girder the following member sizes were selected.

Top and bottom chords : 219.1 x 10.0
Bracings 20 and 21 : 139.7 x 5.0
Bracings 22 to 25 : 114.3 x 5.0
Bracings 26 to 29 : 88.9 x 3.2

6.4.3 CHS Joint Capacity Check

Again, it has been assumed that gap joints will be used throughout the girder and initially that all centre lines node, although, in order to meet the joint parameter limits it will be necessary to move away from this.

6.4.3.1 CHS Joint parameter check

The table below contains all of the parameter checks required for all of the joints in the girder.

Joint or member	Parameter	Limiting value	Actual value	Remarks
Chords	d_0/t_0	≤ 50	219.1/10 = 21.9	pass
Bracings	d_i/t_i	≤ 50 for tension and compression	139.7/5 = 27.9 114.3/5 = 22.9 88.9/3.2 = 27.8	all pass
Bracing on chord	d_1/d_0	≥ 0.2	139.7/219.1 = 0.64 114.3/219.1 = 0.52 88.9/219.1 = 0.41	all pass
Joints 1, 9 and 10	gap	$\geq t_1 + t_2 = 6.4$	44.9	all pass
Joints 2	gap	$\geq t_1 + t_2 = 8.2$	29.4	pass
Joints 3, 4, 6, 7, and 8	gap	$\geq t_1 + t_2 = 10.0$	joint 3 & 7, g = 13.9	pass
			joint 8, g = 44.9	pass
			joint 4, g = -1.62	fail, increase gap to 12.5, ecc = 10.1
			joint 6, g = -17.1	fail, increase gap to 12.5, ecc = 21.2

6.4.3.2 CHS Joint capacity check

The joint capacity check procedure is the same as for the RHS girder joints, and the general notes for that girder still apply. The results of the joint capacity checks for the normal K-joints (all except 5 and 11) are given in the table below.

Joint number	Factored applied load, kN	Calculated joint capacities, kN, for failure modes		Joint unity factor
		Chord face deformation	Chord punching shear	
Joint 1	N27 = -183	185.2	601.1	0.99
	N28 = 61	185.2	601.1	0.33
Joint 2	N25 = -304	292.5	772.8	**1.04**
	N26 = 183	292.5	601.1	0.63
Joint 3	N23 = -427	387.3	772.8	**1.10**
	N24 = 304	387.3	772.8	0.78
Joint 4	N21 = -548	542.5	944.6	**1.01**
	N22 = 427	542.5	772.8	0.79
Joint 6	N21 = -548	577.7	944.6	0.95
	N20 = 548	577.7	944.6	0.95
Joint 7	N27 = -427	492.9	772.8	0.87
	N28 = 427	492.9	772.8	0.87
Joint 8	N25 = -304	360.9	601.1	0.84
	N24 = 304	360.9	601.1	0.84
Joint 9	N27 = -183	360.9	601.1	0.51
	N26 = 183	360.9	601.1	0.51
Joint 10	N29 = -61	360.9	601.1	0.17
	N28 = 61	360.9	601.1	0.17

Joints 2, 3 and 4 all fail due to the chord face deformation criterion by 4%, 10% and 1% respectively. Either member sizes or joint configurations will have to be changed, or the joints could be reinforced.

6.4.4 CHS Girder Reanalysis

There are various ways of increasing the capacity of the failed joints, for example :

1) Change the top chord to one diameter lower and one thickness higher, i.e. 193.7 x 12.5. This would increase the girder weight by 3.84%, it would also mean that the profiling at each end of a bracing would be different.

2) Change the compression bracings 21, 23 and 25 to one diameter up. This would increase the weight by 1.44% and, in this case, increase the number of bracing sizes used in the girder to four. New sizes would be member 21 - 168.3 x 5.0, members 23 and 25 - 139.7 x 5.0.

3) As 2) above, but rationalise the bracing sizes to give three sizes only. The new sizes would be members 20 and 21 - 168.3 x 5.0, members 22 to 25 139.7 x 5.0 and members 26 to 29 remaining as 88.9 x 3.2. This would increase the girder weight by 2.9%.

4) Reinforce the six failed joints by adding a saddle plate 12.5mm thick (see section 4.6.1). If only one or two joints are involved, this could be an economic solution.

5) Change the joints to overlap joints. This will increase fabrication costs since the ends of the bracings will require double profiling.

The actual choice from the above options will depend upon the circumstances of a particular project such as:- number of identical girders required, material available or in stock, relative costs of fabrication and materials, etc. In this particular case option 3) will be used.

6.4.4.1 Re-selection of CHS sizes

The actual section sizes will now be as follows

Chords both 219.1 x 10.0, as before
Bracings 20 and 21 - 168.3 x 5.0
Bracings 22 to 25 - 139.7 x 5.0
Bracings 26 to 29 - 88.9 x 3.2, as before

This results in an increase in the girder weight of 2.9%.

6.4.4.2 Revised CHS girder parameter limits and joint capacity checks

The joint parameter limits are all satisfied, and the joint capacity check is given in the table below. Due the size changes the bracing gaps result in different eccentricities of loading, these are also shown in the table.

