DESIGN GUIDE
FOR RECTANGULAR HOLLOW SECTION (RHS) JOINTS UNDER PREDOMINANTLY STATIC LOADING

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

Verlag TÜV Rheinland
DESIGN GUIDE
FOR RECTANGULAR HOLLOW SECTION (RHS) JOINTS
UNDER PREDOMINANTLY STATIC LOADING
CONSTRUCTION
WITH HOLLOW STEEL
SECTIONS

Edited by: Comité International pour le Développement et l'Étude
de la Construction Tubulaire
Authors: Jeffrey A. Packer, University of Toronto
Jaap Wardenier, Delft University of Technology
Yoshiaki Kurobake, Kumamoto University
Dipak Dutta, Chairman Technical Commission of Cidect
Noel Yeomans, Chairman Cidect Joint and Fatigue Working Group
DESIGN GUIDE
FOR RECTANGULAR HOLLOW SECTION (RHS) JOINTS UNDER PREDOMINANTLY STATIC LOADING

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

Verlag TÜV Rheinland
Preface

Square and rectangular hollow section is the youngest member in the family of steel sections. Their industrial production started as late as in 1959 in England applying the process of forming a round section first and then squaring the round in hot or cold operation conditions. They possess practically all the favourable structural qualities of circular hollow sections in various degrees, such as high ratio of strength to weight, higher compressive load than that of conventional open sections, effective cross section for resisting torsional moments (material is uniformly distributed about the polar axis), low surface area (less painting and external fire protection leading to economy in maintenance), lack of ledges and edges (clean structures), closed aerodynamic shape (reduced wind or wave loads), and aesthetical appearance (pleasing architecture). The inner space in both CHS and RHS gives possibilities of increasing the strength of the sections by filling them with concrete, which, as the research works in recent years show, also raises the fire resistance time. Other facilities due to the hole in the hollow sections are heating and ventilation of buildings by water or air circulation. However, the distinct advantage of applying RHS in lieu of CHS emerges in the field of fabrication, where the facility of end preparation with flat cuts for RHS is significantly easier and more economical than that with profile cuts for CHS. The trend in fabrication techniques with hollow sections, mainly by welding and bolting, is increasingly towards simple types of joints i.e. whenever possible joints without gussets, stiffeners or other means of reinforcement, since the ratio of labour to material costs has increased rapidly in all industrialized countries in the last thirty years. Due to the very shape of hollow sections and the mere fact that they are closed sections, the welding is by far the most appropriate and common fabrication technique for hollow Section joints. bolted connections remain nonetheless desirable in many cases and especially for site joints between prefabricated subassemblies. It is however of significant importance that the behaviour of these joints depending on various geometrical parameters and loading types should be understood before they can be correctly designed and calculated. This knowledge is precisely necessary to establish, first of all, simple design formulae and then to select particular structural arrangements conducive to a rational and economical application of hollow sections.

In order to obtain this knowledge, which was nearly totally missing in the early sixties as regards RHS joints, CIDECT from its very inception in 1962 concentrated its research activities predominantly on the various aspects of RHS jointing and some fifty individual projects have been undertaken up to now in the research institutes of various parts of the world with CIDECT involvement. The results of the CIDECT research works on the static behaviour of RHS joints together with those obtained from other sources form the basis of this book presenting the guide lines to design RHS joints in structures and the simple formulae to calculate them. The aim is to disseminate to the architects, engineers and constructors the most recent design methods for RHS joints along with worked-out examples, which will enable them to design and construct secure and economical RHS structures.

It is worth mentioning that the CIDECT research results have been used in many national and international design recommendations, e.g. DIN (Deutsche Industrie Normung German Standard), NF (Norme Francaise - French standard), BS (British Standard), ACNOR/CSA (Canadian Standard), AJI (Architectural Institute of Japan), AWS (American Welding Standard) und IWIW (International Institute of Welding). The design formulae and guide lines given in this book have been fully incorporated into Eurocode 3 (ENV 1993-1-1) "Design of steel structures, Part 1: General rules and rules for buildings, February 1992". This design guide is the third of a series of five in total, which CIDECT is in the process of publishing recently:
- Design guide for circular hollow section (CHS) joints under predominantly static loading
- Structural stability of hollow sections
- Design guide for rectangular hollow section (RHS) joints under predominantly static loading
- Design guide for hollow section columns susceptible to fire
- Design guide for circular and rectangular hollow section joints under fatigue loading

We express our sincere thanks to the three very well-known researchers in the field of application technology of hollow sections – Professor Jeff Packer of University of Toronto, Canada, Professor Jaap Wardenier of Delft University of Technology, The Netherlands and Professor Yoshiaki Kurobane of Kumamoto University, Japan – who wrote this book in close cooperation. Our special thanks go to Mr. D. Grotmann of Technical University of Aix-la-Chapelle, Germany, who checked the contents of the book and made many important comments.

Further we acknowledge the support of the CIDECT member firms with sincere thankfulness.

Dipak Dutta
Chairman of the Technical Commission
CIDECT
# Contents

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 General: Advantages of hollow structural sections, and RHS relative to CHS</td>
<td>9</td>
</tr>
<tr>
<td>2 Design of tubular structures</td>
<td>14</td>
</tr>
<tr>
<td>2.1 Truss configurations</td>
<td>14</td>
</tr>
<tr>
<td>2.2 Truss analysis</td>
<td>14</td>
</tr>
<tr>
<td>2.3 Effective lengths for compression members</td>
<td>16</td>
</tr>
<tr>
<td>2.3.1 Simplified rules</td>
<td>16</td>
</tr>
<tr>
<td>2.3.2 Empirical method for bracing members</td>
<td>17</td>
</tr>
<tr>
<td>2.3.3 Long, laterally unsupported compression chords</td>
<td>17</td>
</tr>
<tr>
<td>2.4 Truss deflections</td>
<td>17</td>
</tr>
<tr>
<td>2.5 General connection considerations</td>
<td>17</td>
</tr>
<tr>
<td>2.6 Truss design procedure</td>
<td>18</td>
</tr>
<tr>
<td>2.7 Design of welds</td>
<td>19</td>
</tr>
<tr>
<td>3 Welded uniplanar truss connections between RHS chords and RHS or CHS bracing members</td>
<td>22</td>
</tr>
<tr>
<td>3.1 K and N connections</td>
<td>23</td>
</tr>
<tr>
<td>3.2 T, Y and X connections</td>
<td>29</td>
</tr>
<tr>
<td>3.3 KT connections</td>
<td>30</td>
</tr>
<tr>
<td>3.4 Graphical design charts</td>
<td>32</td>
</tr>
<tr>
<td>3.5 Reinforced connections</td>
<td>41</td>
</tr>
<tr>
<td>3.5.1 With stiffening plates</td>
<td>41</td>
</tr>
<tr>
<td>3.5.1.1 K and N connections</td>
<td>41</td>
</tr>
<tr>
<td>3.5.1.2 T, Y and X connections</td>
<td>42</td>
</tr>
<tr>
<td>3.5.2 With concrete filling</td>
<td>44</td>
</tr>
<tr>
<td>3.5.3 Design example</td>
<td>46</td>
</tr>
<tr>
<td>3.6 Cranked-chord connections</td>
<td>49</td>
</tr>
<tr>
<td>3.6.1 Design example</td>
<td>49</td>
</tr>
<tr>
<td>4 Truss design examples</td>
<td>51</td>
</tr>
<tr>
<td>4.1 Uniplanar truss</td>
<td>51</td>
</tr>
<tr>
<td>4.2 Arch-formed truss</td>
<td>56</td>
</tr>
<tr>
<td>5 Bolted connections</td>
<td>57</td>
</tr>
<tr>
<td>5.1 Flange-plate connections</td>
<td>57</td>
</tr>
<tr>
<td>5.1.1 Bolted on two sides of the RHS</td>
<td>57</td>
</tr>
<tr>
<td>5.1.2 Bolted on four sides of the RHS</td>
<td>59</td>
</tr>
<tr>
<td>5.1.3 Design example for bolted flange-plate connection</td>
<td>59</td>
</tr>
<tr>
<td>5.2 RHS to gusset-plate connections</td>
<td>60</td>
</tr>
<tr>
<td>5.2.1 Net area, effective net area, and reduced effective net area</td>
<td>61</td>
</tr>
<tr>
<td>6 RHS to RHS moment connections</td>
<td>63</td>
</tr>
<tr>
<td>6.1 Vierendeel connections</td>
<td>63</td>
</tr>
<tr>
<td>6.1.1 Introduction to Vierendeel trusses</td>
<td>63</td>
</tr>
<tr>
<td>6.1.2 Connection behaviour and strength</td>
<td>64</td>
</tr>
<tr>
<td>6.1.2.1 In-plane bending moments for T and X connections</td>
<td>70</td>
</tr>
</tbody>
</table>
Roof support structure for a swimming centre
1 General: Advantages of hollow structural Sections, and RHS relative to CHS

