DESIGN GUIDE
FOR CIRCULAR HOLLOW SECTION (CHS) JOINTS UNDER PREDOMINANTLY STATIC LOADING

J. Wardenier, Y. Kurobame, J.A. Packer, D. Dutta, N. Yeomans

Verlag TÜV Rheinland
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Jaap Wardenier, Yoshiaki Kurobane, Jeffrey A. Packer,
Dipak Dutta, Noel Yeomans

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Preface

The necessity to solve the design problems concerning the versatile applications of hollow sections, which are somewhat supplementary to the general structural engineering with plates and open sections and apply particularly to this youngest member in the family of steel sections, led to the foundation of CIDECT in 1962 as an international organization of major hollow section manufacturers. The aim is to combine together all the resources worldwide from industry, universities and other national and international bodies for research and application of technical data, development of simple design and calculation methods and dissemination of the results of the researches by publications.

Since its inception CIDECT activities have been focussed on virtually all aspects of the hollow section design including buckling behaviour of empty and concrete-filled columns, static and fatigue strength of joints, aerodynamic properties, corrosion resistance and workshop fabrication. The results of the researches sponsored by CIDECT are available in extensive reports and monographs and have been incorporated into many national and international design recommendations e.g. DIN (Deutsche Industrie Normung – German Standard), NF (Norme Francaise – French Standard), BS (British Standard), ACON/CSCA (Canadian Standard), A/J (Architectural Institute of Japan), IWI (International Institute of Welding), EUROCODE 3 (draft) etc. This design guide for the design and calculation of circular hollow section joints in steel structures under predominantly static load is the first of a series, which CIDECT has planned to publish in the near future. Four further design manuals are now in preparation:

- Design guide for circular and rectangular hollow section joints under fatigue loading
- Structural stability of hollow sections.
- Design guide for rectangular hollow section joints under predominantly static loading
- Design guide for hollow section columns susceptible to fire

The design of the connections in welded latticed structures of structural hollow sections requires not only the knowledge about proper welding but also special insight into the connection behaviour mainly dependent on the connection configuration governed by the geometrical parameters. In order to secure the structural integrity of a hollow section connection, it is of vital importance that the dimensions of the constructional members as well as the configuration of the connection result in adequate deformation and rotation capacity. It was necessary to carry out extensive experimental investigations besides theoretical analysis to come to the proper understanding of the solution. Simple design formulae and constructional rules have been derived from these technical data obtained by the analytical and experimental research works.

The intention of this design guide is to communicate to the architects, structural engineers and constructors these simplified design methods with worked-out examples in order to enable them to construct a technically secure and economic steel structure in circular hollow sections.

We wish to express our hearty thanks to three of the outstanding personalities in the field of research of hollow section structures – Professor J. Wardenier of Delft University of Technology, The Netherlands, Professor Y. Kurobane of Kumamoto University, Japan and Professor J.A. Packer of University of Toronto, Canada, who kindly consented to participate in writing this guide.

Further, our thanks go to all CIDECT member firms, who made this design guide possible.

Dipak Dutta
Chairman of the Technical Commission
of CIDECT
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Lift shaft with glass houses supported by tubular lattice frames
1 General

Many examples in nature demonstrate the excellent properties of the circular hollow section as a structural element in resisting compression, tension, bending and torsion. Further, the circular hollow section has proved to be the best shape for elements subjected to wind-, water- or wave loading. The circular hollow section combines these characteristics with an architecturally attractive shape. Structures made of hollow sections have a smaller surface area than comparable structures of open sections. This, in combination with the absence of sharp corners, results in a better performance of corrosion protection. These excellent properties should result in light "open" designs with a small number of simple joints in which gussets or stiffening plates can often be eliminated. Since the joint strength is influenced by the geometrical properties of the members, optimum design can only be obtained if the designer understands the joint behaviour and takes it into account in the conceptual design. Although present the unit material cost of hollow sections is higher than that of open sections, this can be compensated by the lower weight of the construction, smaller painting area for corrosion protection and reduction of fabrication cost by the application of simple joints without stiffening elements. Many existing constructions in hollow sections show that tubular structures can economically compete with designs in open sections.

Over the last twenty five years CIDECT has initiated many research programmes in the field of tubular structures: e.g. in the field of stability, fire protection, wind loading, composite construction, and the static and fatigue behaviour of joints. The results of these investigations are available in extensive reports and have been incorporated into many national and international design recommendations with background information in CIDECT Monographs. Initially many of these research programmes were a combination of experimental and analytical research. Nowadays many problems can be solved in a numerical way and the use of the computer opens up new possibilities for developing the understanding of structural behaviour. It is important that the designer understands this behaviour and is aware of the influence of various parameters on structural performance.

This practical design guide shows how tubular structures under predominantly static loading should be designed in an optimum way, taking account of the various influencing factors. This guide concentrates on the ultimate limit states design of lattice girders or trusses. Joint resistance formulae are given and also presented in a graphical format, to give the designer a quick insight during conceptual design.

The graphical format also allows a quick check of computer calculations afterwards. The basic design rules for uni-planar joints (Fig. 6) satisfy the safety procedures e.g. used in the European Community and in Canada. The formulae for other types of joints are in a certain way related to those for the basic types of joints.
2 Design of tubular structures

2.1 Introduction

In designing tubular structures it is important that the designer considers the joint behaviour right from the beginning. Designing members e.g. of a girder based on member loads only may result in undesirable stiffening of joints afterwards. This does not mean that the joints have to be designed in detail at the conceptual design phase. It only means that chord and bracing members have to be chosen in such a way that the main governing joint parameters such as diameter ratio $d_1/d_2$, thickness ratio $t_1/t_2$, chord diameter to thickness ratio $d_1/t_1$, gap $g$ between bracings, overlap $O_t$ of bracings and angle $\theta$, provide an adequate joint strength and an economical fabrication.

Since the design is always a compromise between various requirements, such as static strength, stability, economy in fabrication and maintenance, which are sometimes in conflict with each other, the designer should be aware of the implications of a particular choice. The following guidance is given to arrive at optimum design:

- Lattice structures can usually be designed assuming pin jointed members. Secondary bending moments due to the actual joint stiffness can be neglected for static design if the joints have sufficient rotation capacity. This will be the case if the joint parameters are within the range recommended in this design guide.

- It is common practice to design the members with the centre lines nodding. However, for ease of fabrication it is sometimes required to have a certain nodding eccentricity. If this eccentricity is kept within the limits $-0.55 \leq e/d_2 \leq 0.25$ indicated in Fig. 1, the resulting bending moments can be neglected for joint design and for chord members loaded in tension.

Chord members loaded in compression, however, have always to be checked for the bending effects of nodding eccentricity (i.e. designed as beam-columns, with all of the moment due to nodding eccentricity distributed to the chord sections).

Full overlapping results in an eccentricity $e = -0.55 d_2$ but provides a more straightforward fabrication than partial overlap joints and a better girder behaviour than gap joints.

![Fig. 1 - Nodding eccentricity](image)

- Secondary bending moments due to the end fixities of the members can be generally omitted with respect to design of both members and connections, provided there is adequate deformation and rotation capacity in both members and connections. This can be
achieved by limiting the wall slenderness of certain members, particularly the compression bracing members, which is the basis for some of the geometric limits of validity shown in Fig. 3.

- Gap joints are preferred to partial overlap joints (Figs. 1C and 2) since the fabrication is easier with regard to end cutting, fitting and welding. However, fully overlapped joints (Fig. 1D) provide better joint strength with similar fabrication than gap joints. The gap g is defined as the distance measured along the length of the connecting face of the chord, between the toes of the adjacent bracing member (ignoring welds). The percentage overlap O, defined in Fig. 2, is such that the dimension p pertains to the overlapping bracing.

In good designs a minimum gap should be provided such that \( g \geq t_1 + t_2 \) so that the welds do not overlap each other; on the other hand, in overlap joints the overlap should be at least \( O \geq 25\% \).

![Fig. 2 – Gap and overlap](image)

- In common lattice structures, (e.g. trusses), about 50% of the material weight is used for the chords in compression, roughly 30% for the chord in tension and about 20% for the web members or bracings. This means that with respect to material weight, the chords in compression should likely be optimised to result in thin walled sections. However, for corrosion protection (painting) the outer surface area should be minimized. Furthermore joint strength increases with decreasing chord diameter to thickness ratio \( d_c/t_c \) and increasing chord thickness to bracing thickness ratio \( t_c/t_b \). As a result the final diameter to thickness ratio \( d_c/t_c \) for the chord in compression will be a compromise between joint strength and buckling strength of the member and relatively stocky sections will usually be chosen. For the chord in tension the diameter to thickness ratio \( d_c/t_c \) should be chosen to be as small as possible.

- Since the joint strength efficiency (i.e. joint strength divided by the bracing yield load \( A_b \cdot f_y \)) increases with increasing chord to bracing thickness \( t_c/t_b \), this ratio should be chosen to be as high as possible. Furthermore the weld volume required for a thin walled bracing is smaller than that of a thick walled bracing with the same cross section.

- Since the joint strength also depends on the yield stress of the chord, the use of higher strength steel for chords (when available and practical) may offer economical possibilities.

2.2 Design procedure

The design of tubular structures should be approached in the following way to obtain an efficient and economical structure:

- Determine structure or truss geometry keeping the number of joints to a minimum.
- Determine member forces assuming pinned joints and nodding centre lines.
- Determine chord member sizes considering axial loading, corrosion protection and joint geometry (usual \( d_c/t_c \) ratios are 20 to 30). Usually an effective buckling length of 0.9 times
the system length is assumed if supports in-plane and out-of-plane are available at the joints [16].

- The use of high strength steel ($f_y = 355$ N/mm²) for the chords should be considered. The delivery time of the required sections has to be checked.
- Determine bracing member sizes, (based on axial loading), preferably with thicknesses smaller than the chord thickness.
- The effective length for the bracings can be assumed conservatively to be 0.75 times the system length [16,32,33]. A more precise calculation method for the effective length is given in chapter 6.4.
- Standardize the bracing members to a few selected dimensions (or even two) to minimize the number of the section sizes for the structure. Due to aesthetic reason one outer diameter with differentiated wall thicknesses may be preferred.
- Check joint geometry with regard to eccentricity limits and fabrication.
- Check joint efficiency with the diagrams given in chapter 4. From a fabrication point of view gap joints are preferred to overlap joints.
- If the joint strengths are not adequate, change the bracing or chord dimensions. Only a few joints will normally require to be checked.
- Check the effects of eccentricity noding moments (if any) on the chord members, by checking the moment-axial force interaction.
- If required, check truss deflections, at the unfactored load level, by analyzing the truss as a pin-connected frame if it has noding non-overlapped joints. If joints are overlapped throughout, check the truss deflection by assuming continuous chord members and pin-ended bracing members taking account of the eccentricity.
3 Fabrication of Tubular Structures

In designing tubular structures the designer should keep in mind that the costs of the structure are significantly influenced by the fabrication costs. This means that cutting, end preparation and welding costs should be minimized.
- Taking account of the standard mill lengths in design may reduce the end to end connections of chords. For large projects it may be agreed that special lengths are delivered.
- The end profile cutting of tubular members which have to fit other tubular members, as shown in Fig. 5 is normally done by automatic flame cutting (see Fig. 3). However, if such equipment is not available especially for small sized tubular members, other methods do exist, such as single, double or triple plane cuttings as shown in Fig. 4 [1, 4, 24].