Joint number	Factored applied load, kN	Calculated joint capacities, kN, for failure modes		Joint unity factor	Gap mm	Ecc. mm
		Chord face deformation	Chord punching shear			
Joint 1	N27 = -183	185.2	601.1	0.99	44.9	0.0
	N28 = 61	185.2	601.1	0.33		
Joint 2	N25 = -304	363.8	944.6	0.84	13.9	0.0
	N26 = 183	363.8	601.1	0.50		
Joint 3	N23 = -427	453.9	944.6	0.94	12.5	21.2
	N24 = 304	453.9	944.6	0.67		
Joint 4	N21 = -548	629.5	1138	0.87	12.5	33.6
	N22 = 427	629.5	944.6	0.68		
Joint 6	N21 = -548	670.3	1138	0.82	12.5	46.1
	N20 = 548	670.3	1138	0.82		
Joint 7	N27 = -427	577.7	944.6	0.74	12.5	21.2
	N28 = 427	577.7	944.6	0.74		
Joint 8	N25 = -304	577.7	944.6	0.53	12.5	21.1
	N24 = 304	577.7	944.6	0.53		
Joint 9	N27 = -183	360.9	601.1	0.51	44.9	0.0
	N26 = 183	360.9	601.1	0.51		
Joint 10	N29 = -61	360.9	601.1	0.17	44.9	0.0
	N28 = 61	360.9	601.1	0.17		

The joints with the most highly loaded chords, joints 1, 2, 9 and 10, have zero noding eccentricity and the chords will not have to carry any moment due to eccentricity. Where there is an eccentricity, the chords have relatively small axial loads (e.g. at joint 3 only 75% of its axial capacity) and will therefore also be able to carry the moment generated.

Although they have not been checked here, joints 5 and 11 would be checked using the same procedure as for the RHS girder.

Thus all the joints are within all the parameter limits, all the factored loads are below the respective joint capacities and the girder is satisfactory.

7. List of symbols

7.1 General alphabetic list

A_0, A_i	area of chord and bracing member i, respectively
A_v	shear area of chord
E	modulus of elasticity (205 000N/mm^2)
$M_{ip,i}$	joint design capacity in terms of in plane moment in bracing member i
$M_{ip,i,App}$	factored applied in plane moment in bracing member i
$M_{op,i}$	joint design capacity in terms of out of plane moment in bracing member i
$M_{op,i,App}$	factored applied out of plane moment in bracing member i
N_i	joint design capacity in terms of axial load in bracing member i
$N_{i,App}$	factored applied axial load in bracing member i
O_v	percentage overlap, q sinθ / h_i x 100%, see figure 31
V_{App}	factored applied shear load in gap between bracings
V_p	chord shear capacity
$W_{el,i}$	elastic section modulus of member i
$W_{pl,i}$	plastic section modulus of member i
a	fillet weld throat thickness
b_0, b_i	width of RHS chord and bracing member i, respectively
b_{eff}	effective bracing width, bracing to chord
b_{ep}	effective bracing width for chord punching shear
b_{eov}	effective bracing width, overlapping to overlapped bracing
b_f	I-section flange width
b_r	width of reinforcement plate for RHS chord
d_0, d_i	diameter of chord and bracing member i, respectively
d_s	arc length of saddle reinforcement plate for CHS chord
d_w	I-section web depth
e	joint eccentricity
f_{y0}, f_{yi}	nominal yield (design) strength of chord and bracing member i, respectively
g	gap / overlap between bracings at the chord face, a negative value denotes an overlap
h_0, h_i	height of RHS chord and bracing member i, respectively
h_r	length of reinforcement plate
q	overlap between bracings at the chord face
t_0, t_i	thickness of chord and bracing member i, respectively
t_f	I-section flange thickness
t_r	thickness of reinforcement plate
t_w	I-section web thickness
α	non-dimensional factor for the effectiveness of the chord face to carry shear
ß	mean bracing to chord width ratio, b_1/b_0 or d_1/d_0 or $\frac{b_1+b_2}{2b_0}$ or $\frac{d_1+d_2}{2d_0}$
γ	chord width to thickness ratio, $d_0/(2\ t_0)$ or $b_0/(2\ t_0)$
μ	multiplanar factor
θ_i	angle between bracing member i and the chord
κ	efficiency factor
σ_0	factored applied stress in RHS chord joint
σ_p	factored applied stress in CHS chord joint

Member identification suffices, i

0	the chord member
1	the compression bracing for joints with more than one bracing or the bracing where only one is present
2	the tension bracing for joints with more than one bracing
j	the overlapped bracing for overlapped bracing joints