The structural advantages of hollow sections have become apparent to most designers, particularly for structural members loaded in compression or tension. Circular hollow sections (CHS) have a particularly pleasing shape and offer a very efficient distribution of steel about the centroidal axes, as well as the minimum possible resistance to fluid, but specialized profiling is needed when joining circular shapes together. As a consequence, rectangular hollow sections (RHS) have evolved as a practical alternative, allowing easy connections to the flat face, and they are very popular for columns and trusses.

Fabrication costs of all structural steelwork are primarily a function of the labour hours required to produce the structural components. These need not be more with hollow section design (RHS or CHS) than with open sections, and can even be less depending on connection configurations. In this regard it is essential that the designer realize that the selection of hollow structural section truss components, for example, determines the complexity of the connections at the panel points. It is not to be expected that members selected for minimum mass can be joined for minimum labour time. That will seldom be the case because the efficiency of hollow section connections is a subtle function of a number of parameters which are defined by relative dimensions of the connecting members.

Handling and erecting costs can be less for hollow section trusses than for alternative trusses. Their greater stiffness and lateral strength mean they are easier to pick up and more stable to erect. Furthermore, trusses comprised of hollow sections are likely to be lighter than their counterparts fabricated from non-tubular sections, as truss members are primarily axially loaded and hollow structural sections represent the most efficient use of a steel cross-section in compression.

Protection costs are appreciably lower for hollow section trusses than for other trusses. A square hollow section has about 2/3 the surface area of the same size I-section shape, and hollow section trusses may have smaller members as a result of their higher structural efficiency. The absence of re-entrant corners makes the application of paint or fire protection easier and the durability is longer. Rectangular (which includes square) hollow sections, if closed at the ends, also have only four surfaces to be painted, whereas an I-section has eight flat surfaces for painting. These combined features result in less material and less labour for hollow section structures.

Regardless of the type of shape used to design a truss, it is generally false economy to attempt to minimize mass by selecting a multitude of sizes for bracing members. The increased cost to source and to separately handle the various shapes more than offsets the apparent savings in materials. It is therefore better to use the same section size for a group of bracing members. Circular hollow section connections are more expensive to fabricate than rectangular hollow section connections. Joints of CHS require that the tube ends be profile cut when the tubes are to be fitted directly together, unless the bracing tubes are much smaller than the chords. More than that, the bevel of the end cut must generally be varied for welding access as one progresses around the tube, if automated equipment for this purpose is not available, semi automatic or manual profile cutting has to be used, which is much more expensive than straight bevel cuts on RHS.

In structures where deck or paneling is laid directly on the top chord of trusses, RHS offer superior surfaces to CHS for attaching and supporting the deck. Other aspects to consider when choosing between circular and rectangular hollow sections are the relative ease of fitting weld backing bars to RHS, and of handling and stacking RHS. The latter is important because material handling is said to be the highest cost in the shop.

Joint configurations are increasingly cheaper progressing from partial overlap, complete overlap, to gap (see Fig 31). Gap joints have the advantage of a single bevel cut, if the chord is
Fig. 1 – Noding eccentricity, with permitted limits between which the resulting moment on the connection can be ignored, for connection design.

Aircraft museum; RHS columns and roof girders
an RHS, and complete ease of fitting. Partial overlap joints have double cuts with minimum flexibility in fitting.

This detracts from the initial appeal of overlap connections, which the designer will come to recognize as usually having superior static and fatigue connection strength compared to gap connections. Also, when the concealed portion of an overlapped bracing member needs to be welded, it must be done before the overlapping bracing member is fitted. This prevents fitting and tack welding of all members prior to final structural welding, an economical sequence preferred by many fabricators.

Welding costs are sensitive to joint geometry, weld type and weld size. Fillet welds usually do not require the preparation of bevel surfaces that is inherent in almost all partial or full penetration butt welds. A 12 mm fillet weld has twice the resistance of a 6 mm fillet; however, it has four times the volume. Therefore cost per unit resistance is clearly better with smaller member thicknesses and thus with smaller size welds. An RHS bracing member whose width is the same as an RHS chord member presents a condition where the side walls of the bracing line up with the round corners of the chord. Depending on the corner radius and the wall thicknesses, at best there is a flare bevel joint (more awkward than a fillet), or more likely,
especially for cold formed sections a flare bevel with a gap requiring a custom fitted backing bar. Thus, less-than-full width RHS bracing to chord connections are more economical. Connection pieces such as gusset plates obviously add material and labour costs. Welding is essentially doubled because loads are transferred twice instead of once, first from a member to the connecting piece, then from that piece to another member. Hence, direct connection of one hollow section to another is to be preferred.

Stiffeners and other reinforcement, which similarly increase costs of material and labour, should always be kept to a minimum and used only when truly needed (e.g. for repair). The design recommendations presented in the Guide are applicable for hollow sections having a nominal yield stress of up to 355 N/mm² and a nominal yield to ultimate stress ratio \( (f_y/f_u) \leq 0.8 \). The scope of this guide covers connections subjected to “predominantly static loading” and alternative design philosophies may be required under different loading conditions. (For example, under earthquake loading, when connection resistances greater than member resistances may be required.)
Container lifting vehicle
2 Design of tubular structures

This chapter deals with the design philosophy applicable to triangulated (e.g. Warren or Pratt) planar RHS trusses with bracing members directly welded to single-section chord members.

2.1 Truss configurations

Some of the common truss types are shown in Fig. 2. Warren trusses will generally provide the most economical solution since their long compression bracing members can take advantage of the fact that RHS are very efficient in compression. They have about half the number of bracing members and half the number of connections compared to Pratt trusses, resulting in considerable labour and cost savings. The panel points of a Warren truss can be located at the load application points on the chord, if necessary with an irregular truss geometry, even if the chord is loaded in bending, that disadvantage is usually less significant with RHS chords than with alternatives. If support is required at all load points to a chord (for example, to reduce the unbraced length), a modified Warren truss could be used rather than a Pratt truss by adding vertical members as shown in Fig. 2(a).

Warren trusses provide greater opportunities to use gap joints, the preferred arrangement at panel points. Also, when possible, a regular Warren truss achieves a more “open” truss suitable for practical placement of mechanical, electrical, and other services.

Truss depth is determined in relation to the span, loads, maximum deflection, etc., with increased truss depth reducing the loads in the chord members and increasing the lengths of the bracing members. The ideal span to depth ratio is usually found to be between 10 and 15 [1]. If the total costs of the building are considered, a ratio nearer 15 will represent optimum value.

![Fig. 2 - Common RHS trusses](image)

(a) Warren trusses (modified Warren with verticals)
(b) Pratt truss (shown with a sloped roof, but may have parallel chords)
(c) Fink truss
(d) U-framed truss

2.2 Truss analysis

Elastic analysis of RHS trusses is frequently performed by assuming that all members are pin connected. Nodal eccentricities between the centrelines of intersecting members at panel points should preferably be kept within the limits shown in Fig. 1. These eccentricities produce primary bending moments which, for a pinned joint analysis, need only be taken into account in member design when proportioning the compression chord, by treating it as a beam.
column. This is done by distributing the panel point moment (sum of the horizontal components of the bracing member forces multiplied by the nodal eccentricity) to the chord on the basis of relative chord stiffness on either side of the connection (i.e. in proportion to the values of moment of inertia divided by chord length to the next panel point, on either side of the connection). The eccentricity moments can be ignored for the design of the tension chord and bracing members. Eccentricity moments can be ignored for the design of the connections provided that the eccentricities are within the limits shown in Fig. 1. If these eccentricity limits are violated, the eccentricity moment may have a detrimental effect on connection strength and the eccentricity moment must be distributed between the members at a connection. If moments are distributed to the bracing members, the connection capacity must then be checked for the interaction between axial load and bending moment, for each bracing member.