Fig. 3 – Automatic flame cutting

- In a tubular joint, fillet welds, full penetration butt welds or fillet/butt welds are applied depending on the geometry as shown in Fig. 5. When welds are used, these have to be designed on the basis of the strength of the member to be connected. They have to be considered as automatically prequalified for any member load.
- The weld at the toe of the bracing is most important. If the bracing angle is less than 60°, the toe should always be bevelled and a butt weld used as shown in Fig. 5–C2.
- To allow proper welding at the heel of the bracing the bracing angle should not be less than 30°.
- Since the welding volume is proportional to $t^2$ thin walled bracings can generally be welded more economically than thick walled bracings.
- A minimum gap limit of $t_1 + t_2$ is recommended for K and N joints to ensure that adequate space is available to enable welding at the bracing toes to be performed satisfactorily.
Sizes of CHS bracings which can be fitted to CHS main members with a single cut; \( d_1 \) must be equal to or greater than \( 0.08 d_1^2 + 3 \) (with \( d_1 \) in mm)

<table>
<thead>
<tr>
<th>diameter of main ( d_0 ) (mm)</th>
<th>size of bracing ( d_1 ) up to and including straight cut</th>
<th>CHS dia. ( d_1 ) (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>33.7</td>
<td>-</td>
<td>26.9</td>
</tr>
<tr>
<td>42.4</td>
<td>-</td>
<td>26.9</td>
</tr>
<tr>
<td>48.3</td>
<td>-</td>
<td>26.9</td>
</tr>
<tr>
<td>60.3</td>
<td>-</td>
<td>33.7</td>
</tr>
<tr>
<td>76.1</td>
<td>-</td>
<td>33.7</td>
</tr>
<tr>
<td>88.9</td>
<td>42.4</td>
<td>48.3</td>
</tr>
<tr>
<td>114.3</td>
<td>33.7</td>
<td>48.3</td>
</tr>
<tr>
<td>139.7</td>
<td>33.7</td>
<td>60.3</td>
</tr>
<tr>
<td>166.3</td>
<td>42.4</td>
<td>60.3</td>
</tr>
<tr>
<td>193.7</td>
<td>48.3</td>
<td>60.3</td>
</tr>
<tr>
<td>219.1</td>
<td>48.3</td>
<td>60.3</td>
</tr>
<tr>
<td>323.9</td>
<td>60.3</td>
<td>60.3</td>
</tr>
<tr>
<td>355.6</td>
<td>60.3</td>
<td>60.3</td>
</tr>
<tr>
<td>406.4</td>
<td>60.3</td>
<td>76.1</td>
</tr>
<tr>
<td>457.0</td>
<td>60.3</td>
<td>76.1</td>
</tr>
<tr>
<td>506.0</td>
<td>60.3</td>
<td>76.1</td>
</tr>
</tbody>
</table>

(all dimensions are in mm)

Fig. 4 – Single, double or triple plane cuttings

Fig. 5 – Weld details
- From a fabrication point of view gap joints are preferred to overlap joints not only because the cutting and end preparation are easier but also because of tolerances and inspection.
- In partially overlapped joints the toe of the overlapped member ("hidden part") is usually not welded.

If the bracing load components perpendicular to the chord wall are rather unbalanced (e.g. exceed a factor of 1.5) it is recommended that the most heavily loaded member is the through bracing with its full circumference being welded to the chord, that means also the hidden part has to be welded.

---

**Fig. 6 – Various types of flattening**

Especially for small sized tubular structures, or in those cases where the fabricator does not have proper equipment for end profile cutting (partial), flattening of the ends of members can be used as shown in Fig. 6. More detailed information regarding fabrication is given in refs. [1, 426].

---

Transparent roof with tubular trusses and columns for a Tropic Bush Garden
4 Joint design under predominantly static loading

4.1 Introduction

All joint design strength formulae given in this guide are developed in ultimate limit state terms. This means that the effect of the characteristic loads $Q_k$ multiplied by appropriate load factors $\gamma_k$ should not exceed the joint design strength $N^*$, i.e.

$$\gamma_k \cdot Q_k \leq N^*$$

where $N^* = \frac{N_k}{\gamma_m}$

If the allowable load (or allowable stress) method is used, the joint design strengths should be divided by the load factor $\gamma_k$ applicable, i.e.

$$Q_k \leq \frac{N_k}{\gamma_k \cdot \gamma_m}$$

In this case $\gamma_k = 1.5$ is recommended.

The chord, bracing and joint symbols generally used are indicated in Fig. 7 for uni-planar joints and are defined in chapter 7.

Fig. 7 – Chord, bracing and joint symbols
Roof structure for an automobile exhibition hall
The joint design strength formulae incorporating the effect of the value of $\gamma_m$ are given in tables as well as in diagrams [11]. The formulae given in Fig. 8 can be used for computer calculations whereas the diagrams of Figs. 9 to 12 are very helpful in design and for a quick check of computer calculations.

In the diagrams the joint strength is expressed in terms of the efficiency of the connected bracings, i.e. the joint strength for axially loaded joints $N^*$ is divided by the yield load $A_i \cdot f_{yi}$ of the connected bracing.

This results in efficiency formulae of the following type:

$$\frac{N^*}{A_i \cdot f_{yi}} = C_e \cdot \frac{f_{yo}}{f_{yi}} \cdot \frac{t_o}{t_i} \cdot \frac{f(n')}{\sin \Theta_i}$$  \hspace{1cm} (4.1.1)

The efficiency parameter $C_e$ is given for each type of joint in diagrams as a function of the diameter ratio $\beta$ and the chord diameter/thickness ratio $d_o/t_o$.

The value of the parameter $C_e$ in the formula above gives the efficiency for the bracing of a joint with a tensile prestress loading in the chord $f(n') = 1.0$, a bracing angle $\Theta_i = 90^\circ$ and the same wall thickness and design yield stress for chord and bracing.

From the efficiency equation it can be easily observed that yield stress and thickness ratio between chord and bracing are extremely important for an efficient material use of the bracing. Decreasing the angle $\Theta_i$ increases the efficiency. The function $f(n')$ depends on the chord loading ($f(n') \leq 1.0$ for compression prestressing). The efficiency formula shows directly that the following measures are favourable for the joint efficiency:

- higher strength steel for chords than for the bracings ($f_{yo} > f_{yi}$)
- bracing wall thickness as small as possible ($t_i < t_o$) but such that the limits for local buckling or interaction are satisfied, see chapter 4.2.
- angle $\Theta_i > 90^\circ$; hence, prefer K-joints to N-joints.

For moment loading the design formulae are shown in Fig. 19. The respective design charts are given in Figs. 20 and 21. In these charts the joint efficiency is based on the plastic yield moment capacity $M_{pl}$ of the bracings. Here the same rules apply for an efficient design as those mentioned for axially loaded joints.

![Tubular triangular trusses for a highway tax paying station](image-url)
<table>
<thead>
<tr>
<th>Type of joint</th>
<th>Design strength ($i = 1, 2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T- and Y-joints</td>
<td>chord plastification</td>
</tr>
<tr>
<td><img src="image" alt="Diagram" /></td>
<td>$N_i = \frac{f_{pe} \cdot b}{\sin \theta_i} \cdot (2.8 + 14.2 \beta) \cdot \gamma^{1.2} \cdot f(n')$ (eq. 4.2.1)</td>
</tr>
<tr>
<td>X-joints</td>
<td>chord plastification</td>
</tr>
<tr>
<td><img src="image" alt="Diagram" /></td>
<td>$N_i = \frac{f_{pe} \cdot b}{\sin \theta_i} \left[ \frac{5.2}{1 - 0.81 \beta} \right] \cdot f(n')$ (eq. 4.2.2)</td>
</tr>
<tr>
<td>K and N gap or overlap joints</td>
<td>chord plastification</td>
</tr>
<tr>
<td><img src="image" alt="Diagram" /></td>
<td>$N_i = \frac{f_{pe} \cdot b}{\sin \theta_i} \left( 1.8 + 10.2 \frac{d_i}{d_o} \right) \cdot f(\gamma, g') \cdot f(n')$ (eq. 4.2.3)</td>
</tr>
<tr>
<td></td>
<td>$N_2 = N_i \cdot \frac{\sin \theta_i}{\sin \theta_2}$ (eq. 4.2.4)</td>
</tr>
<tr>
<td>general</td>
<td>punching shear</td>
</tr>
<tr>
<td><img src="image" alt="Diagram" /></td>
<td>$f(n') = 1.0$ for $n' &gt; 0$  $n' = \frac{f_{pc}}{f_{co}}$ (tension)  $f(\gamma, g') = \gamma^{1.2} \cdot \left[ 1 + \frac{0.024 \gamma^{1.2}}{\exp(0.5 g' - 1.33) + 1} \right]$ (eq. 4.2.5)</td>
</tr>
<tr>
<td>punching shear check for T, Y, X and K, N, KT joints with gap</td>
<td>$N_i = \frac{f_{pc}}{\sqrt{3}} \cdot \frac{1 + \sin \theta_i}{2 \sin^2 \theta_i}$</td>
</tr>
<tr>
<td>functions</td>
<td>$f(n') = 1 + 0.3 n' - 0.3 n'^2$ for $n' &lt; 0$ (compression) (see Fig. 13)</td>
</tr>
<tr>
<td>validity ranges</td>
<td>$0.2 &lt; \frac{d_i}{d_o} \leq 1.0$  $\frac{d_i}{2t_1} \leq 25$  $30^\circ \leq \theta_i \leq 90^\circ$  $-0.55 \leq \frac{\theta_i}{\theta_o} &lt; 0.25$  $\gamma \leq 25$  $\gamma \leq 20$ (X-joints)  $O_v \geq 25%$  $g \geq t_1 + t_2$</td>
</tr>
</tbody>
</table>

Fig. 8 – Design recommendations for uni-planar joints

20
4.2 Joints in uni-planar trusses

Typical uni-planar joints are illustrated in Fig. 7. The most recent design recommendations for uni-planar T-, X- and K-joints are given in Fig. 8. These formulae have also been adopted by the International Institute of Welding and by the Eurocode 3 Drafting Committee. Most of these formulae are based on the basic formulae originally developed by Kurobane [9, 10]. The design formulae for T-, Y- and X-joints have been based on the strength under compression loading but can also be used for tensile loading. The ultimate resistance under tensile loading is usually higher than under compressive loading, however, it is not always possible to take advantage of this strength due to large deformations or due to premature cracking. The strength of other types of joint configurations not given in Fig. 6 can be related to these basic types as will be shown in section 4.6.

The design strength is generally governed by two criteria, i.e., plastification of the chord cross section and chord punching shear. In order to design a joint, both criteria have to be checked according to the formulae in Fig. 8. These design strengths are presented graphically in terms of bracing efficiency in Figs. 9 to 12. These figures show that punching shear (horizontal cut off of the curves) only becomes critical for joints with thick chords (low d/t1 ratio) and generally in combination with low k ratio. The horizontal cut off for punching shear in Figs. 9 to 12 is conservative if the bracing angle θ < 90°.

The most common types of T- and X-joints are those with 90° angle between bracing and chord axes. The graphs and the examples show that T- and X-joints are less efficient than K-joints, especially for high d/t1 ratios. However, these types of joints are less important in common tubular structures. K- and N-joints are the common types of joints used in tubular structures. Figs. 11 and 12 show four design diagrams i.e. with gaps of 2d, 6d, 10d respectively and with overlap. The effect of the gap or overlap is also shown in Fig. 11. It can be observed that overlapping of the bracings is especially efficient for thin walled chords. As shown in Fig. 11, for the design of gap K-joints an initial value Cg = 0.3 can be used as a design basis for k ratios of 0.4 to 1.0, d/t1 ratios of 20 to 30 and a relative gap g/t1 of 4 to 10. To minimize the number of joints and to allow good welding, a bracing angle θ of about 40° will be efficient. For tension loaded chords with f(n) = 1.0, and with θ1 = 40°, the bracing can be fully effective if t1/t1 is larger than about 2.0. If the chords are made of steel with a higher yield stress than that of the bracings the thickness ratio may need to be even lower, i.e.

\[ \frac{f_{\text{e}}}{f_{\text{m}}} : \frac{f_{\text{c}}}{f_{\text{m}}} \geq 2.0. \] (4.2.7)

The design charts 1, 2 and 4 and Figs. 9, 10 and 12 show the function f(n). It should be noted here that only the prestress of the chord has to be considered; thus the horizontal bracing load components have to be extracted, as shown in Fig. 14.

For continuous girders which are simply supported at the ends of the span, the prestressing is small at the girder ends whereas the bracing loads are highest and the prestressing is high where the bracing loads are low (in the centre).

For continuous lattice girders the effect of f(n) needs special attention at the supports. K-, N- and KT-joints with external cross chord loading (e.g. through purlin loads), can be calculated using the criteria for K-joints by checking the larger normal component of the bracing force. If, however, all the bracing loads act either in tension or in compression (in the same sense) or if only one bracing is load bearing, the joint should be checked as an X-joint (see also Fig. 24). The KT-type and other types are dealt with in chapter 4.6.