7.2 Pictorial

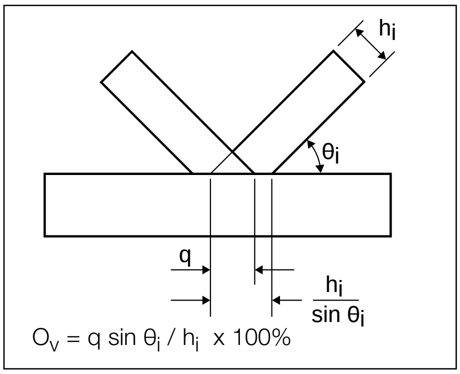


Figure 31 : Definition of percentage overlap

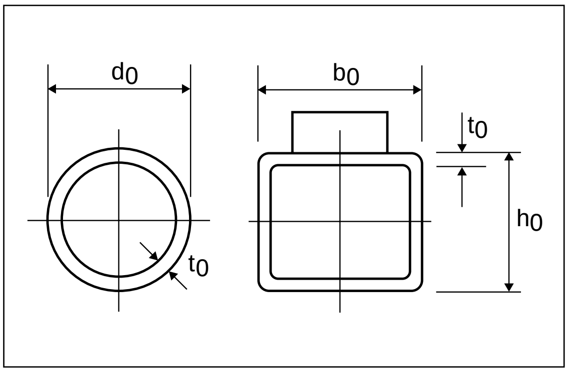


Figure 32 : Definition of SHS chord symbols

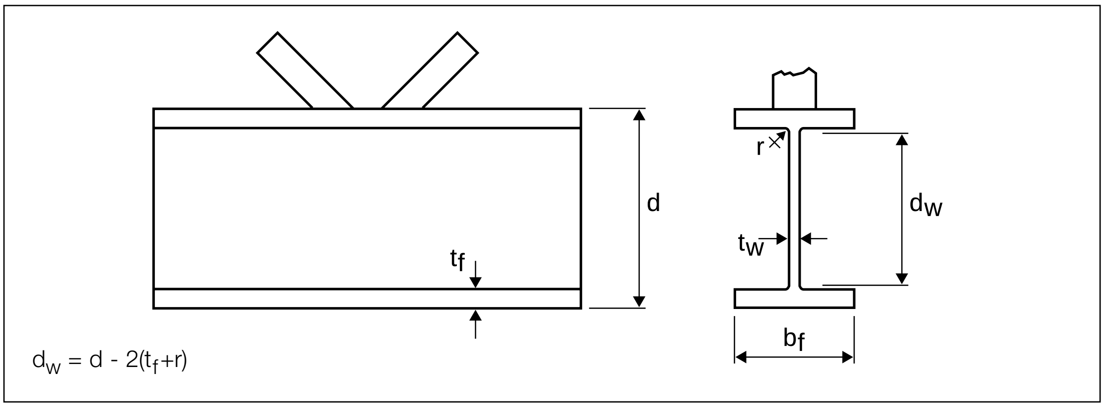


Figure 33 : Definition of I-section chord symbols

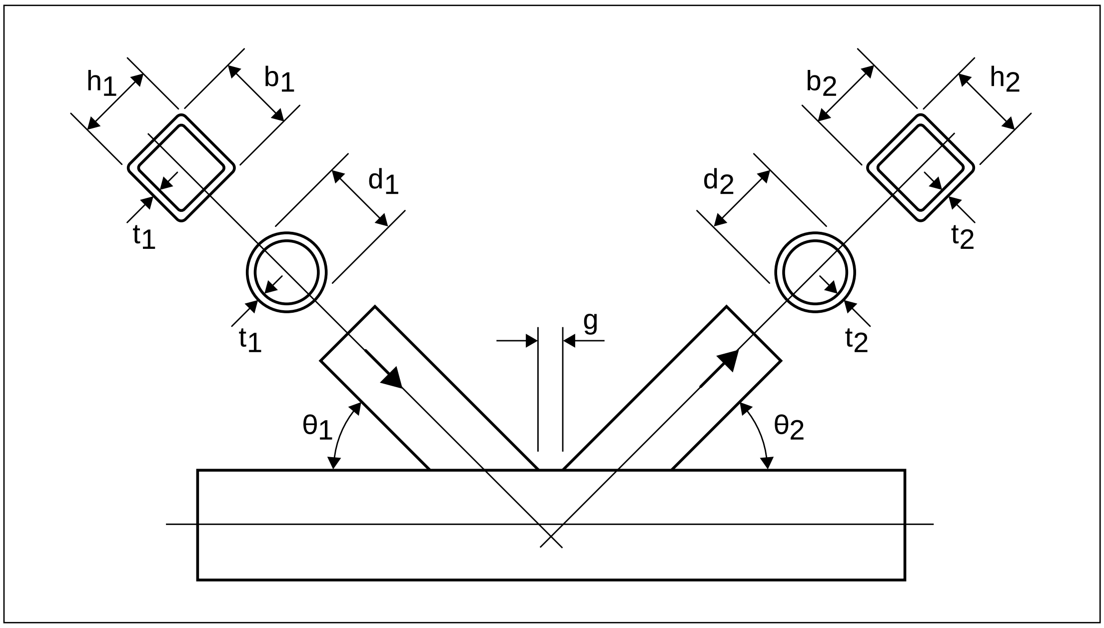


Figure 34 : Definition of bracing symbols

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*CIDECT design guides are available from the Steel Construction Institute, Silwood Park, Ascot, Berkshire, SL5 7QN, England. E-mail: publications@steel-sci.com. URL: http//www.steel-sci.org.

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English Language

CT16:1000:UK:08/2005