A rigid joint frame analysis is not recommended for most planar, triangulated, single-chord, directly-welded trusses, as it generally tends to exaggerate bracing member moments, and the axial force distribution will still be similar to that for a pin-jointed analysis. Transverse loads applied to either chord away from the panel points produce primary moments which must always be taken into account when designing the chords.

Computer plane frame programs are regularly used for truss analysis. In this case the truss can be modelled by considering a continuous chord with bracing members pin connected to it at distances of $\pm e$ or $-e$ from it (e being the distance from the chord centreline to the intersection of the bracing member centrelines). The links to the pins are treated as being extremely stiff as indicated in Fig. 3. The advantage of this model is that a sensible distribution of bending moments is automatically generated throughout the truss, for cases in which bending moments need to be taken into account in the design of the chords.

Fig. 3 - Plane frame joint modelling assumptions to obtain realistic forces for member design

| type of moment | primary | primary | secondary 
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>moments due to</td>
<td>nodal eccentricity</td>
<td>transverse member loading</td>
<td>secondary effects such as local deformations</td>
</tr>
<tr>
<td>compression chord design</td>
<td>yes</td>
<td>yes</td>
<td>no</td>
</tr>
<tr>
<td>design of other members</td>
<td>no</td>
<td>yes</td>
<td>no</td>
</tr>
<tr>
<td>design of connections</td>
<td>no, provided eccentricity limits are not exceeded</td>
<td>yes, influences f(n)</td>
<td>no, provided parametric limits of validity are met</td>
</tr>
</tbody>
</table>
Secondary moments, resulting from end fixity of the bracing members to a flexible chord wall, can generally be ignored for both members and connections, provided that there is deformation and rotation capacity adequate to redistribute stresses after some local yielding at the connections. This is the case when the prescribed geometric limits of validity for design formulae given in the next chapter are followed. Welds in particular need to have potential for adequate stress redistribution without premature failure, and this will be achieved with the recommendations given in Section 2.7.

Table 1 summarizes when moments need to be considered for designing an RHS truss. Plastic design could be used to proportion the chords of a truss by considering them as continuous beams with pin supports from the bracing members. In such a design the plastically designed members must be plastic design sections and the welds must be sized to develop the capacity of the connected bracing members.

### 2.3 Effective lengths for compression members

To determine the effective length $K_L$ for a compression member in a truss, the effective length factor $K$ can always be conservatively taken as 1.0. However, considerable end restraint is generally present for compression members in an RHS truss, and it has been shown that $K$ is generally appreciably less than 1.0. This restraint offered by members framing into a connection could disappear, or be greatly reduced, if all members were designed optimally for minimum mass, thereby achieving ultimate capacity simultaneously under static loading.

In practice, design for optimal or minimum mass will rarely coincide with minimum cost; the bracing members are usually standardized to a few selected dimensions (perhaps even two), to minimize the number of section sizes for the truss. In the unlikely situation that all compression bracing members are proportioned on the basis of a single load combination and all reach their compressive resistances at approximately the same truss loading, an effective length factor of 1.0 is recommended.

CIDECT has sponsored and co-ordinated extensive research work to specifically address the determination of effective lengths in hollow section trusses, resulting in reports from CIDECT Programs 3E-3G and Monograph No. 4. More recently, a re-evaluation of all test results has been undertaken to produce recommendations for Eurocode 3. This has resulted in the following effective length recommendations, which are implemented in the truss design example given in this guide (chapter 4).

#### 2.3.1 Simplified rules

For RHS chord members:

- In the plane of the truss
  \[ K_L = 0.9 \frac{L}{L} \]  \hspace{1cm} \text{where } L \text{ is distance between chord panel points.} \tag{2.1}

- In the plane perpendicular to the truss
  \[ K_L = 0.9 \frac{L}{L} \]  \hspace{1cm} \text{where } L \text{ is the distance between points of lateral support for the chord.} \tag{2.2}

For RHS or CHS bracing members:

- In either plane
  \[ K_L = 0.75 \frac{L}{L} \]  \hspace{1cm} \text{where } L \text{ is the panel point to panel point length of the member.} \tag{2.3}

These values of $K$ are only valid for RHS members which are connected around the full perimeter of the member, without cropping or flattening of the members. Compliance with the connection design requirements of chapter 3 will likely place even more restrictive control on the member dimensions.
2.3.2 Empirical method for bracing members

For trusses with identical width RHS top and bottom chord members and with a:

circular bracing member welded to rectangular chords
\[ K = 2.35 \left( \frac{d}{b} \right) \left( \frac{L_b}{h} \right)^{0.25} \text{ but } 0.5 \leq K \leq 0.75 \]  \hfill (2.4)

rectangular bracing member welded to rectangular chords
\[ K = 2.3 \left( \frac{b}{L_b} \right) \left( \frac{L_b}{h} \right)^{0.25} \text{ but } 0.5 \leq K \leq 0.75 \]  \hfill (2.5)

where L is again the panel point to panel point length of the bracing member and “rectangular” includes “square”.

CIDECT Monograph No. 4 [4] presented a method for determining a bracing member effective length in trusses which had different width, or section shape, members for the top and bottom chords. This has not been addressed in the latest provisions for Eurocode 3 [10], so it is recommended that the effective length factor K be calculated for the connection condition at each end of the bracing member, and the higher value be used. Another conservative rider from CIDECT Monograph No. 4, which impacts upon Eqns. 2.4 and 2.5 above, should also be added to the above recommendations:

For RHS chord members, \( b_r \) is replaced by \( h_r \) when \( h_r < b_r \).
For RHS bracing members, \( b_r \) is replaced by \( h_r \) when \( h_r > b_r \).

2.3.3 Long, laterally unsupported compression chords

Long, laterally unsupported compression chords can exist in pedestrian bridges such as U-framed trusses, and in roof trusses subjected to large wind uplift. The effective length of such laterally unsupported truss chords can be considerably less than the unsupported length. For example, the actual effective length of a bottom chord, loaded in compression by uplift, depends on the loading in the chord, the stiffness of the bracing members, the torsional rigidity of the tension chord, the purlin to truss connections and the bending stiffness of the purlins. The bracing members act as local elastic supports at each panel point. When the stiffness of these elastic supports is known, the effective length of the compression chord can be calculated. A detailed method for effective length factor calculation has been given by CIDECT Monograph No. 4 [4].

2.4 Truss deflections

For the purpose of checking the serviceability condition of overall truss deflection under specified (unfactored) loads, an analysis with all members being pin-jointed will provide a conservative (over) estimate of truss deflections when all the connections are overlapped [11, 12]. A better assumption for overlap conditions is to assume continuous chord members and pin-jointed bracing members. However, for gap-connected trusses, a pin-jointed analysis still generally underestimates overall truss deflections, because of the flexibility of the connections [11, 12, 13, 14]. At the service load level, gap-connected RHS truss deflections have been underestimated by around 12 to 15% [11, 13]. Thus, a conservative approach for gap-connected RHS trusses is to estimate the maximum truss deflection by 1.15 times that calculated from a pin-jointed analysis.

2.5 General connection considerations

It is essential that the designer has an appreciation of factors which make it possible for RHS members to be connected together at truss panel points without extensive (and expensive)
reinforcement. Apparent economies from minimum-mass member selection will quickly
vanish at the connections if a designer does not have a knowledge of the critical
considerations which influence connection efficiency.

1. Except for 100% overlap connections, chords should generally have thick walls rather than
thin walls. The stiffer walls resist loads from the bracing members more effectively, and the
connection resistance thereby increases as width to thickness ratios decrease. For the
compression chord, however, a large thin section is more efficient in providing buckling
resistance, so for this member the final RHS wall slenderness will be a compromise
between connection strength and buckling strength, and relatively stocky sections will
usually be chosen.