To avoid interaction between bracing local buckling and joint strength it is recommended to limit the joint strength efficiencies by the compression bracing for high bracing diameter to wall thickness ratios d/t1.
Design chart 1  Tubular joints

T- and Y-joints of circular hollow sections

<table>
<thead>
<tr>
<th>symbols</th>
<th>ranges of validity</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\beta$</td>
<td>$0.2 \leq \beta \leq 1.0$</td>
</tr>
<tr>
<td>$\gamma$</td>
<td>$\frac{d_0}{t_0} \leq 50$</td>
</tr>
<tr>
<td>$n'$</td>
<td>$\frac{d_1}{t_1} \leq 50$</td>
</tr>
<tr>
<td>$f_{yo}$</td>
<td>$f_{y1} \leq 355 \text{ N/mm}^2$</td>
</tr>
</tbody>
</table>

$f_{yo}$ - chord stress as a result of additional axial force or bending moment

welds are to be dimensioned on the yield strength of the bracing

Efficiency of T- and Y-joints

\[
\frac{N_1}{A_1 - f_{y1}} = C_T \cdot \frac{f_{yo} \cdot t_0}{f_{y1} \cdot t_1 \cdot \sin \theta_1 \cdot f(n')} \cdot \frac{1}{d_0/t_0}
\]

Fig. 9 – Design chart for T- and Y-joints of circular hollow sections
Function \( f(n') \)

\[
\begin{align*}
\text{for } n' \geq 0: & \quad f(n') = 1 \\
\text{for } n' < 0: & \quad f(n') = 1
\end{align*}
\]

<table>
<thead>
<tr>
<th>Calculation Example</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chord: ( \varnothing 219.1 \times 10.0 )</td>
</tr>
<tr>
<td>Bracing: ( \varnothing 168.3 \times 4.5 )</td>
</tr>
<tr>
<td>( f_o = f_{o1}; \ \theta_t = 90^\circ )</td>
</tr>
<tr>
<td>( \beta = d_s/d_b = 168.3 )</td>
</tr>
<tr>
<td>( 219.1 )</td>
</tr>
<tr>
<td>( f(n') = 0.79 )</td>
</tr>
<tr>
<td>( C_t = 0.35 )</td>
</tr>
<tr>
<td>( d_s/l_b = \frac{219}{10} = 21.9 )</td>
</tr>
<tr>
<td>( N_1^* )</td>
</tr>
<tr>
<td>( A_1 \cdot l_p = 4.5 \cdot 0.79 = 0.61 )</td>
</tr>
</tbody>
</table>

Fig. 9 – contd. – Design chart for T- and Y-joints of circular hollow sections

22
Design chart 2. Tubular joints

X-joints of circular hollow sections

<table>
<thead>
<tr>
<th>symbols</th>
<th>ranges of validity</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\beta = \frac{d_1}{d_0}$</td>
<td>$0.2 \leq \beta \leq 1.0$</td>
</tr>
<tr>
<td>$\gamma = \frac{d_0}{2t_0}$</td>
<td>$\frac{d_0}{t_0} \leq 40$</td>
</tr>
<tr>
<td>$n' = \frac{f_{\text{y0}}}{f_{\text{y1}}}$</td>
<td>$\gamma \leq 50$</td>
</tr>
<tr>
<td>$f_{\text{y1}} = 355 \text{ N/mm}^2$</td>
<td>$30^\circ \leq \theta_1 \leq 90^\circ$</td>
</tr>
</tbody>
</table>

$f_{\text{y0}}$ = chord stress as a result of additional axial force or bending moment

welds are to be dimensioned on the yield strength of the bracing

Efficiency of X-joints

Fig. 10 - Design chart for X-joints of circular hollow sections
Function $f(n')$

For $n' \geq 0$: $f(n') = 1$

Für $n' \geq 0$: $f(n') = 1$

Calculation example

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>chord:</strong></td>
<td></td>
</tr>
<tr>
<td>$\odot 219.1 \times 10.0$</td>
<td></td>
</tr>
<tr>
<td>$d_1/t_h = 21.9$</td>
<td></td>
</tr>
<tr>
<td><strong>bracing:</strong></td>
<td></td>
</tr>
<tr>
<td>$\odot 168.3 \times 5.6$</td>
<td></td>
</tr>
<tr>
<td>$d_1/t_h = 30.0$</td>
<td></td>
</tr>
<tr>
<td>$t_{op} = t_{pl}$; $\theta_1 = 90^\circ$</td>
<td></td>
</tr>
<tr>
<td>$t_{op} = -0.48 t_{pl}$</td>
<td></td>
</tr>
<tr>
<td>$\beta = d_1/d_2 = \frac{168.3}{219.1} = 0.77$</td>
<td></td>
</tr>
<tr>
<td>$f(n') = 0.79$</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>$C_x = 0.26$</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>$d_2/t_h = \frac{219}{10} = 21.9$</td>
<td></td>
</tr>
<tr>
<td>$\sin \theta_1 = 1.0$</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>$N^*<em>{A_1} = \frac{A \cdot t_h}{t</em>{pl} \cdot 0.79} = 0.26 \cdot 0.79 = 0.37$</td>
<td></td>
</tr>
</tbody>
</table>

Fig. 10 – contd. – Design chart for X-joints of circular hollow sections
Design chart 3  Tubular joints

K- and N-joints with gap of circular hollow sections

<table>
<thead>
<tr>
<th>symbols</th>
<th>ranges of validity</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\beta = \frac{d_1 + d_2}{2d_0}$</td>
<td>$0.2 \leq \frac{d_1}{d_0} \leq 1.0$</td>
</tr>
<tr>
<td>$\gamma = \frac{d_0}{2t_0}$</td>
<td>$d_1 \leq 50$ ($i = 0, 1, 2$)</td>
</tr>
<tr>
<td>$n' = \frac{f_{yo}}{f_y}$</td>
<td>$f_y \leq 355$ N/mm$^2$</td>
</tr>
<tr>
<td>$g' = \frac{g}{t_0}$</td>
<td>$g \geq t = t_2$</td>
</tr>
<tr>
<td>$f_{ep} = \text{chord stress as a result of additional axial force or bending moment}$</td>
<td>$-0.55 \leq \frac{g}{d_0} \leq 0.25$ $30^\circ \leq \theta_i \leq 90^\circ$</td>
</tr>
<tr>
<td>$N_{eq} = \sum N_i \cos \theta_i + N_{eq}$</td>
<td>welds are to be dimensioned on the yield strength of the bracing</td>
</tr>
</tbody>
</table>

Efficiency K- and N-gap with $g' = 2$

![Graph](image)

Fig. 11 - Design chart for K- and N-joints with gap of circular hollow sections
Efficiency K- and N-gap with $g' = 6$

\[
\frac{N_t}{A_1 \cdot f_{y1}} = C_K \cdot f_{y0} \cdot \frac{t_0}{t_1} \cdot \frac{1}{\sin \theta_1} \cdot f(n')
\]

Efficiency K- and N-gap with $g' = 10$

\[
\frac{N_t}{A_1 \cdot f_{y1}} = C_K \cdot f_{y0} \cdot \frac{t_0}{t_1} \cdot \frac{1}{\sin \theta_1} \cdot f(n')
\]

Fig. 11 - contd. – Design chart for K- and N-joints with gap of circular hollow sections
**calculation example**

- **chord (0):** $219.1 \times 10.0$ (compr.) $d_1/t_0 = 21.9$
- **bracing (1):** $139.7 \times 6.3$ (compr.) $d_1/t_0 = 22.2$
- **bracing (2):** $114.5 \times 5.0$ (tension) $d_2/t_2 = 22.9$

\[
\begin{align*}
\beta &= \frac{d_1}{d_0} = \frac{219.1}{139.7} \\
g^* &= 0.64 \\
f(n^*) &= 0.88
\end{align*}
\]

- $f_{sp} = f_{sp}'; f_{sp} = -0.3 f_{sp}; \quad \theta_1 = \theta_2 = 40^\circ; \quad g = 85$ mm

\[
\begin{align*}
\frac{N_1^*}{A_1 \cdot f_{sp}} &= 0.33 \times 10 \times \frac{1}{6.3} \times 0.643 = 0.88 = 0.72 \\
\frac{N_2^*}{A_2 \cdot f_{sp}} &= \frac{1}{A_2 \cdot f_{sp}} \sin \theta_1 = 1.10 > 1.0
\end{align*}
\]

---

**Design chart 4: Tubular joints**

**K- and N-overlap joints of circular hollow sections**

<table>
<thead>
<tr>
<th>symbols</th>
<th>ranges of validity</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\beta = \frac{d_1 + d_2}{2d_0}$</td>
<td>$\frac{d_0}{d_1} \leq 1.0$</td>
</tr>
<tr>
<td>$\gamma = \frac{d_0}{2l_0}$</td>
<td>$0.2 \leq \frac{d_0}{d_1}$</td>
</tr>
<tr>
<td>$n' = \frac{f_{sp}}{f_{sp}'}$</td>
<td>$\frac{d_0}{d_1} \leq 50$ (i = 0, 1, 2)</td>
</tr>
<tr>
<td>$f_{sp} = \text{chord stress as a result of additional axial force or bending moment}$</td>
<td></td>
</tr>
<tr>
<td>$f_{sp} = \text{yield strength of the chord}$</td>
<td></td>
</tr>
<tr>
<td>$f_{sp}' = \text{yield strength of the brace}$</td>
<td></td>
</tr>
<tr>
<td>$\theta_1 = \theta_2 = 40^\circ$</td>
<td></td>
</tr>
<tr>
<td>$\theta_1 = 30^\circ$</td>
<td></td>
</tr>
<tr>
<td>$\theta_2 = 90^\circ$</td>
<td></td>
</tr>
<tr>
<td>$\text{welds are to be dimensioned on the yield strength of the brace}$</td>
<td></td>
</tr>
<tr>
<td>$Ov &gt; 25%$</td>
<td></td>
</tr>
<tr>
<td>$0.55 &lt; \frac{g}{d_0} \leq 0.25$</td>
<td></td>
</tr>
</tbody>
</table>

**calculation example**

- $219.1 \times 10.0$ (compr.) $d_1/t_0 = 21.9$
- $139.7 \times 6.3$ (compr.) $d_1/t_0 = 22.2$
- $114.5 \times 5.0$ (tension) $d_2/t_2 = 22.9$

\[
\begin{align*}
\beta &= \frac{d_1}{d_0} = \frac{139.7}{219.1} \\
g^* &= 0.64 \\
f(n^*) &= 0.88
\end{align*}
\]

\[
\begin{align*}
\frac{N_1^*}{A_1 \cdot f_{sp}} &= 0.44 \times 10 \times \frac{1}{6.3} \times 0.643 = 0.95 \\
\frac{N_2^*}{A_2 \cdot f_{sp}} &= \frac{1}{A_2 \cdot f_{sp}} \sin \theta_1 = 1.46 > 1.0
\end{align*}
\]

---

**Fig. 11 – contd. – Design chart for K- and N-joints with gap of circular hollow sections**

**Fig. 12 – Design chart for K- and N-overlap joints of circular hollow sections (see next page for $C_{\text{pr}}$ and $f(n^*)$ diagrams)**
Efficiency K- and N-overlap joints

\[ \frac{N^1}{A_1 \cdot f_y^1} = \frac{f_{y0} \cdot t_0 \cdot 1}{\sin \theta_1} \cdot f(n') \]

Function \( f(n') \)

Fig. 12 – contd. – Design chart for K- and N-overlap joints of circular hollow sections
As a formula these efficiency limits can be expressed by:

\[ \text{eff} \leq 0.22 \left( \frac{E}{f_p} \frac{t}{d_i} \right)^{0.5} \leq 1.0 \]

(4.28)

Considering member buckling the above mentioned limitations will not frequently be critical.
4.3 Joints in multi-planar structures

Multi-planar joints are frequently used in tubular structures e.g. in towers, offshore jacket structures, triangular or quadrangular girders, etc. Design rules covering the multi-planar effects are given only in [17]. However, the multi-planar effects in [17] have been on elastic considerations and have not yet been checked sufficiently against the actual plastic behaviour of joints. For design, however, some guidelines can be given.

One can imagine that the multi-planar effects are most substantial for double X-joints as shown in Fig. 15. Finite element calculations [18] have shown that multi-planar loading has a substantial influence on the strength and stiffness as compared to a uni-planar X-joint. In the case where the loads acting in one plane have the same magnitude as those in the other plane, but with an opposite sense (e.g. compression vs. tension), the joint strength may drop by about 1/3 compared to the uni-planar joint (see Fig. 17). On the other hand, for loadings with the same sense the joint strength increases considerably. However, this increase in strength may be accompanied by a reduction in deformation and rotation capacity. A conservative assumption for the time being will be to adopt the same percentage increase in strength for loads in the same sense as the percentage reduction for opposite loads.
For K-joints in triangular girders as shown in Fig. 16, various tests have been carried out by Makin [20]. Although an interaction equation is established in [20], this function can easily be replaced by a constant of 0.9, to be applied to the strength of uni-planar joints.