2. Bracing members should have thin walls rather than thick walls (except overlap
connections), as connection efficiency increases as the ratio of chord wall thickness to
bracing wall thickness increases. In addition, thin bracing member walls will require
smaller fillet welds for a prequalified joint.

3. Ideally, RHS bracing members should not be the same width as RHS chord members, as
this can present an awkward flare bevel weld situation for the joint at the corner of the chord
section. A preferred arrangement is bracing members just sufficiently narrower than the
chord to permit the bracing member, and some of the fillet weld, to sit on the “flat” of the
RHS chord member.

4. Gap connections (for K and N situations) are preferred to overlap connections because the
members are easier to prepare, fit and weld.

5. When overlap connections are used, at least a quarter of the width (in the plane of the truss)
of the overlapping member needs to be engaged in the overlap, however 50% is
preferable.

6. An angle of less than 30° between a bracing member and a chord creates serious welding
difficulties and is not covered by the scope of these recommendations (see section 2.7).

2.6 Truss design procedure

In summary, the design of an RHS truss should be approached in the following way to obtain
an efficient and economical structure.

I. Determine the truss layout, span, depth, panel lengths, truss and lateral bracing by the
usual methods, but keep the number of connections to a minimum.

II. Determine loads at connections and on members; simplify these to equivalent loads at
the panel points.

III. Determine axial forces in all members by assuming that joints are pinned and that all
member centrelines are nodding.

IV. Determine chord member sizes by considering axial loading, corrosion protection and
tube wall slenderness. (Usual width to thickness ratios are 15 to 25.) An effective length
factor of $K = 0.9$ can be used for the design of the compression chord.

V. Determine bracing member sizes based on axial loading, preferably with thicknesses
smaller than the chord thickness. The effective length factor for the compression bracing
members can initially be assumed to be 0.75 (see section 2.3.1).

VI. Standardize the bracing members to a few selected dimensions (perhaps even two), to
minimize the number of section sizes for the structure. Consider availability of all
sections when making member selections. For aesthetic reasons, a constant outside
member width may be preferred for all bracing members, with wall thicknesses varying; but this will require special quality control procedures in the fabrication shop.

VII. Layout the connections, trying gap joints first. Check that the connection geometry and member dimensions satisfy the validity ranges for the dimensional parameters given in Chapter 3, with particular attention to the eccentricity limits. Consider the fabrication procedure when deciding on a connection layout.

VIII. Check the connection efficiencies with the charts given in Chapter 3. In some instances (for example when using rectangular rather than square RHS members, or when a slightly more accurate solution is needed), direct use of the connection factored resistance equations given in Chapter 3 may be required.

IX. If the connection resistances (efficiencies) are not adequate, modify the connection layout (for example, overlap rather than gap), or modify the bracing or chord members as appropriate, and recheck the connection capacities. Generally, only a few connections will need checking.

X. Check the effect of primary moments on the design of the chords. For example, use the proper load positions (rather than equivalent panel point loading); determine the bending moments in the chords by assuming either: (a) pinned joints everywhere or (b) continuous chords with pinned bracing members. For the compression chord, also determine the bending moments produced by any noding eccentricities, by using either of the above analysis assumptions. Then check that the factored resistance of all chord members is still adequate, under the influence of both axial loads and primary bending moments.

XI. Check truss deflections (see section 2.4) at the specified (unfactored) load level, using the proper load positions.

XII. Design welds.

2.7 Design of welds

Except for certain K and N connections with partially overlapped bracing members (as noted below), a welded joint should be established around the entire perimeter of a bracing member by means of a butt weld, a fillet weld, or a combination of the two. Fillet welds which are automatically prequalified for any bracing member loads should be designed to give a resistance that is not less than the bracing member capacity. This results in the following minimum throat thickness \( t \) for fillet welds around bracing members, assuming matched electrodes and ISO steel grades [2]:

\[
\begin{align*}
\text{a} &> 0.95 \ t, \text{ for } Fe \ 360 (f_y = 235 \text{ N/mm}^2) \\
\text{a} &> 1.00 \ t, \text{ for } Fe \ 430 (f_y = 275 \text{ N/mm}^2) \\
\text{a} &> 1.07 \ t, \text{ for } Fe \ 510 (f_y = 355 \text{ N/mm}^2)
\end{align*}
\]

If welds are proportioned on the basis of particular bracing member loads, the designer must recognize that the entire length of the weld may not be effective, and the model for the weld resistance must be justified in terms of strength and deformation capacity. An effective length of RHS bracing member welds in planar, gap, K and N connections subjected to predominantly static axial load, is given by [16]:

\[
\text{Effective length} = (2h/\sin \theta) + b_e \quad \text{for } \theta \geq 60^\circ \quad (2.6)
\]

\[
\text{Effective length} = (2h/\sin \theta) + 2b_e \quad \text{for } \theta \leq 50^\circ \quad (2.7)
\]

For \( 50^\circ < \theta < 60^\circ \), a linear interpolation has been suggested [16].
For overlapped K and N connections, limited experimental research on connections with 50% overlap has shown that the entire overlapping bracing member contact perimeter can be considered as effective [15].

These recommendations for effective weld lengths in RHS K and N connections satisfy the required safety levels for use in conjunction with both European and North American steelwork specifications [15]. Based on the weld effective lengths for K and N connections, an extrapolation has been postulated for RHS T, Y and X connections under predominantly static load [16, 17].

Effective length = \( (2h/\sin \theta) \)  

\[ (2.8) \]
With overlapped K and N connections, welding of the toe of the overlapped member to the chord is particularly important for 100% overlap situations. For partial overlaps, the toe of the overlapped member need not be welded, providing the components, normal to the chord, of the bracing member forces do not differ by more than about 20%. When these force components do not balance, the more heavily loaded bracing member should be the "through member" and its full circumference should be welded to the chord. Generally, the weaker member (defined by wall thickness times yield strength) should be attached to the stronger member, regardless of the load type.

It is generally more economical to use fillet welds than butt (groove) welds, providing the fillet weld sits on the "flat" of the RHS member, however the upper limit on throat or leg size for fillet welds will depend on the fabricator. Most welding specifications only allow fillet welding at the toe of a bracing member if \( \theta_1 \geq 60^\circ \). Because of the difficulty of welding at the heel of a bracing member at low \( \theta_1 \) values, a lower limit for the applicability of the connection design rules given herein has been set at \( \theta_1 = 30^\circ \). Some recommended weld details\(^2\) are illustrated in Fig. 4.
3 Welded uniplanar truss connections between RHS chords and RHS or CHS bracing members

All connection design formulas presented in this guide are given in limit states strength (or LRFD\textsuperscript{*}) terms. This means that the effect of the factored loads (the specified or unfactored loads or characteristic loads multiplied by the appropriate load factors), should not exceed the factored resistance of the connection, $N^*$ . The connection factored resistance expressions given in this guide, in general, already include a material and connection partial safety factor ($R_n$) or connection resistance factor ($q$). If allowable stress design (ASD), or working stress design, is used, the connection resistance expressions should in addition be divided by an appropriate load factor to obtain allowable connection resistances. In this case a load factor of 1.5 is recommended.

This guide uses terminology adopted by CIDECT and IIW (International Institute of Welding) to define connection parameters, wherever possible. Fig. 5 shows some of the common connection parameters for gap and overlap uniplanar K connections. Definitions of all terms are given in the List of Symbols.

![Connection shown isolated below](image)

**Fig. 5** - Typical RHS chord truss K-connection

Eccentricity, $e$, is positive when measured towards the outside of a chord, and negative towards the inside. The gap or overlap, $g$ or $q$ respectively, as well as the eccentricity, $e$, may be calculated by Eqns. 3.1 and 3.2 below. See Figs. 1, 5 and 16 for illustration of the connection parameters.

$$g = \left( e + \frac{h_0}{2} \right) \frac{\sin \left( \theta_1 + \theta_2 \right)}{\sin \theta_1 \sin \theta_2} - \frac{h_1}{2 \sin \theta_1} - \frac{h_2}{2 \sin \theta_2}$$  \hspace{1cm} (3.1)

Note that a negative value of gap ($g$) corresponds to an overlap ($q$).

$$e = \left( \frac{h_1}{2 \sin \theta_1} + \frac{h_2}{2 \sin \theta_2} + g \right) \frac{\sin \theta_1 \sin \theta_2}{\sin (\theta_1 + \theta_2)} - \frac{h_0}{2}$$  \hspace{1cm} (3.2)

Note that $g$ above will be negative for an overlap.

These equations also apply for panel points which have a stiffening plate on the surface of the chord, in which case the term $h_0/2$ is replaced by $(h_0/2 + t_p)$, where $t_p$ is the stiffening plate thickness.

\textsuperscript{*} LRFD = Load resistance factor design

22
3.1 K and N connections

The majority of RHS truss connections have one compression bracing member and one tension bracing member welded to the chord as shown in Fig. 5. The Warren arrangement is commonly referred to as a K connection and the Pratt as an N connection. The latter is basically a particular case of the former; both can be either gap type or overlap type connections.

Experimental research (for example by Wardenier and Stark [18]) on RHS welded truss connections has shown that different failure modes can exist depending on the type of connection, loading conditions, and various geometric parameters. Failure modes have been described [18] for RHS as illustrated in Fig. 6.

Fig. 6 - Failure modes for K and N-type RHS truss connection

Mode A: Plastic failure of the chord face (one bracing member pushing the face in, and the other pulling it out).
Mode B: Punching shear of the chord face around a bracing member (either compression or tension).
Mode C: Rupture of the tension member or its welding.
Mode D: Local buckling of the compression bracing member.
Mode E: Shear failure of the chord member in the gap.
Mode F: Chord wall bearing or local buckling under the compression bracing member.
Mode G: Local buckling of the chord face behind the heel of the tension bracing member.
Table 2 – Factored resistance of axially loaded welded connections between square or circular bracing members and a square chord section

<table>
<thead>
<tr>
<th>type of connection</th>
<th>factored connection resistance ( i = 1, 2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>T-, Y- and X-connections</td>
<td>( \beta = 0.85 ) basis: chord face yielding</td>
</tr>
<tr>
<td></td>
<td>( N^* = \frac{f_{y, c} \cdot \Sigma \delta}{(1 - \beta) \sin \theta} \cdot \left[ \frac{2 \beta}{\sin \theta} + 4 (1 - \beta)^{0.5} \right] \cdot f(n) )</td>
</tr>
<tr>
<td>K- and N-gap connections</td>
<td>( \beta = 1.0 ) basis: chord face plastification</td>
</tr>
<tr>
<td></td>
<td>( N^* = 8.9 \frac{f_{y, c} \cdot \Sigma \delta}{\sin \theta} \cdot \left[ \frac{b_1 + b_2}{2b_0} \right] \cdot \phi_{1.0} \cdot f(n) )</td>
</tr>
<tr>
<td>K- and N-overlap connections(^1)</td>
<td>( 25% \leq O_n &lt; 50% ) basis: effective width</td>
</tr>
<tr>
<td></td>
<td>( N^* = f_y \cdot t \left( \frac{O_n}{55} \right) (2h_n - 4t) + b_2 + b_{\text{prod}} )</td>
</tr>
<tr>
<td></td>
<td>( 50% \leq O_n &lt; 80% ) basis: effective width</td>
</tr>
<tr>
<td></td>
<td>( N^* = f_y \cdot t \left[ 2h_n - 4t + b_2 + b_{\text{prod}} \right] )</td>
</tr>
<tr>
<td></td>
<td>( O_n \geq 80% ) basis: effective width</td>
</tr>
<tr>
<td></td>
<td>( N^* = f_y \cdot t \left[ 2h_n - 4t + b_2 + b_{\text{prod}} \right] )</td>
</tr>
<tr>
<td>circular bracings</td>
<td>multiply formulae by ( \pi/4 ) and replace ( b_{\text{prod}} ) and ( h_{\text{prod}} ) by ( d_{\text{prod}} )</td>
</tr>
</tbody>
</table>

functions

\( f(n) = 1.0 \) for \( n \geq 0 \) (tension)

\( f(n) = 1.3 + 0.4 \frac{b_2}{b_0} \cdot n \) for \( n < 0 \) (compression)

but \( \leq 1.0 \)

\( b_y = \frac{10}{b_2} \cdot \frac{f_{y, c}}{f_y} \cdot t \) \( \leq b_2 \)

\( b_{\text{prod}} = \frac{10}{b_2} \cdot \frac{f_{y, c}}{f_y} \cdot t \) \( \leq b_2 \)

Note 1: Effective width computations need only be done for the overlapping bracing member. However, the effective (the factored connection resistance divided by the full yield capacity of the bracing member), of the overlapped bracing member is not to be taken higher than that of the overlapping bracing member.
### Table 2a – Range of validity of Table 2

<table>
<thead>
<tr>
<th>type of connection</th>
<th>connection parameters ($i = 1 \text{ or } 2$, $j = \text{overlapped bracing}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$b_i/b_0$</td>
</tr>
<tr>
<td>T, Y, X</td>
<td>0.25 ≤ $b_i/b_0$ ≤ 0.85&lt;sup&gt;4&lt;/sup&gt;</td>
</tr>
<tr>
<td>K, N gap</td>
<td>≥ 0.1 + 0.01 $\frac{b_0}{l_0}$</td>
</tr>
<tr>
<td></td>
<td>$\beta ≥ 0.35$</td>
</tr>
<tr>
<td>K, N overlap</td>
<td>≥ 0.25</td>
</tr>
<tr>
<td>for circular bracings (web members)</td>
<td>0.4 ≤ $\frac{d_i}{b_0} ≤ 0.8$</td>
</tr>
</tbody>
</table>

Note: <sup>1</sup> Outside this range of validity other failure criteria may be governing, e.g., punching shear, effective width, side wall failure, chord shear or local buckling. If these particular limits of validity are violated the connection may still be checked as one having a rectangular chord using Table 3, provided the limits of validity in Table 3a are still met.

<sup>2</sup> $f_{y1}$, $f_{y2} ≤ 355 \text{ N/mm}^2$, $f_{y2}$ (or $f_{y1}$)/$f_{y2} ≤ 0.8$

<sup>3</sup> If $\frac{b_i}{b_0} >$ the larger of 1.5 (1 − $\beta$) and $(l_i + l_j)$, treat as a T or Y connection.
Table 3 – Factored resistance of axially loaded welded connections between rectangular, square or circular bracing members and a rectangular chord section