Fig. 16 - Multi-planar K-joints
For T-joints, tests have been carried out only on double T-joints (V-joints) with a 90° included angle between bracings and both bracings loaded in compression (Fig. 17). Compared to the strength of uni-planar joints the multi-planar joint strength did not vary substantially, although the stiffness increased considerably [19].

Based on the available evidence it is recommended to design multi-planar joints using the formulae for uni-planar joints with the correction factors as given in Fig. 17.

<table>
<thead>
<tr>
<th>Type of joint</th>
<th>Correction factor to uni-planar joint (limits according to Fig. 8)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TT</td>
<td>1.0</td>
</tr>
<tr>
<td>XX</td>
<td>(1 + 0.33 \frac{N_2}{N_1}) Note: take account of the sign of (N_2) and (N_1) ((N_1 &gt; N_2))</td>
</tr>
<tr>
<td>KK</td>
<td>0.9</td>
</tr>
</tbody>
</table>

Fig. 17 – Correction factors for multi-planar joints

### 4.4 Joints under moment loading

One should distinguish between primary bending moments due to nodding eccentricities (Fig. 1) needed for the equilibrium with the external loading and secondary bending moments due to end fixities of the joint members as a result of induced deformations in the structural system. The secondary moments are in principle not needed for the equilibrium with the external loading e.g., the secondary moments in members of lattice girders. As already mentioned in chapter 4.2, these secondary moments do not influence the load bearing capacity of lattice girders if the joints have sufficient deformation capacity, i.e., within the parameter limits of the formulae given in Fig. 8.

The moments due to nodding eccentricity in lattice girders may be assumed to be taken by the chord members. Joints predominantly loaded by in-plane bending moments are generally of the T-type and called Vierendeel joints (Fig. 18). These joints also exist in framed structures.

Fig. 18 – Uni-planar Vierendeel joints
Out-of-plane bending moments are not very common in uni-planar structures. This type of loading generally appears more frequently in multi-planar structures.

The joint design strength for joints loaded by bending can also be used for other joint configurations such as K-, N- and KT-joints\(^5\).

<table>
<thead>
<tr>
<th>Type of joint</th>
<th>Design strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>T.Y.X</td>
<td>chord plastification</td>
</tr>
<tr>
<td>( M_{c} = 4.85 \left( f_{y} \cdot \frac{r}{d_{1}} \cdot \gamma \right) \cdot (d_{1} \cdot \sin \beta) ) ( f(n) )</td>
<td></td>
</tr>
<tr>
<td>T.Y.X.K.N.</td>
<td>chord plastification</td>
</tr>
<tr>
<td>( M_{c} = f_{y} \cdot \frac{r_{o}}{1 - 0.81 \beta} \cdot \frac{2.7}{d_{1}} \cdot f(n) )</td>
<td></td>
</tr>
<tr>
<td><strong>General</strong></td>
<td></td>
</tr>
<tr>
<td><em>Punching shear check</em></td>
<td></td>
</tr>
<tr>
<td>for ( d_{1} &lt; d_{2} - 2 \cdot t_{b} )</td>
<td></td>
</tr>
<tr>
<td>( M_{r} = \frac{f_{y}}{\sqrt{3}} \cdot \frac{1 + 3 \sin \delta_{1}}{4 \sin ^{2} \delta_{1}} ) ( d_{1} )</td>
<td></td>
</tr>
<tr>
<td>( M_{c} = \frac{f_{y}}{\sqrt{3}} \cdot \frac{3 + 3 \sin \delta_{1}}{4 \sin ^{2} \delta_{1}} ) ( d_{1} )</td>
<td></td>
</tr>
<tr>
<td><strong>Same range of validity as for axially loaded joints, see Fig. 8</strong></td>
<td></td>
</tr>
<tr>
<td>( f(n') = 1 + 0.3 n' - 0.3 n'^{2} ) for ( n' \leq 1.0 )</td>
<td></td>
</tr>
<tr>
<td>( f(n') = 1 ) for ( n' &gt; 1.0 )</td>
<td></td>
</tr>
</tbody>
</table>

Fig. 19 - Design recommendations for joints loaded by primary bending moments

For punching shear the plastic shear moment capacity is given, however, the angle function is based on an elastic approach.

In a similar manner to axially loaded joints, these formulae are presented as efficiency design charts [Figs. 20 and 21]. The joint efficiency \( C_{\text{rad}} \) or \( C_{\text{con}} \) gives the joint moment design strength divided by the plastic moment capacity \( M_{pc} \), \( f_{y} \), of the bracing. The horizontal cut off line gives the limitation based on punching shear (plastic punching shear moment capacity).

These diagrams show that in most cases the in-plane bending moment resistance is considerably better than that for out-of-plane bending.

It should be noted that the joint rotational stiffness \( C \) (moment per radian) may considerably influence the moment distribution in statically indeterminate structural systems, e.g. portal frames and Vierendeel trusses. If rigid connections are required it is recommended to choose a \( \beta \) ratio near 1.0 or low \( d_{2}/t_{b} \) ratios in combination with high \( t_{f}/t_{b} \) ratios.

Figs. 22 and 23 give a graphical presentation of the rotational joint stiffness of T-joints\(^{21}\) for in-plane and out-of-plane bending moments.

33
Fig. 20 – Design diagram for joints loaded by in-plane bending moments

Fig. 21 – Design diagram for joints loaded by out-of-plane bending moments

Fig. 22 – Joint stiffness for in-plane bending of T-joints

Fig. 23 – Joint stiffness for out-of-plane bending of T-joints
4.5 Interaction between axial loading and bending moments

Especially in three dimensional structures the joints may be loaded by combinations of axial loading and bending moments. Several investigations have been carried out to study this problem and as a result many interaction formulae exist. All investigations have shown that in-plane bending is less severe than out-of-plane bending and a reasonable simplified lower bound interaction function is given by \[ 16 \]

\[
\frac{N_i}{N_i^*} + \left( \frac{M_{ip}}{M_{ip}^*} \right)^2 + \frac{M_{op}}{M_{op}^*} \leq 1.0
\] (4.5.1)

in which:

\(N_i, M_{ip}\) and \(M_{op}\) are the loads acting, and \(N_i^*, M_{ip}^*\) and \(M_{op}^*\) are the design strengths. It should be noted that the joint stiffnesses given in Figs. 22 and 23 can be affected considerably by the presence of axial loading [22]; however not sufficient test evidence is available for a more precise recommendation.

Triangular girder (85 m length) for the support of a roof.
4.6 Special types of uni-planar joints

4.6.1 Other configurations

In tubular structures various other joint configurations exist which have not been dealt with in the previous chapters. However, the strength of several types of joints can be directly related to the basic types dealt with in chapter 4.2. Fig. 24 shows some special types of joints with tubular bracings directly welded to the tubular chord.

<table>
<thead>
<tr>
<th>Type of joint</th>
<th>Relationship with the formulae in Fig. 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>( N_1 \leq N_1^* ), ( N_1^* ) from X-joint</td>
</tr>
<tr>
<td>b</td>
<td>( N_1 \cdot \sin \theta_1 + N_2 \cdot \sin \theta_2 \leq N_1^* \cdot \sin \theta_1 ) ( (N_1^* ) from K-joint)</td>
</tr>
<tr>
<td></td>
<td>( N_2 \cdot \sin \theta_2 \leq N_1^* \cdot \sin \theta_1 ) ( (N_1^* ) from K-joint)</td>
</tr>
<tr>
<td></td>
<td>replace ( d_0 ) by ( \frac{d_0 + d_1 + d_2}{3} ) in K-joint strength formula</td>
</tr>
<tr>
<td>c</td>
<td>( N_1 \cdot \sin \theta_1 + N_2 \cdot \sin \theta_2 \leq N_1^* \cdot \sin \theta_1 ) ( (N_1^* ) from X-joint)</td>
</tr>
<tr>
<td></td>
<td>where ( N_1^* \cdot \sin \theta_1 ) is the larger of ( N_1^* \cdot \sin \theta_1 ) and ( N_2 \cdot \sin \theta_2 )</td>
</tr>
<tr>
<td>d</td>
<td>( N_1 \leq N_1^* ) ( (K\text{-joint}) )</td>
</tr>
<tr>
<td></td>
<td>( N_2 \leq N_2^* ) ( (K\text{-joint}) )</td>
</tr>
<tr>
<td></td>
<td>check cross section 1–1 for plastic shear capacity ( (\text{gap joints only}) )</td>
</tr>
</tbody>
</table>

Fig. 24 – Other configurations of uni-planar tubular joints
Arch-formed trusses for a sport hall

Fish-shaped trusses for an ice-skating hall
4.6.2 Plate type joints

Various joint configurations are possible for joints with gusset plates. The design strength of these joints is mainly based on tests carried out in Japan. In the original research reports a distinction is made between TP-joints (plate to CHS T-joints) and XP-joints (plate to CHS X-joints), with the former having a plate on one side of the tube and the latter having plates on both sides of the tube.

The design strength formulae in Fig. 25 have been simplified in a conservative way so that they cover both types for various load conditions. However for TP-joints with \( f(\beta) = 4 + 20 \beta^2 \) fits the test results better than \( f(\beta) = \frac{5}{1 - 0.81 \beta} \).

Furthermore, all joints have to be checked for punching shear:
- for TP-5/XP-5: \( (f_a + f_b) \cdot t_x \leq 0.58 f_{yo} \cdot t_o \);
- for other joints: \( (f_a + f_b) \cdot t_x \leq 1.16 f_{yo} \cdot t_o \),

where \( f_a \) and \( f_b \) are the axial and bending stress in the connected plate, I- or RHS section.

The design recommendations in the first row cover XP-1/TP-1 and XP-3/TP-3 joints.

The XP-1/TP-1 joints only have a plate perpendicular to the main chord axis whereas the XP-3/TP-3 joints also have a plate parallel to the chord axis.

Since the stiffness of a longitudinal plate parallel to the chord axis is considerably smaller than that perpendicular to the chord axis, the strengths of both joint types are about similar.
<table>
<thead>
<tr>
<th>Type of Joint</th>
<th>Axial Loading $N^* = f(\beta) \cdot f(q) \cdot f(n) \cdot f_{yFC} \cdot t^2$ (eq. 4.6.2.1)</th>
<th>Bending in Plane</th>
<th>Bending Out of Plane</th>
</tr>
</thead>
<tbody>
<tr>
<td>XP-1/TP-1</td>
<td>$f(\beta) f(q) f(n) f_{yFC} t^2$</td>
<td>$M_{yP} = 0.5 b_1 \cdot N_{XP-1}$ (eq. 4.6.2.5)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\frac{5.0}{1 - 0.81 \beta}$</td>
<td>$f(n)$</td>
<td></td>
</tr>
<tr>
<td>XP-2/TP-2</td>
<td>$f(\beta) f(q) f(n) f_{yFC} t^2$</td>
<td>$M_{yP} = h_1 \cdot N_{XP-2}$ (eq. 4.6.2.2)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\frac{5.0}{1 + 0.25 s}$</td>
<td>$f(n)$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\eta = 4$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>XP-3/TP-3</td>
<td>$f(\beta) f(q) f(n) f_{yFC} t^2$</td>
<td>$M_{yP} = 0.5 b_1 \cdot N_{XP-3}$ (eq. 4.6.2.3)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\frac{5.0}{1 - 0.81 \beta}$</td>
<td>$f(n)$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\eta = 4$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>XP-4/TP-4</td>
<td>$f(\beta) f(q) f(n) f_{yFC} t^2$</td>
<td>$M_{yP} = h_1 \cdot N_{XP-4}$ (eq. 4.6.2.6)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\frac{5.0}{1 - 0.81 \beta}$</td>
<td>$f(n)$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\eta = 4$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>XP-5/TP-5</td>
<td>$f(\beta) f(q) f(n) f_{yFC} t^2$</td>
<td>$M_{yP} = 0.5 b_1 \cdot N_{XP-5}$ (eq. 4.6.2.7)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\frac{5.0}{1 - 0.81 \beta}$</td>
<td>$f(n)$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\eta = 2$</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

General remarks: for symbols, parameters and limitations: see axially loaded joints. $\beta = b_1/d_y$, $\eta = h_1/d_y$.