<table>
<thead>
<tr>
<th>Type of connection</th>
<th>Factored connection resistance (l = 1.2)</th>
<th>Basis: chord face yielding</th>
</tr>
</thead>
<tbody>
<tr>
<td>T-, Y- and X-connections</td>
<td>$N_t^* = \frac{f_{y,t} \cdot t_b \cdot L}{(1-\beta) \sin \theta_t}$</td>
<td>$\beta = 0.85$</td>
</tr>
<tr>
<td></td>
<td>$+ \frac{2n}{\sin \theta_t} + 4(1-\beta)^{0.5} \cdot f(n)$</td>
<td>$\beta = 1.0$</td>
</tr>
<tr>
<td></td>
<td>chord side wall failure$^1$</td>
<td>For $0.85 &lt; \beta \leq 1.0$</td>
</tr>
<tr>
<td></td>
<td>$N_t^* = \frac{f_{y,t} \cdot t_b}{\sin \theta_t} \left[ \frac{2h_t}{\sin \theta_r} + 10 t_b \right]$</td>
<td>Use linear interpolation of chord face yielding and chord side wall criteria</td>
</tr>
<tr>
<td></td>
<td>$\beta &gt; 0.85$</td>
<td>Basis: effective width</td>
</tr>
<tr>
<td></td>
<td>$N_t^* = f_{y,t} \cdot t_b \left[ 2h_t - 4t_b + 2b_c \right]$</td>
<td>$0.85 \leq \beta \leq 1/\gamma$</td>
</tr>
<tr>
<td></td>
<td>Basis: punching shear</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$N_t^* = \frac{f_{y,t} \cdot t_b}{\sqrt{3} \sin \theta_t} \left[ \frac{2h_t}{\sin \theta_r} + 2b_c \right]$</td>
<td>$K$- and N-gap connections</td>
</tr>
<tr>
<td></td>
<td>Basis: chord face yielding</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$N_t^* = 8.9 \cdot f_{y,t} \cdot t_b \cdot \left[ \frac{b_h + b_w + h_i + h_d}{4b_c} \right] \cdot \gamma^0.5 \cdot f(n)$</td>
<td>$(l = 1.2)$</td>
</tr>
<tr>
<td></td>
<td>Basis: chord shear</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$N_{t,\gamma,\text{open}} \leq (A_p - A_c) f_{y,t} + A_c f_{y,c} \left[ 1 - (V/V_p)^{0.5} \right]$</td>
<td>$N_t^* = f_{y,t} \cdot t_b \left[ 2h_t - 4t_b + b_c + b_d \right]$</td>
</tr>
<tr>
<td></td>
<td>Basis: effective width</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$K$- and N-overlap connections</td>
<td>$\beta \leq 1/\gamma$</td>
</tr>
<tr>
<td></td>
<td>Similar to connections of square hollow sections (Table 2)</td>
<td>Basis: punching shear</td>
</tr>
<tr>
<td></td>
<td>Circular bracings</td>
<td>$N_t^* = \frac{f_{y,t} \cdot t_b}{\sqrt{3} \sin \theta_t} \left[ \frac{2h_t}{\sin \theta_r} + b_c + b_w \right]$</td>
</tr>
<tr>
<td></td>
<td>Multiple formulae by $\gamma/4$ and replace $b_{d_y}$ and $h_{d_y}$ by $d_{d_y}$</td>
<td></td>
</tr>
</tbody>
</table>

Functions

<table>
<thead>
<tr>
<th>Function</th>
<th>Expression</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f(n)$</td>
<td>$f(n) = \frac{f_{y,t} \cdot A_c}{\sqrt{3}} \cdot \alpha = \frac{1}{\left( 1 + \frac{4g^2}{36} \right)^{0.5}}$</td>
</tr>
<tr>
<td>For square and rectangular bracings, $A_c = (2h_t + \alpha \cdot b_h) \cdot t_b$</td>
<td></td>
</tr>
<tr>
<td>For circular bracings, $A_c = 2h_t \cdot t_b$</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Condition</th>
<th>Expression</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension: $f_t = f_{y,t}$</td>
<td>$f(n) = 1.0$ for $n \geq 0$</td>
</tr>
<tr>
<td>Compression: $f_c = f_{y,c}$</td>
<td>$f(n) = 1.3 + 0.4 \cdot n$ for $n &lt; 0$</td>
</tr>
<tr>
<td>$\alpha = 1.0$ for $\beta \leq 1.0$</td>
<td></td>
</tr>
<tr>
<td>$b_{y,\text{weld}} = \frac{10}{b_d/b_c} \cdot b_t^{0.5}$</td>
<td>$b_{y,\text{weld}} = \frac{10}{b_d/b_c} \cdot b_t^{0.5}$</td>
</tr>
<tr>
<td>$\leq b_t$</td>
<td>$\leq b_t$</td>
</tr>
<tr>
<td>$b_{w,\text{weld}} = \frac{10}{b_d/b_c} \cdot b_t$</td>
<td>$\leq b_t$</td>
</tr>
</tbody>
</table>

Note$^1$: For X-connections with angles $\theta < 90^\circ$, the chord side walls must be checked for shear. 26
<table>
<thead>
<tr>
<th>type of connection</th>
<th>connection parameters ($i = 1$ or $2$, $j = \text{overlapped bracing}$)</th>
<th>eccentricity</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$b_i/b_{i0}$ $h_i/h_{i0}$ $b_{i0}/b_{i0}$ $h_{i0}/h_{i0}$</td>
<td>$b_i/b_{i0}$ $h_i/h_{i0}$ $b_{i0}/b_{i0}$ $h_{i0}/h_{i0}$</td>
</tr>
<tr>
<td>T, Y, X</td>
<td>$\geq 0.25$ $\leq 1.25 \sqrt{\frac{E}{f_{y1}}}$ $\leq 35$ $\leq \frac{h_i}{b_i} \leq 2$</td>
<td>$\leq 35$</td>
</tr>
<tr>
<td>K, N gap</td>
<td>$\geq 0.1 + 0.01 \frac{b_{i0}}{c_0}$ $\beta \geq 0.35$ $\leq 35$ $\leq 0.5 \leq \frac{h_i}{b_i} \leq 2$</td>
<td>$\leq 35$</td>
</tr>
<tr>
<td>K, N overlap</td>
<td>$\geq 0.25$ $\leq 1.1 \sqrt{\frac{E}{f_{y1}}}$ $\leq 40$</td>
<td>$\leq 40$</td>
</tr>
<tr>
<td>for circular</td>
<td>$0.4 \leq \frac{d_i}{b_{i0}} \leq 0.8$ $\leq 1.5 \sqrt{\frac{E}{f_{y1}}}$ $\leq 50$</td>
<td>$\leq 50$</td>
</tr>
</tbody>
</table>

Note:  
1. $f_{y1}$, $f_{y2} = 355$ N/mm$^2$, $f_{y1}$ (or $f_{y2}$)/$f_{y2} \leq 0.8$  
2. If $\frac{b_i}{b_{i0}} >$ the larger of $1.5 (1 - \beta)$ and $(t_i + t_j)$, treat as a T or Y connection
Failure in test specimens has also been observed to be a combination of more than one failure mode. It should be noted here that Modes C and D are generally combined together under the term "effective width failures", and are treated identically since the connection resistance in both cases is determined by the effective cross section of the critical bracing member, with some bracing member walls possibly being only partially effective.

Plastic failure of the chord face (Mode A) is the most common failure mode for gap connections with small to medium ratios of the bracing member widths to the chord width $\beta$. For medium width ratios ($\beta = 0.6$ to $0.8$), this mode generally occurs together with tearing in the chord (Mode B) or the tension bracing member (Mode C) although the latter only occurs in connections with relatively thin walled bracing members. Mode D, involving local buckling of the compression bracing member, is the most common failure mode for overlap connections. Shear failure of the entire chord section (Mode E) is observed in gap connections where the width (or diameter) of bracing members is close to that of the chord ($\beta = 1.0$), or where $h_2 < b_2$. Local buckling failure (Modes F and G) occurs occasionally in RHS connections with high chord width (or depth) to thickness ratios ($b_2/h_2$ or $h_2/t_2$). It has been found that, in some cases, one or two governing modes can be used to predict connection resistance. For example, Mode G failure, is excluded from the subsequent design expressions by restrictions on the range permitted for geometric parameters.

Various formulae exist for the connection failure modes described above. Some have been derived theoretically, while others are primarily empirical. The general criterion for design is ultimate resistance, but the recommendations presented herein, and their limits of validity, have been set so that a limit state for deformation is not exceeded by specified (service) loads.

Limit states design recommendations are summarized in Tables 2 and 2a (for square chords), and Tables 3 and 3a (for rectangular chords). A number of observations for K and N connections can be made from an examination of these tables:

A common design criterion for all K and N gap connections is Mode A, plastic failure of the chord face. The constants in the resistance equations are derived from extensive experimental data, and the other terms reflect ultimate strength parameters such as plastic moment capacity of the chord face per unit length ($I_p E/4$), bracing to chord width ratio, $\beta$, chord wall slenderness $\gamma$, and the term $f(n)$ which accounts for the influence of compression chord longitudinal stresses.

Tables 2 and 3 show that the resistance of a gap K or N connection with an RHS chord is largely independent of the gap size (no gap size parameter). Table 2 which is restricted to square RHS chords was derived from the more general Table 3 and uses more confined geometric parameters. The result is that gap K and N connections with square RHS need only be examined for failure Mode A, whereas those with rectangular

![Fig. 7 – Shear area ($A_s$) of the chord in the gap region of an RHS K or N-connection](image-url)
RHS must be considered for failure Modes B, C or D, and E as well. This approach has allowed the creation of helpful graphical design charts which are presented later for connections between square RHS.

In Table 3, the Mode E check for chord shear in the gap of K and N connections involves dividing the chord cross section into two portions. The first is a shear area, $A_s$, comprising the side walls plus part of the top flange, shown in Fig. 7, which can carry both shear and axial loads interactively. The second is the remaining area, $A_t - A_s$, which is effective in carrying axial forces but not shear.

Tables 2 and 3 present a range of resistances based on the concept of effective width for square and rectangular RHS overlap connections, starting with 25% overlap, which is the minimum to ensure overlap behaviour. The resistance increases linearly with overlap from 25% to 50%, is constant from 50% to 80%, then is constant above 80% at a higher level. Fig. 8 illustrates the physical interpretation of the expressions for effective width given in the tables.

![Fig. 8 - Physical interpretation of effective width terms](image)

### 3.2 T, Y and X connections

In the same way that an N connection can be considered a particular case of the general K connection, the T connection is a particular case of the Y connection. The basic difference between the two types is that the component of load normal to the chord in T and Y connections is resisted by shear and bending in the chord, whereas with K or N connections the normal component from one bracing member is balanced primarily by the similar component in the other.

The limit states design recommendations for T, Y and X connections are summarized in Tables 2 and 2a (for square chords), and Tables 3 and 3a (for rectangular chords). As with K and N connections, various observations can be made from the Tables:

Resistance equations in Tables 2 and 3, for $\beta \leq 0.85$, are based on a yield line mechanism in the RHS chord face. By limiting connection design capacity under factored loads to the connection yield load, one ensures that deformations will be acceptable at specified (service) load levels.

For full width ($\beta = 1.0$) RHS T, Y and X connections, flexibility is no longer a problem, and resistance is based on either the tension capacity or the compression instability of the chord side walls, for tension and compression bracing members respectively.

Compression loaded, full-width X connections for RHS are differentiated from T or Y connections as their side walls exhibit greater deformation than T connections. Accordingly, the value of $f_s$ in the resistance equation is reduced to $0.8 \sin \theta$, of the value which is used for T or Y situations. In both instances, a linear progression is followed for resistances from values...
for $\beta = 0.85$ (where flexure of the chord face governs) to values for $\beta = 1.0$ (where chord side wall failure governs).

All RHS T, Y and X connections with high bracing width to chord width ratios ($\beta \geq 0.85$) are also checked for the "effective width" failure modes and for punching shear of the chord face. For this range of width ratios, the bracing member loads are largely carried by their side walls parallel to the chord while the walls transverse to the chords carry relatively little. The upper limit of $\beta = 1 - 1/\gamma$ for checking punching shear is determined by the physical possibility of such a failure, when one considers that the shear has to be between the outer limits of the bracing width and the inner face of the chord wall.

3.3 KT connections

As shown in Fig. 9, KT connections occur in some trusses, and the strength of gap connections can be related to K and N connections by replacing $(b_1 + b_2)/2b_0$ in Table 2 with $(b_1 + b_2 + b_3)/3b_0$, and by replacing $(b_1 + b_2 + h_1 + h_2)/4b_0$ in Table 3 with $(b_1 + b_2 + b_3 + h_1 + h_2 + h_3)/6b_0$. In gap KT connections the gap should be taken as the largest gap between two bracing members having significant forces acting in the opposite sense.

In the case of KT connections with gap, the force components, normal to the chord, of the two members acting in the same sense are added together to represent the load. The connection resistance component, normal to the chord, of the remaining diagonal is then required to exceed that load. For the examples shown in Fig. 9,

$$N_2 \sin \theta_2 \geq N_1 \sin \theta_1 + N_3 \sin \theta_3 \quad \text{(Fig. 9 a)} \quad \text{(3.3)}$$
$$N_1^* \sin \theta_1 \geq N_2 \sin \theta_2 + N_3 \sin \theta_3 \quad \text{(Fig. 9 b)} \quad \text{(3.4)}$$

where $N_1^*$ is calculated from Tables 2 or 3 in which $N_2^* \sin \theta_2 = N_1^* \sin \theta_1$. 

30
When there is a cross-chord load (for instance from a purlin or hanger), which acts in the same direction as the load components that were combined above, the connection resistance of the remaining diagonal needs to be examined directly. For the examples shown in Fig. 9,

\[
\begin{align*}
N_2^* \sin \theta_2 & \geq N_2 \sin \theta_2 \quad \text{(Fig. 9c)} \tag{3.5} \\
N_1^* \sin \theta_1 & \geq N_1 \sin \theta_1 \quad \text{(Fig. 9d)} \tag{3.6}
\end{align*}
\]

again, where \( N_i^* \) is calculated from Tables 2 or 3 in which \( N_2^* \sin \theta_2 = N_1^* \sin \theta_1 \).

Fig. 9 – Four KT connections

Technology centre
One should also remember that for load cases such as shown in Figs. 9c and 9d, the purlin or hanger connection to the chord may require an additional check. If the vertical bracing member in the gap KT connection shown in Fig. 9 had no force in it, the gap should be taken as the distance between the toes of members 1 and 2, and the connection treated as a K connection with $\beta = (b_1 + b_2 + h_1 + h_2)/4b_2$.

For overlap KT connections, which are actually more likely to occur, the resistance of an RHS connection can be determined by checking each overlapping bracing member and ensuring that $N^*$ (from Table 2) $\geq N_0$. (Overlapped bracing members would also have a restriction on their connection efficiency as noted at the bottom of Table 2.) For the bracing member effective width terms, care should be taken to ensure that the member sequence of overlapping is properly accounted for.

Tests have shown that the resistance of K and N connections in the presence of cross-chord loading (i.e., X connection loading) is similar to that given by the resistance formulæ for K and N connections, and the same is true for KT connections. If all the bracing forces on one side of a connection act in the same sense, or if only one bracing member is carrying load, the connection should be checked as an X connection using an equivalent bracing member size.

### 3.4 Graphical design charts

Fig. 10 allows a quick evaluation of whether a K or N gap connection can be configured from proposed square bracing sections on a rectangular or square chord section. The chart indicates the maximum value of $\beta$ (average width of bracing members relative to the chord width) which can be accommodated while remaining within the allowable bounds of nodal eccentricity. Input parameters are chord to bracing angles $\theta$ and chord aspect ratio $h_0/b_0$.

Reusink and Wardner [20] have produced a set of consistent design charts for the preliminary design of K, N, T, Y, and X connections which are based on IIM [2] and therefore Eurocode 3 [21] recommendations. Figs. 11 to 18 are for square HSS and they relate to formulæ in Table 2.

Fig. 10 - Allowable range of bracing member width ratios for RHS gap connections, based on the allowable eccentricity limits
T, Y and X connections with bracings in tension: calculation example

Connection and symbols:

\[
\beta = \frac{b_2}{b_1}, \quad \beta = \frac{b_1}{b_0}, \quad n = \frac{f_{tp}}{f_{tp}}, \quad N_{TP} = \frac{N_{TP}}{f_{tp}}
\]

Ranges of validity:

- \(0.25 \leq \beta \leq 1.0\)
- \(b_2/b_1 \leq 35\)
- \(b_1/t_1 \leq 35\) (for tension bracing)
- \(f_{tp} \leq 355\) N/mm²
- \(30^\circ \leq \delta_i \leq 90^\circ\)

Assume a 45° X connection with these members (ISO sizes):
- Chord: 200 × 200 × 8.0 (\(A_n = 6050\) mm²)
- Bracings: 100 × 100 × 5.0 (\(A_r = 1890\) mm²)
- \(f_{tp} = f_{tp} = 355\) N/mm²
- \(\delta_i = 45^\circ\) and \(\sin \delta_i = 0.707\)
- \(n = 0.48\)

- \(\beta = 100/200 = 0.5\)
- \(f_{tp} = 0.92\) from Fig. 19
- \(b_2/t_2 = 20\)
- \(b_1/t_1 = 25\)

\[
N_{TP} = 0.16 \left(\frac{8.0}{5.0}\right) \left(\frac{1}{0.707}\right) (0.92) = 0.33
\]

\[
N_{TP} = 0.33 (1890)(0.355) = 224\) kN

Fig. 11 – Bracing efficiency for square hollow section T, Y and X connections with a tension-loaded bracing
Efficiency coefficient $C_T$, compr.