Fig. 25 – Gusset plated connections
4.6.3 Flattened and cropped end bracing joints

Joints with flattened end bracings are sometimes used, especially for small sized and temporary tubular structures. As shown in Fig. 6, various types of flattening can be provided. In the case of full or partial flattening, the maximum taper from the tube to the flat should remain within 25% (or 1:4), as shown in Fig. 6B and C. For $d_i/t$ ratios exceeding 25 the flattening will reduce the compressive strength.

For welded connections the length of the flat part should be minimized for compression members to avoid local buckling. Recommended design strength formulae for cropped-web N-joints with overlap [23] are given in Fig. 26. Compared to the ultimate joint strength given in [23] for the vertical bracing loaded in compression a factor of 1.25 has been adopted to account for the transformation from ultimate strength to design strength. Since the behaviour of this type of joint may be influenced by size effects, care should be taken in using these empirical formulae, and that is why the validity is restricted to the dimensional range tested:

<table>
<thead>
<tr>
<th>dimensions tested (mm)</th>
<th>parameters tested</th>
</tr>
</thead>
<tbody>
<tr>
<td>$114 \leq d_i \leq 169$</td>
<td>$14 \leq \frac{d_i}{t} \leq 50$</td>
</tr>
<tr>
<td>$42 \leq d_i \leq 90$</td>
<td>$0.35 \leq \frac{d_i}{t} \leq 0.8$</td>
</tr>
<tr>
<td>$3 \leq t_e \leq 8$</td>
<td>$d_i \quad d_i = 1.0$</td>
</tr>
<tr>
<td>$3 \leq t_e \leq 4.6$</td>
<td>$t_e \quad t_e = 1.0$</td>
</tr>
<tr>
<td>$f_p \leq 400 \text{N/mm}^2$</td>
<td>$\theta_1 = 90^\circ; \theta_2 = 45^\circ$</td>
</tr>
</tbody>
</table>

For chords prestressed in compression up to 80% of the yield load the joint strength should be multiplied by $f(n) = 1 + 0.2n (0 \leq n \leq 0.8)$. Higher chord prestress loads should not be accepted since sufficient test evidence is not available. For trusses with flattened and cropped end bracings an effective buckling length $L_e$ of 1.0 times the system length is recommended.

Partial-flattened end bracing joints, as shown in Fig. 27, have recently been investigated in CIDECT programme 5AP [26]. These joints can be designed with the same joint strength formulae as given in Fig. 8 provided that the following modifications are adopted:

- **T- and X-joints in compression**: replace in the formula for $N_x^c$
  
  $d_i$ by $d_{min}$

- **K-joints with gap**: replace in the formula for $N_x^c$
  
  $d_i$ by $\frac{d_i + d_{min}}{2}$
Fig. 26 - Design diagram for cropped end bracing connections

Fig. 27 - K-joint with partial-flattened end bracings
5 Bolted connections

The calculation methods used for many types of bolted connections between or to hollow sections are not basically different from those used for any other type of connection in conventional steel construction. (Some calculation examples will be given in chapter 6.5.) Bolted connections are especially desirable for site joints between prefabricated sub-assemblies. Various examples of bolted connections are given in Figs. 28 to 30 and 33.

Fig. 28 - Bolted truss support connections

Fig. 29 - Bolted purlin connections

Fig. 30 - Bolted end connections

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For flange joint connections various investigations have been carried out. However, for simple designs the recommendations which are included in the 1990 edition of the Japanese Recommendations for the Design and Fabrication of Tubular Structures in Steel [28] are most simple and are given in Fig. 31. Implicit in these connection details is an allowance for prying forces amounting to 1/3 of the total bolt force at the ultimate limit state and the assumption that the tube yield strength must be developed. The modes of failure assumed in determining these details are those due to plastification of flange plates and not due to tensile failure of high strength bolts. The standard details shown in Fig. 31 are for STK41 tubes. (specified minimum \( f_p = 235 \) N/mm² and minimum ultimate tensile strength = 402 N/mm²), SS41 plates (specified minimum yield strength = 245 N/mm²) and F 10T bolts (about equal to 10.9 bolts with a specified minimum ultimate tensile strength of 981 N/mm²).

<table>
<thead>
<tr>
<th>max. tube dimensions ( d_1 \times t_1 ) (mm)</th>
<th>thickness of flange plate ( t_2 ) (mm)</th>
<th>nominal diameter of bolt (mm)</th>
<th>minimum no. of bolts</th>
<th>edge distance ( e_1 = e_2 ) (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>60.5 × 4.0 through 89.1 × 4.0</td>
<td>12</td>
<td>16</td>
<td>4</td>
<td>25</td>
</tr>
<tr>
<td>101.6 × 4.0 through 114.3 × 3.6</td>
<td>12</td>
<td>16</td>
<td>5</td>
<td>25</td>
</tr>
<tr>
<td>114.3 × 5.6 through 139.6 × 4.5</td>
<td>16</td>
<td>20</td>
<td>5</td>
<td>30</td>
</tr>
<tr>
<td>165.2 × 5.0</td>
<td>20</td>
<td>22</td>
<td>5</td>
<td>35</td>
</tr>
<tr>
<td>190.7 × 5.0</td>
<td>20</td>
<td>22</td>
<td>6</td>
<td>35</td>
</tr>
<tr>
<td>216.3 × 6.0</td>
<td>20</td>
<td>22</td>
<td>8</td>
<td>35</td>
</tr>
<tr>
<td>216.3 × 8.0</td>
<td>22</td>
<td>24</td>
<td>9</td>
<td>40</td>
</tr>
<tr>
<td>267.4 × 9.0</td>
<td>24</td>
<td>24</td>
<td>13</td>
<td>40</td>
</tr>
<tr>
<td>318.5 × 7.0</td>
<td>24</td>
<td>24</td>
<td>12</td>
<td>40</td>
</tr>
<tr>
<td>355.6 × 12.0</td>
<td>24</td>
<td>24</td>
<td>23</td>
<td>40</td>
</tr>
<tr>
<td>406.4 × 9.0</td>
<td>24</td>
<td>24</td>
<td>20</td>
<td>40</td>
</tr>
</tbody>
</table>

Fig. 31 - Standard details for flange joint connections (full strength connections)

According to [28] the flange plate thickness \( t_2 \) can be determined from:

\[
t_2 = \sqrt{\frac{2N_e}{f_{y_2} \cdot \gamma_m \cdot t_1}}
\]  

(5.1)

where

- \( N_e \) = tensile member force
- \( f_{y_2} \) = yield strength of plate
- \( \gamma_m \) = 1.1 (partial safety factor)
- \( t_1 \) = dimensionless to be obtained from Fig. 32
- \( t_2 \) = thickness of plate

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Fig. 32 – Parameter $f_3$ for use in Eq. 5.1 for the design of a CHS flange plate connections
The dimension \( e_1 \) (see Fig. 31) should be kept as low as possible to minimize prying action (around 1.5 \( d \) to 2.0 \( d \); \( d \) = bolt diameter), but the clearance between the nut and the weld should be at least 5 mm.

The number of bolts \( n \) can be determined from:

\[
n \geq \frac{N}{0.67T_u} \left[ 1 - \frac{1}{r_2} + \frac{1}{r_1} \ln \left( \frac{r_1}{r_2} \right) \right]^{1/\gamma}
\]

(5.2)

where

\[ r_1 = \left( \frac{d}{2} + 2e_1 \right) \]

\[ r_2 = \left( \frac{d}{2} + e_1 \right) \]

\( T_u \) = ultimate tensile resistance of a bolt

Other factors, see eq. 5.1. 

Fig. 33 - Some examples of bolted connections
6 Worked out designing examples

6.1 a) Uni-planar truss

- **Truss lay-out:**
  
  The following dimensions are assumed:
  
  Span = 36 m, Trusses L 12 m centres
  
  Purlins,
  
  L 6 m centres
  
  Truss depth ~ \( \frac{\text{span}}{15} \) = 2.40 m (considering overall costs, e.g. costs of all cladding of the building, deflections, etc. \( l/15 \) is generally an economical height)

![Truss Lay-out Diagram](image)

\[ \tan \theta = \frac{2.4}{3} = 0.8 \Rightarrow \theta = 38.7^\circ \]

Fig. 34 - Truss lay-out

A warren type truss with K-joints is chosen to limit the number of joints.

The factored design load \( P \) from the purlins including the weight of the truss have been calculated as \( P = 106 \text{ kN} \).

- **Member loads (kN)**

A pin-jointed analysis of the truss gives the following member forces:

![Truss Member Axial Loads Diagram](image)

Fig. 35 - Truss member axial loads

- **Design of members**

In this example the chords will be made from steel with a yield stress of 355 N/mm² and bracing from steel with a yield stress of 275 N/mm².

For member selection use either member resistance tables for the applicable effective length or the applicable buckling curve. Check the availability of the member sizes selected. Since the joints at the truss ends are generally decisive, the chords should not be too thin walled. As a consequence a continuous chord with the same wall thickness over the whole truss length is often the best choice.

**top chord**

use a continuous chord with an effective in-plane and out-of-plane length of:

\( t_c = 0.9 \times 6000 = 5400 \text{ mm} \) [7, 16], see chapter 2.2

\( N_c = 1148 \text{ kN} \)
<table>
<thead>
<tr>
<th>$f_y$</th>
<th>$N_0$</th>
<th>$l_0$</th>
<th>possible sections</th>
<th>$A_0$</th>
<th>$d_0/l_0$</th>
<th>$x^*$</th>
<th>$x \cdot f_y \cdot A_0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>355</td>
<td>1148</td>
<td>5400</td>
<td>193.7 - 10.0</td>
<td>5771</td>
<td>19.4</td>
<td>1.09</td>
<td>0.61</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>219.1 - 7.1</td>
<td>4728</td>
<td>30.9</td>
<td>0.94</td>
<td>0.71</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>219.1 = 8.0</td>
<td>5305</td>
<td>27.4</td>
<td>0.95</td>
<td>0.71</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>244.5 = 5.6</td>
<td>4202</td>
<td>43.7</td>
<td>0.84</td>
<td>0.78</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>244.5 = 6.3</td>
<td>4714</td>
<td>36.8</td>
<td>0.84</td>
<td>0.78</td>
</tr>
</tbody>
</table>

* Eurocode 3 buckling curve "a" *

From a material point of view the sections 244.5 x 5.6 and 219.1 = 7.1 are most efficient; however, these two dimensions are, for the supplier considered in this example, not available from stock (only to be delivered from factory). These dimensions can only be used if a large quantity is required, which is assumed in this example.

**Bottom chord**

<table>
<thead>
<tr>
<th>$f_y$</th>
<th>$N_0$</th>
<th>possible sections</th>
<th>$A_0$</th>
<th>$d_0/l_0$</th>
<th>$f_y \cdot A_0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>355</td>
<td>1215</td>
<td>168.3 - 7.1</td>
<td>3595</td>
<td>23.7</td>
<td>1276</td>
</tr>
<tr>
<td></td>
<td></td>
<td>177.8 - 7.1</td>
<td>3807</td>
<td>25.0</td>
<td>1351</td>
</tr>
<tr>
<td></td>
<td></td>
<td>193.7 - 6.3</td>
<td>3709</td>
<td>30.7</td>
<td>1317</td>
</tr>
</tbody>
</table>

**Diagonals**

Try to select members which satisfy $\frac{f_y}{f_y - f_t} \geq 2.0$; i.e.

$$\frac{355 \cdot 7.1}{275 \cdot t} \geq 2.0 \text{ or } t \leq 4.5 \text{ mm, see eq. 4.2.7.}$$

Use for the bracings loaded in compression an initial effective length of $0.75 \cdot l = 0.75 \sqrt{2.4^2 + 3.0^2} = 2.98 \text{ m [7, 16], see chapter 2.2.}$

**Compression diagonals**

<table>
<thead>
<tr>
<th>$f_y$</th>
<th>$N_t$</th>
<th>$l_0$</th>
<th>possible sections</th>
<th>$A_t$</th>
<th>$x^*$</th>
<th>$x \cdot f_y \cdot A_t$</th>
</tr>
</thead>
<tbody>
<tr>
<td>275</td>
<td>432</td>
<td>2.881</td>
<td>168.3 - 3.6</td>
<td>1862</td>
<td>0.57</td>
<td>0.90</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>139.7 - 4.5</td>
<td>1911</td>
<td>0.69</td>
<td>0.85</td>
</tr>
<tr>
<td>275</td>
<td>259</td>
<td>2.881</td>
<td>114.6 - 3.6</td>
<td>1252</td>
<td>0.85</td>
<td>0.77</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>101.6 - 4.0</td>
<td>1226</td>
<td>0.96</td>
<td>0.70</td>
</tr>
<tr>
<td>275</td>
<td>86</td>
<td>2.881</td>
<td>88.9 - 2.0</td>
<td>546</td>
<td>1.08</td>
<td>0.61</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>76.1 - 2.6</td>
<td>600</td>
<td>1.28</td>
<td>0.49</td>
</tr>
</tbody>
</table>

* Eurocode 3 buckling curve "a" *

** the wall thickness is rather small for welding **
Tension diagonals

<table>
<thead>
<tr>
<th>$t_y$ (N/mm²)</th>
<th>F (kN)</th>
<th>possible sections (mm)</th>
<th>$A_y$ (mm²)</th>
<th>$f_{y2} \cdot A_2$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>275</td>
<td>432</td>
<td>133.3 – 4.0</td>
<td>1621</td>
<td>445</td>
</tr>
<tr>
<td>275</td>
<td>259</td>
<td>86.9 – 3.6</td>
<td>964</td>
<td>265</td>
</tr>
<tr>
<td>275</td>
<td>86</td>
<td>48.3 – 2.3</td>
<td>332</td>
<td>91</td>
</tr>
</tbody>
</table>

Member selection

The number of sectional dimensions depends on the total tonnage to be ordered. In this example for the bracings only two different dimensions will be selected.

Comparison of the members suitable for the tension members and those suitable for the compression members shows that the following sections are most convenient:

- bracings: 139.7 – 4.5
  - 88.9 – 3.6
- top chord: 219.1 – 7.1
- bottom chord: 193.7 – 6.3 (These chord sizes allow gap joints; no eccentricity is required).

It is recognized that the $d_i/t_o$ ratios of the chords selected are high. This may give joint strength problems in joints 2 and 5.

Fig. 36 - Member dimensions

Commentary and revision

**Joint 1**

In joint 1 between plate and bracing a gap $g = 2\ t_o$ is chosen. This joint is checked as a K(N) joint.

Attention should be paid to the top chord shear capacity, i.e. cross section A should be able to resist the shear of $2.5\ P = 2.5 \times 108 = 270 \text{ kN}$.

Since joint 1 is rather heavily loaded it is recommended to use conservatively the elastic shear capacity of the top chord, i.e.:

\[
0.5 A_t \cdot \frac{f_{y2}}{\sqrt{3}} = 0.5 \cdot 4728 \cdot \frac{0.355}{\sqrt{3}} = 485 \text{ kN} > 270 \text{ kN}
\]

Fig. 37
## Check joint strength

<table>
<thead>
<tr>
<th>joint</th>
<th>chord (mm)</th>
<th>bracings (mm)</th>
<th>dₗ/dₗ₀</th>
<th>dₗ₀/tₗ₀</th>
<th>Γ₀</th>
<th>n' = 10ₗ₀</th>
<th>A_i - 1 / N_i</th>
<th>cₗ₀</th>
<th>fₗ₀ - fₗ₀</th>
<th>f(l'ₗ₀) sin l</th>
<th>N°</th>
<th>N° &gt; N_i</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>219.1 – 7.1</td>
<td>plate 139.7 – 4.5</td>
<td>0.64</td>
<td>30.9</td>
<td>2.0</td>
<td>not appl.</td>
<td>0.32</td>
<td>2.04</td>
<td>1.60</td>
<td>&gt; 1.00</td>
<td>yes</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>219.1 – 7.1</td>
<td>139.7 – 4.5</td>
<td>0.64</td>
<td>30.9</td>
<td>12.8*</td>
<td>-0.20</td>
<td>0.62</td>
<td>0.32</td>
<td>2.04</td>
<td>1.49</td>
<td>0.70</td>
<td>no</td>
</tr>
<tr>
<td></td>
<td></td>
<td>88.9 – 3.6</td>
<td>0.64</td>
<td>30.9</td>
<td>12.8*</td>
<td>-0.20</td>
<td>0.98</td>
<td>0.23</td>
<td>2.55</td>
<td>&gt; 1.00</td>
<td>yes</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>219.1 – 7.1</td>
<td>139.7 – 4.5</td>
<td>0.64</td>
<td>30.9</td>
<td>12.8*</td>
<td>-0.52</td>
<td>0.49</td>
<td>0.23</td>
<td>2.04</td>
<td>1.22</td>
<td>0.58</td>
<td>yes</td>
</tr>
<tr>
<td></td>
<td></td>
<td>88.9 – 3.6</td>
<td>0.64</td>
<td>30.9</td>
<td>12.8*</td>
<td>-0.52</td>
<td>0.32</td>
<td>0.23</td>
<td>2.55</td>
<td>&gt; 1.00</td>
<td>yes</td>
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<td>4</td>
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<td>88.9 – 3.6</td>
<td>0.41</td>
<td>30.9</td>
<td>18.5</td>
<td>-0.68</td>
<td>0.32</td>
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<td>-0.68</td>
<td>0.32</td>
<td>0.26</td>
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<tr>
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<td>139.7 – 4.5</td>
<td>0.72</td>
<td>30.7</td>
<td>2.9</td>
<td>+</td>
<td>0.82</td>
<td>0.29</td>
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<td>1.60</td>
<td>0.85</td>
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<td></td>
<td>139.7 – 4.5</td>
<td>0.72</td>
<td>30.7</td>
<td>2.9</td>
<td>+</td>
<td>0.82</td>
<td>0.29</td>
<td>1.81</td>
<td>1.60</td>
<td>0.85</td>
<td>yes</td>
</tr>
<tr>
<td>6</td>
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<td>88.9 – 3.6</td>
<td>0.72</td>
<td>30.7</td>
<td>9.4</td>
<td>+</td>
<td>0.98</td>
<td>0.23</td>
<td>2.26</td>
<td>1.60</td>
<td>0.67</td>
<td>yes</td>
</tr>
<tr>
<td></td>
<td></td>
<td>139.7 – 4.5</td>
<td>0.72</td>
<td>30.7</td>
<td>9.4</td>
<td>+</td>
<td>0.49</td>
<td>0.23</td>
<td>1.81</td>
<td>1.60</td>
<td>0.67</td>
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</tr>
<tr>
<td>7</td>
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<td>88.9 – 3.6</td>
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<td>30.7</td>
<td>15.8</td>
<td>+</td>
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<td>2.26</td>
<td>1.60</td>
<td>0.91</td>
<td>yes</td>
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<td></td>
<td></td>
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<td>30.7</td>
<td>15.8</td>
<td>+</td>
<td>0.32</td>
<td>0.25</td>
<td>2.26</td>
<td>1.60</td>
<td>0.91</td>
<td>yes</td>
</tr>
</tbody>
</table>

* See Commentary and revision: Joint 4 is here treated as a K-joint, since the plates stiffen the joint, although the loading is similar to a X-joint.
Joint 2

The strength of joint 2 is not sufficient. The easiest way to obtain sufficient joint strength will be to decrease the gap from 12.8 to 3.6, resulting in a joint efficiency of 0.86 > 0.82. However, this means that a (negative) eccentricity of $e = 28$ mm is introduced resulting in a moment due to eccentricities of:
$$M = (878 - 338) \cdot 28 \cdot 10^{-3} = 15.12 \text{ kNm}.$$ 

Since the length and the stiffness EI of the top chord members between joints 1 – 2 and 2 – 3 are the same (see Fig. 36) this moment can be equally distributed over both members, i.e. both members have to be designed additionally for $M_e = 7.56 \text{ kNm}$.

The chord members between joints 1 – 2 and 2 – 3 have now to be checked as a beam-column. From these, the chord member 2 – 3 is most critical. This check depends on the national code to be used.

However, the criterion to be checked has generally a form of:
$$\frac{N_0}{k \cdot A_o \cdot f_{yo}} + k \cdot \frac{M_0}{M_{pe}} \leq 1.0 \quad (6.1.1.)$$

where:

- $M_{pe} =$ plastic resistance ($W_{pe} \cdot f_u$) of the chord (class 1 or 2 sections);
- use for class 3 elastic moment resistance ($W_{e} \cdot f_u$);
- $k =$ factor including second order effects depending on slenderness, section classification and moment diagram (in this case use triangle)

$$\frac{878}{1189} + k \cdot \frac{7.56}{113.3} = 0.74 + 0.067k < 1.0 \text{ (Independent on the code used this will not be critical).}$$

Purlin connections

Depending on the type of purlins various purlin connections are possible.

If corrosion will not occur, a cut out of the channel section welded on top of the chord at the purlin support location and provided with bolt stubs provides an easy support.

Fig. 39 – Purlin load

If single beam purlins are used, a plate a shown in Fig. 29 can be used.
Fig. 25 provides evidence for the design of plate to tube connections.

For the purlin connection at the centre, another alternative is given to allow the site bolted connection of the truss.

Fig. 41 - Purlin support at the site connection of the truss

The top chord can also be provided with end plates only. In that case a T-stub for purlin support has to be fitted in between the end plates.

**Bolted site flange connections**

Bottom tensile chord connection:

According to Fig. 31, 6 bolts $\varnothing$ 22 – 10.9 are required for a full strength connection (i.e. $A_p \cdot f_p$) of $\varnothing$ 190.7 – 5.0 with $f_p = 235$ N/mm$^2$.

In this example the cross section area and the yield stresses of the beam and the plate are different from those in Fig. 31 whereas the bolt strength remains the same. This means that the number of bolts has to be determined taking account of these effects.

For $\varnothing$ 193.7 – 6.3 with $f_p = 355$ N/mm$^2$ and the same bolt strength the number of bolts can be estimated to be:

$$\frac{193.7 \times 6.3}{355} \quad 190.7 \times 5.0 \quad 235 \quad 6 = 12 \text{ bolts } \varnothing 22 – 10.9$$

The end plate thickness can be 20 mm ($f_p = 355$ N/mm$^2$) and the edge distances $e_1 = e_2 = 35$ mm.

An alternative will be to use:

$$\frac{193.7 \times 6.3}{216.3 \times 8.0} \quad 9 = 10 \text{ bolts } \varnothing 24 – 10.9 \text{ with an end plate thickness of 22 mm}$$

($f_p = 355$ N/mm$^2$) and the edge distances $e_1 = e_2 = 40$ mm.

Considering the c.o.c. distance of the bolts, both options are possible.

Top compression chord connection:

For the top chord connection the compression loading is transferred through contact pressure. The number of bolts required depends on the erection loads which can be in tension and the national code requirements with regard to minimum strength related to the member tensile strength.

In order to determine the number of bolts for the compression flange connection, one could conservatively treat it as a full strength tensile connection.

In the case of a full strength connection 12 bolts $\varnothing$ 24 – 10.9 are needed with end plates of 22 mm ($f_p = 355$ N/mm$^2$) and edge distances of 40 mm.

$$\frac{219.1 \times 7.1}{216.3 \times 8.0} \quad 235 \quad 9 = 12$$

The end support connection can also be made as shown in Fig. 28.
b) Arch-formed truss

Fig. 42 – Arched truss

The joints of arch-formed trusses can be designed in a similar way as those of straight chord trusses. If the arch-formed chords are made by bending at the joint locations only, the chord members can also be treated in a similar way as those of straight chord trusses provided that the bending radius remains within the limits to avoid distortion of the cross section. If the arch-formed chords are made by continuous bending, the chord members have a curved shape between the joint locations. In this case the curvature should be taken into account in the member design (moment = axial force × eccentricity) by treating the chord as a beam-column, see eq. 6.1.1. The k factor in eq. 6.1.1 will now be smaller than in the example of chapter 6.1 due to the moment diagram.

Fig. 43 – Eccentricity moments in compression loaded chords of an arched truss

c) Vierendeel truss

Fig. 44 – Vierendeel truss

For Vierendeel trusses with top and bottom chords of the same bending stiffness initially a simplified design calculation can be used, if:
- the loads act at the joints
- the connections are rigid
- the longitudinal displacements of the chords can be disregarded

Under these conditions the moments will be zero at the centres of the chord members between the joints and the load and moment distribution can be determined easily.

Fig. 45 – Simplified modelling
Chord members

The chord member loaded in compression has to be designed as a beam-column for the following conditions (P in kN):

\[ N_c = 1.5 \, P \text{ kN} \quad \text{and} \quad M_c = 1.88 \, P \text{ kNm} (1.25 \, P \text{ kN shear}) \]

\[ N_e = 3.9 \, P \text{ kN} \quad \text{and} \quad M_e = 1.12 \, P \text{ kNm} (0.75 \, P \text{ kN shear}) \]

\[ N_v = 5.1 \, P \text{ kN} \quad \text{and} \quad M_v = 0.38 \, P \text{ kNm} (0.25 \, P \text{ kN shear}) \]

The first case with the highest moment will be decisive.

Bracings

The bracings have to be checked for the moments, axial and shear loads. As shown, the second bracing is decisive with:

\[ M_s = 3.0 \, P \text{ kNm}, N_s = 0.5 \, P \text{ kN} \text{ and } 2.4 \, P \text{ shear load.} \]

Joints

The joints of the second bracing have the largest moment loading. The moments in the chords are in equilibrium with the moment in the bracing. Consequently, they do not have to be considered for the prestress function \( f(n') \). Since the bottom chord is loaded in tension, the connections with the top chord are decisive if the top and bottom chord have the same dimension.

For the chord sections the moments are most critical.
In the bracings the largest moment occurs in the second one. At this point the prestress force \( N_{ps} \) in the chord is relatively low.
\[ M_1 = 3.0 \text{ kNm} \quad N_{ps} = 1.5 \text{ P kN} \]

For Vierendeel trusses the rotational stiffness of the joints is very important. This requires joints with diameter ratios close to one, see chapter 4.4.

**Evaluation**

As shown before, the second bracing is the heaviest loaded member in the Vierendeel truss. The diameter is limited by the chord members and an increase in bracing wall thickness does not increase the joint strength, see Fig. 20. Consequently, if all truss members are made from the same dimension, this results in over design of the other bracings and the chord members if lateral buckling is prevented.

**Joint design**

In this example it is assumed that all members are made from the same circular hollow sections \( \phi \) 193.7 – 6.3 with \( f_y = 355 \text{ N/mm}^2 \).

According to Fig. 19, eq. 4.4.1 the design strength is given by:

\[
M_{u}^d = 4.85 f_{yd} f'_{iy} \frac{d_1^2}{d_1^2} \gamma_y \beta \cdot \frac{d_1}{d_1} \frac{f(n')}{\sin \theta} \]
\[
= 4.85 \cdot 355 \cdot 6.3^2 \left( \frac{193.7}{2 \cdot 6.3} \right)^{0.5} \cdot 1 \cdot 193.7 \cdot f(n')
\]

\[
M_{u}^d = 51.9 \cdot 10^6 \text{ Nmm}
\]
\[
= 51.9 \text{ kNm} \leq W_{400} f_{y} = 59.7 \text{ kNm} \quad (f(n') = 1.0)
\]

\[
N_{u}^d = 0.3 A_{t} f_{y} = 395 \text{ kN} \quad \text{(see Fig. 9)}.
\]

**Fig. 47 – Joint loading**

For \( P = 17 \text{ kN} \):

\[
n' = \frac{N_{ps}}{A_{t} f_{y}} = \frac{1.5 \cdot 17}{3709 \cdot 0.355} = 0.02, \text{ thus: } f(n') = 1.0
\]

\[
N_{1} = 0.5 \cdot 17 = 8.5
\]

\[
N_{1}^d = \frac{3 \cdot 17}{51.9} = 0.98
\]

\[
M_1 = 3 \cdot 17 = 51.9
\]

interaction: eq. 4.5.1

\[
\frac{N_{1}^d}{N_{1}^d} + \left( \frac{M_1}{M_{1}^d} \right)^2 = 0.99 \leq 1.0
\]

54
This calculation confirms that axial forces have a minor effect. The joints for the end bracing can be made in various ways.

![Fig. 48 - Vierendeel end corner connections](image)

The use of type “a” is in accordance with the previous calculations. Since the end cap compensates for the increasing joint rigidity, the chord is not continuous. Type “b” can only be used for low loads since the diagonal reaction forces in the corner cannot be transferred satisfactorily. Type “c” with a fill-in plate provides an adequate load transfer.

**Remarks**

Especially, if thin walled sections are used or if the joint stiffness has to be taken into account ($\beta < 1.0$), a more precise semi-rigid frame analysis by computer has to be carried out to determine the moment distribution and the deflections.

**Plastic design of Vierendeel trusses**

If all of the tubes selected are class 1 (plastic design) sections, and also meet the criteria for rigid joint behaviour (such as $\beta = 1.0$ in the foregoing example), a more favourable distribution of bending moments may be obtained in the truss by the use of “Plastic Moment Distribution” (i.e. a set of moments which is in equilibrium with the applied loads, by the Lower Bound Theorem).

### 6.2 Multiplanar truss (triangular girder)

![Fig. 49 - Triangular truss](image)
Member loads

The member loads will be determined in a similar way as for the uni-planar truss, assuming pin ended members.
The load in the bottom chord follows by dividing the relevant moment by the girder depth.
Since two top chords are used, the load at the top has to be divided by 2.
The loads in the bracings follow from the shear forces \( V_i \) in the girder, i.e.:
\[
N_i = \frac{V_i}{2 \cos \theta / 2 \times \sin \theta}
\]

Fig. 50

The top chords should be connected in the top plane for equilibrium of loading. This can be achieved by a bracing system which connects the loading points. Connection of the loading points only, results in a triangular truss which has no torsional rigidity. Combination with diagonals gives torsional resistance.

Fig. 51

It is also possible to use the purlins or the roof structure as the connecting parts between the loading points.
Now the loads in one plane are known and the design can be treated in a similar way as for uni-planar trusses.

Joints

The joints can also be treated in a similar way as for uni-planar joints, however, taking account of the reduction factors of Fig. 17. This means a reduction factor of 0.9 for the joints with the bottom chord.

From a fabrication point of view, it is better to avoid overlaps of the intersecting bracings from both planes. This may result sometimes in an offset.

Fig. 52 - Gap and offset

The offset (if ≤ 0.25 \( d_s \)) need not be incorporated in the joint design. For chords loaded in tension this offset moment can also be neglected in the member design. For compression-loaded chords the moments due to this offset have to be distributed into the chord members and taken into account in the member design.

Design calculation

Assume \( P = 187 \) kN (limit state)

This means that the loads acting in the side planes of the triangular truss are:

\[
\frac{P}{2 \cos 30^\circ} = 108 \text{ kN}
\]

Fig. 53 - Cross section of the triangular truss
This is equal to the purlin loads used in the design example for the uni-planar truss in chapter 6.1. As a consequence the top chord and the diagonals can be the same to those for the uni-planar truss provided the same steel grades are used. Only for the bottom chord the required cross section should be twice that required for the uni-planar truss, i.e. $219.1 - 11.0$ with $A_p = 7191 \text{ mm}^2$. (This section may have a longer delivery time.)

Fig. 54 – Member dimensions and steel grades

The detailed check of the members is already given in chapter 6.1 and is the same. The bracings between the top chords are determined by the horizontal loads of 54 kN at each purlin support or by loads resulting from not equally distributed loading of the roof. Since transport is simpler for V-trusses than for triangular trusses, it is also possible to use the purlins as connection between the top chords.

A simple bolted connection as given in Fig. 40 can easily transfer the shear load of 54 kN. However, in this way the truss has no torsional rigidity and cannot act as horizontal windbracing of the roof. If this is required, bracings between the top chords have to be used.

- **check joint strength**

In the table on the following page the joints have been checked in a similar way as for the uni-planar truss in chapter 6.1. However, the factor 0.9 has been included for the joint strength of joints 5, 6 and 7.

A connection without any eccentricity would result in an overlap of the bracing in the two planes (Fig. 55a). To allow welding an out-of-plane gap of 22.5 mm is chosen which results in an eccentricity of 50 mm (in-plane 43 mm). As a consequence the in-plane gap increases, resulting in lower $C_n$ values which are given between brackets.

Fig. 55 – Connection diagonals with bottom chord

If an overlap is used, it should be borne in mind that the two overlapped diagonals have the same type of loading. Now the sum of the vertical diagonal loading components should be considered and the joint strength can be determined with the uni-planar joint strength
### Check joint strength

<table>
<thead>
<tr>
<th>joint</th>
<th>chord (mm)</th>
<th>bracings (mm)</th>
<th>$d_i/d_o$</th>
<th>$d_i/t_o$</th>
<th>$Q_1$</th>
<th>$Q_2$</th>
<th>$n^* = \frac{f_{sp}}{f_{tp}}$</th>
<th>$N_1$</th>
<th>$t_{sp}$</th>
<th>$t_{tp}$</th>
<th>$f(n^*) \sin f_i$</th>
<th>$N^* \leq N$</th>
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<tr>
<td>1 – 4</td>
<td>The checks for joints 1 to 4 are given in the table in chapter 6.1</td>
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</tr>
<tr>
<td>5</td>
<td>219.1 – 11.0</td>
<td>139.7 – 4.5</td>
<td>0.64</td>
<td>19.9</td>
<td>4.5</td>
<td>(9.4)</td>
<td>+</td>
<td>0.82</td>
<td>0.38</td>
<td>3.16</td>
<td>1.60</td>
<td>&gt; 1.00( (&gt; \times 1.00))</td>
</tr>
<tr>
<td>6</td>
<td>219.1 – 11.0</td>
<td>88.3 – 3.6</td>
<td>0.64</td>
<td>19.9</td>
<td>8.2</td>
<td>(17.7)</td>
<td>+</td>
<td>0.98</td>
<td>0.35( = 0.31)</td>
<td>3.94</td>
<td>1.60</td>
<td>&gt; 1.00( (&gt; \times 1.00))</td>
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<tr>
<td>7</td>
<td>219.1 – 11.0</td>
<td>88.9 – 3.6</td>
<td>0.41</td>
<td>19.9</td>
<td>11.9</td>
<td>(21.4)</td>
<td>+</td>
<td>0.32</td>
<td>0.39( = 0.35)</td>
<td>3.94</td>
<td>1.60</td>
<td>&gt; 1.00( (&gt; \times 1.00))</td>
</tr>
</tbody>
</table>

**Notes:**
- An eccentricity $e = 38$ mm has to be introduced to satisfy the condition $g_1 + t_1$. However, for welding, a gap of 22.5 mm is required between the diagonals of both planes, which results in an eccentricity of 50 mm ($= 0.23 d_i$).
- The figures between brackets () show the estimated $g^* = g/t_o$ values for an in-plane eccentricity of $50 \cdot \cos 30^\circ = 43$ mm.
equations, however, substituting \( \frac{d_1 + d_2}{2d_0} \) for \( \frac{d_1}{d_0} \) (see Fig. 55).

The site and end support connections can be dealt with in a similar way as in the example of chapter 6.1.

### 6.3 Truss with semi-flattened end bracings

To avoid overlaps in triangular trusses it is also possible to use semi-flattened end bracings, as shown in Fig. 27.

The design of members is similar to that discussed in chapters 6.1 and 6.2, with the exception of the bracings loaded in compression, for which an out-of-plane effective length factor of 1.0 instead of 0.75 has to be taken into account.

As shown in chapter 4.6.3, the joint strength is reduced due to the fact that \( d_1 \) has to be replaced by \( d_{\text{min}} \) in the joint strength formula. However, due to the increase of \( d_1 \) to \( d_{\text{min}} \) in longitudinal direction of the chord, the gap size is decreased resulting in an increase in joint strength. Since the above mentioned effects partly compensate each other, the actual joint strength may not deviate considerably from that of a joint with profiled bracings.

Full flattening, for which design information is given in Fig. 26, is only recommended for small sized secondary structures.

### 6.4 Effective buckling length of truss members

The buckling length of members of hollow section trusses is dealt with in [7]. The evaluation of test results to design recommendations for Eurocode 3 is given in [16].

An updated Design Guide about buckling is under progress.

In the examples of chapters 6.1 and 6.2 simple reduction factors have been included for the effective lengths, i.e.:

- **chords**
  - in-plane: \( 0.9 \times \text{system length between joints} \)
  - out-of-plane: \( 0.9 \times \text{system length between the lateral supports} \)

- **bracings**
  - in- and out-of-plane: \( 0.75 \times \text{system length between joints} \)

For lateral unsupported truss chords the effective length can be considerably lower than the actual unsupported length. For example, the actual effective length of a bottom chord loaded in compression by uplift loading depends on the loading in the chord, the torsional rigidity of the truss, the bending rigidity of the truss, the bending rigidity of the purlins and the purlin to truss connections. For detailed information, reference is given to [7].

For the example given in the next figure, the buckling length of the unsupported bottom chord can be reduced to 0.32 times the chord length (L).

For the bracings in compression the buckling length \( l_e \) can be determined more precisely by [16]:

\[
\frac{l_e}{L} = 2.2 \left( \frac{d_1}{L - d_0} \right)^{0.25}
\]

\[
0.5 \leq \frac{l_e}{L} \leq 0.75
\]

(6.4.1)
6.5 Bolted connections

The bolts and the plates have to be checked in the normal way for shear, contact pressure and failure of the net cross sectional area. All national and international codes give these criteria. Furthermore, requirements regarding minimum and maximum bolt distances have to be satisfied. For high strength friction grip bolted connections special requirements are given for pre-tensioning of the bolts and the condition of the contact surfaces.

For bolted connections use is made of plates, forks, T-sections or cutouts of I-sections welded to the CHS member as shown in Fig. 50. For these connections general recommendations are not given in the codes. Hence some criteria are indicated here for full strength connections. The shear strength $t_{weld}$ and the axial strength $f_{weld}$ depend on the codes used.

Tube-fork plate connection

$$41 \cdot a \cdot f_{weld} + 41 \cdot a \cdot f_{weld} \geq A_i \cdot f_{y1}$$

$$41 \cdot t_{1} \cdot \frac{f_{y1}}{\sqrt{3}} + 41 \cdot t_{1} \cdot f_{y1} \geq A_i \cdot f_{y1}$$

Fig. 57

$A_{fork\_plate} \cdot f_{y\_plate} \geq A_i \cdot f_{y1}$

Note: Since the two halves of the tube are eccentrically loaded, the bolted connection with the fork plate should be able to withstand the moment of eccentricity.

Tube-plate connection

$$41 \cdot a \cdot f_{weld} + 21 \cdot a \cdot f_{weld} \geq A_i \cdot f_{y1}$$

$$41 \cdot t_{1} \cdot \frac{f_{y1}}{\sqrt{3}} + 21 \cdot t_{1} \cdot f_{y1} \geq A_i \cdot f_{y1}$$

Fig. 58

$A_{plate} \cdot f_{y\_plate} \geq A_i \cdot f_{y1}$

Note: To avoid premature cracking at the location where the tip of the plate is welded to the tube, some standards recommend an efficiency limit of 0.85 for $A_{plate} \cdot f_{y\_plate}$. An alternative is to extend the gusset plate outside the tube by two times the gusset plate thickness.
Tube to T-stub connection

\[ t \cdot d_1 \cdot a \cdot f_{weld} \geq A_t \cdot f_y \]  
(6.5.7)

\[ A_{\text{T-stub weld}} \cdot f_{y, \text{plate}} \geq A_t \cdot f_y \]  
(6.5.8)

\[ t_r \geq \frac{d_1 - t}{5} \]  
(6.5.9)

Note: Eq. 6.5.9 is based on a spread under 2.5 to 1 as generally used in beam to column connections.

Gusset plate connection

\[ 21 \cdot a \cdot f_{weld} \geq N_1 \cos \delta_1 + N_2 \cos \delta_2 \]  
(6.5.10)

\[ 1 \cdot t \cdot f_{y, \text{plate}} \geq N_1 \cos \delta_1 + N_2 \cos \delta_2 \]  
(6.5.11)

\[ 21 \cdot t \cdot m \cdot f_{\text{weld}} \geq N_1 \cos \delta_1 + N_2 \cos \delta_2 \]  
(6.5.12)

\[ \frac{\pi^2}{6} \cdot f_y \geq N_1 \cos \delta_1 \cdot f_y \]  
(6.5.13)

\[ \frac{21(1 + 0.25 \cdot \frac{1}{d_1}) \cdot f_{\text{weld}} \cdot t^2 \cdot n}{1 + 0.25 \cdot \frac{1}{d_1}} \cdot \sin \delta \cdot t \]  
(6.5.14)*

* See Fig. 25. The value \( 1 + 0.25 \cdot \frac{1}{d_1} \) (in formula 6.5.14) shall never exceed 2 in the calculation.

If the welds have a lower strength than the plate, the welds should also be checked for the combined effect of shear and moment.
7 Symbols

CHS  Circular hollow section
RHS  Rectangular hollow section
A,  cross sectional area
A_i  cross sectional area of member (i = 0, 1, 2, 3)
C,  joint rotational stiffness (moment per radian)
C_e  efficiency parameter (general)
C_T  efficiency parameter for T-joints
C_x  efficiency parameter for X-joints
C_y  efficiency parameter for Y-joints
E   modulus of elasticity

Eff  joint efficiency = \( \frac{N_i^*}{A_i f_{p}} \) or \( \frac{M_i^*}{M_{pl}} \) or \( \frac{M_i^{pl}}{M_{pl}} \)

\( i \)  moment of inertia
\( M_{ip} \)  applied in-plane bending moment
\( M_{op} \)  applied out-of-plane bending moment
\( M_i^* \)  joint design resistance for in-plane bending moment
\( M_{pl} \)  joint design resistance for out-of-plane bending moment
\( M_{pl} \)  plastic moment capacity of member \( i \) (\( i = 0, 1, 2, 3 \))
\( N_i \)  applied axial force in member \( i \) (\( i = 0, 1, 2, 3 \))
\( N_i^* \)  joint design resistance expressed in terms of axial load in member \( i \)
\( N_{pl} \)  characteristic joint strength expressed in terms of axial load
\( N_{op} \)  axial prestressing force in the chord, i.e. load in the chord not necessary for the equilibrium of the bracing load components
\( O_v \)  overlap, \( O_v = g/p \times 100\% \)
P   load
Q_a  characteristic load
\( T_u \)  ultimate tensile resistance of a bolt
\( V_i \)  shear force
\( W_{pe} \)  elastic section modulus of member \( i \) (\( i = 0, 1, 2, 3 \))
\( W_{pl} \)  plastic section modulus of member \( i \)
a   throat thickness of a weld
b_i  width of plate
c   coefficient
d   bolt diameter
d_i  external diameter of member \( i \) (\( i = 0, 1, 2, 3 \))
e   eccentricity of noding
\( e_1, e_2 \)  edge distance
\( f_i \)  axial stress
\( f_{weld} \)  design resistance of a weld for axial loading perpendicular to the weld
\( f_{weld} \)  design resistance of a weld for shear loading
\( f_b \)  bending stress
\( f_i \)  axial stress in member \( i \) (\( i = 0, 1, 2, 3 \))
\( f_{pl} \)  specified design yield strength
\( f_{pl} \)  specified design yield strength of a plate
\( f_{pl} \)  specified design yield strength of member \( i \) (\( i = 0, 1, 2, 3 \))
\( f_{op} \)  maximum applied axial stress in chord, or maximum stress due to axial force and bending moment where moment is taken into account
\( f(n') \)  function which incorporates the chord prestress in the joint strength equation
g  
- gap between the bracings of a K-, N- or KT-joint, at the connection face of the chord

\( g' \)  
- gap divided by chord wall thickness, \( g' = g/t_0 \)

\( h_i \)  
- depth of plate or depth of an I or RHS bracing section

\( i \)  
- integer used to denote member of joint as follows:

0: chord
1: refers in general to a bracing for T-, Y- and X-joints
2: refers to the compression bracing for K-, N- and KT-joints
3: refers to the tensile bracing for K-, N- and KT-joints
4: refers to the vertical for KT-joints

\( l_i, L_i \)  
- length

\( l_e \)  
- effective length for buckling

\( n \)  
- number of bolts

\( n' \)  
- number of bracings

\( l_{po} = \frac{A_p}{W_p} + \frac{M_p}{E_d} \)

\( l_{po} \)  
- projected length of overlap between bracings and chord without presence of the overlapped bracing

\( p \)  
- length of projected contact area between overlapping bracing and chord without presence of the overlapped bracing

\( q \)  
- projected length of overlap between bracings of a K- or N-joint, at the chord face (see Fig. 2)

\( r_1, r_2 \)  
- parameters used for bolted flange connections, i.e.

\( r_1 = d_i/2 + 2\varepsilon_1 \)

\( r_2 = d_i/2 + \varepsilon_1 \)

\( t \)  
- thickness

\( t_i \)  
- end plate or flange thickness

\( t_h \)  
- thickness of hollow section member \( i (i = 0, 1, 2, 3) \)

\( \beta \)  
- diameter ratio between bracings and chord

\( \beta = \frac{d_i}{d_h}, (T, Y, X); \quad \beta = \frac{d_i + d_2}{d_h}, (K) \)

or ratio between plate or section width and chord diameter (TP- and XP-joints)

\( \gamma \)  
- half diameter to thickness ratio of the chord, \( \gamma = d_i/2t_0 \)

\( \gamma_m \)  
- material and joint partial safety factor

\( \gamma_l \)  
- load factor

\( \theta_i \)  
- acute angle between bracing member \( i (i = 1, 2, 3) \) and the chord

\( \phi \)  
- angle between bracing members in a multi-planar girder

\( \alpha \)  
- bracing plate or section depth divided by chord diameter (TP- and XP-joints)

\( \lambda \)  
- reduction factor for buckling

Rem: The nominal values of dimensions and the design values for the yield strength should be used in the joint strength formula.
8 References


[27] Kato, B., Hirose, A.: Bolted tension flanges joinin circular hollow section members, Cibed report 8C-84/24-E.


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International Committee
for the Development and Study
of Tubular Structures

CIDECT founded in 1962 as an international association joins together the research
resources of major hollow steel section manufacturers to create a major force in the
research and application of hollow steel sections worldwide.

The objectives of CIDECT are:

☐ to increase knowledge of hollow steel sections and their potential application by
initiating and participating in appropriate researches and studies

☐ to establish and maintain contacts and exchanges between the producers of the
hollow steel sections and the ever increasing number of architects and engineers
using hollow steel sections throughout the world.

☐ to promote hollow steel section usage wherever this makes for good engineering
practice and suitable architecture, in general by disseminating information,
organizing congresses etc.

☐ to co-operate with organizations concerned with practical design recommenda-
tions, regulations or standards at national and international level.

Technical activities

The technical activities of CIDECT have centred on the following research aspects of
hollow steel section design:

☐ Buckling behaviour of empty and concrete-filled columns
☐ Effective buckling lengths of members in trusses
☐ Fire resistance of concrete-filled columns
☐ Static strength of welded and bolted joints
☐ Fatigue resistance of joints
☐ Aerodynamic properties
☐ Bending strength
☐ Corrosion resistance
☐ Workshop fabrication

The results of CIDECT research form the basis of many national and international
design requirements for hollow steel sections.
CIDECT, the future

Current work is chiefly aimed at filling up the gaps in the knowledge regarding the structural behaviour of hollow steel sections and the interpretation and implementation of the completed fundamental research. As this proceeds, a new complementary phase is opening that will be directly concerned with practical, economical and labour saving design.

CIDECT Publications

The current situation relating to CIDECT publications reflects the ever increasing emphasis on the dissemination of research results.

Apart from the final reports of the CIDECT sponsored research programmes, which are available at the Technical Secretariat on demand at nominal price, CIDECT has published a number of monographs concerning various aspects of design with hollow steel sections. These are available in English, French and German as indicated.

Monograph No. 3 – Windloads for Lattice Structures (E, F, G)
Monograph No. 4 – Effective Lengths of Lattice Girder Members (E, F, G)
Monograph No. 5 – Concrete-filled Hollow Section Columns (E, F)
Monograph No. 6 – The Strength and Behaviour of Statically Loaded Welded Connections in Structural Hollow Sections (E)
Monograph No. 7 – Fatigue Behaviour of Hollow Section Joints (E, G)

A book “Construction with Hollow Steel Sections”, prepared under the direction of CIDECT in English, French, German and Spanish, was published with the sponsorship of the European Community presenting the actual state of the knowledge acquired throughout the world with regard to hollow steel sections and the design methods and application technologies related to them.

In addition, copies of these publications can be obtained from the individual members given below to whom technical questions relating to CIDECT work or the design using hollow steel sections should be addressed.

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Mr. D. Dutta
Office of the Chairman of the CIDECT Technical Commission
c/o Mannesmannröhrnen-Werke AG
Mannesmannufer 3
D-4000 Düsseldorf 1
Federal Republic of Germany

Telephone: (49) 211/875-34 80
Telex: 8 581 421
Telefax: (49) 211/875-46 89

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