$$\frac{N_t}{A_t \cdot f_y} = C_T \cdot c \cdot \frac{f_y}{f_y \cdot t_y} \cdot \frac{1}{\sin \beta} \cdot f(n)$$

$b_y/t_y$

0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1.0

---

<table>
<thead>
<tr>
<th>T and Y connections with bracings in compression: calculation example</th>
</tr>
</thead>
<tbody>
<tr>
<td>connection and symbols</td>
</tr>
<tr>
<td>( \beta ) = ( \frac{b_y}{t_y} )</td>
</tr>
<tr>
<td>( N_t ) = ( N_{y_d} )</td>
</tr>
<tr>
<td>( f(n) ) = 1.0 from Fig. 19</td>
</tr>
<tr>
<td>( b_y/t_y ) = 20 &lt; 30.4</td>
</tr>
<tr>
<td>( b_y/t_y ) = 12.5</td>
</tr>
<tr>
<td>ranges of validity</td>
</tr>
<tr>
<td>0.25 ( \leq \beta ) ( \leq 1.0 )</td>
</tr>
<tr>
<td>0.25 ( \leq \beta ) ( \leq 1.0 )</td>
</tr>
<tr>
<td>( b_y/t_y ) ( \leq 35 )</td>
</tr>
<tr>
<td>( b_y/t_y ) ( \leq 1.25 \sqrt{E/f_y} \leq 35 ) (for compression bracing)</td>
</tr>
<tr>
<td>( f_y ) ( \leq 355 \text{ N/mm}^2 )</td>
</tr>
<tr>
<td>( 30^\circ \leq \beta \leq 90^\circ )</td>
</tr>
</tbody>
</table>

Assume a T connection with these members (ISO sizes):
| chord: \( 100 \times 100 \times 8.0 \,(A_c = 2910 \text{ mm}^2) \) |
| bracings: \( 100 \times 100 \times 5.0 \,(A_y = 1890 \text{ mm}^2) \) |
| \( f_y = f_{y_d} = 355 \text{ N/mm}^2 \) |
| \( \beta = 90^\circ \) and \( \sin \beta = 1.0 \) |
| \( n = -0.60 \) |

\[ \beta = \frac{b_y}{t_y} = 100/100 = 1.0 \]

\[ f(n) = 1.0 \text{ from Fig. 19} \]

\[ b_y/t_y = 20 < 30.4 \]

\[ b_y/t_y = 12.5 \]

\[ N_{y_d} = 0.68 \left( \frac{8.0}{5.0} \right) \left( 1.0 \right) \left( 1.0 \right) \]

\[ = 1.09 \text{ take as 1.0} \]

\[ N_{y_d} = 1.0 \times 1890 \times 0.355 = 671 \text{ kN} \]

Note: This resistance is subject to a further bracing "effective width" check, shown in Fig. 14.

Fig. 12 – Bracing efficiency for square hollow section T and Y connections with a compression loaded bracing

34
Efficiency coefficient \( C_X \) compr.

\[
\begin{align*}
\beta < 0.85 & : \quad \frac{N_1}{A_1 \cdot t_1} = C_X \cdot \frac{f_{p0} \cdot t_0}{f_{y1} \cdot t_1} \cdot \frac{1}{\sin \theta_1} \cdot f(n) \\
\beta = 1.0 & : \quad \frac{N_1}{A_1 \cdot t_1} = C_X \cdot \frac{f_{p0} \cdot t_0}{f_{y1} \cdot t_1} \cdot f(n) \\
0.85 < \beta < 1.0 & : \text{linear interpolation}
\end{align*}
\]

X connections with bracings in compression: calculation example

Assume a 30° X connection with these members (ISO sizes):
- Chord: 150 x 150 x 10.0 (\( A_c = 5450 \text{ mm}^2 \))
- Bracings: 140 x 140 x 8.0 (\( A_b = 4130 \text{ mm}^2 \))
- \( f_p = f_y = 355 \text{ N/mm}^2 \)
- \( \theta_1 = 30^\circ \) and \( \sin \theta_1 = 0.5 \)
- \( n = +0.38 \)

- \( \beta = \frac{140}{150} = 0.93 \)
- \( \beta > 1 - 1/t_1 \), so punching shear check not necessary
- \( f(n) = 1.0 \), as chord is in tension

\[
\begin{align*}
\frac{b_1}{t_1} & = 17.5 < 30.4 \\
\frac{b_2}{t_0} & = 15 \\
\frac{N_1}{A_1 \cdot t_1} \cdot \beta & = 0.43 \left( \frac{10.0}{8.0} \right) \left( \frac{1}{2.5} \right) (1.0) = 1.08 \\
\frac{N_2}{A_2 \cdot t_2} \cdot \beta & = 0.49 \left( \frac{10.0}{8.0} \right) (1.0) = 0.61 \\
\end{align*}
\]

- interpolating linearly, for \( \beta = 0.93 \), \( \frac{N_2}{A_2 \cdot t_2} \) = 0.79

Note: This efficiency is still subject to further checks for:
- (I) bracing "effective width" (Fig 14)
- (II) chord shear (Table 3)

Fig. 13 - Bracing efficiency for square hollow section X connections with a compression loaded bracing
Effective width check for T, Y and X connections: calculation example

The T connection from Fig. 12 will be checked for bracing "effective width":
- chord: $100 \times 100 \times 8.0 (A_c = 2910 \text{ mm}^2)$
- bracings: $100 \times 100 \times 5.0 (A_i = 1890 \text{ mm}^2)$
- $f_y = f_{y1} = 355 \text{ N/mm}^2$
- $\theta_i = 90^\circ$ and $\sin \theta_i = 1.0$

- $\beta = \frac{100}{100} = 1.0$
- $b_i/t_i = 20 < 30.4$
- $b_i/t_i = 12.5$
- $t_y/t_i = 1.6$

- $N_i^* = \frac{1.00}{A_i f_{y1}}$

This is also the bracing efficiency obtained in Fig. 12 for the chord side wall failure mode.

Ranges of validity:

- $0.25 \leq \beta \leq 1.0$
- $b_i/t_i \leq 35$
- $b_i/t_i \leq 35$ (for tension bracing)
- $b_i/t_i \leq 1.25 \sqrt{E/\sigma_y} \leq 35$
- $f_y \leq 355 \text{ N/mm}^2$
- $30^\circ \leq \theta_i \leq 90^\circ$

Fig. 14 - Total efficiency for effective width for T, Y and X connections with $\beta > 0.85$
Assume a 45° K connection with these members (ISO sizes) and $f_p = 355$ N/mm²:
- chord: 200 $\times$ 200 $\times$ 10.0 ($A_g = 7450$ mm²)
- compression bracing: 150 $\times$ 150 $\times$ 8.0 ($A_g = 4450$ mm²)
- tension bracing: 140 $\times$ 140 $\times$ 8.0 ($A_g = 4130$ mm²)

Assume $n = -0.8$.

From Fig. 10, a gap connection is feasible providing
- $\beta \leq 1.0$
- eccentricity ($e$) = $0.20h_2 = 40$
- $g = \frac{(40 + 100)(\sin 90^\circ)}{\sin 45^\circ \sin 45^\circ} = \frac{160}{2\sin 45^\circ} = 140$ (Eqn. 3.1)
- $\beta = \frac{(140 + 150)(2 \times 200)}{2} = 0.725$
- $0.5(1-\beta) = 27.5 < g = 82.5 = 1.5(1-\beta)$
- $b_2 = 140 \times 0.77(140 + 150)/2 = 112$
- $b_2/b_2 = 20$, $b_2/b_2 = 17.5$
- $b_1/b_1 = 35.4$ or $b_2/2 = 0.7 > 0.3$ and $> 0.35$ $n = -0.8$, so $f(n) = 0.86$ from Fig. 19.

From the graph, $A_{K/g} = 0.35$ or $A_{N/g} = 0.60$

Note: If $t_1$ and $t_2$ had been lower, the efficiency could have been higher.

Fig. 15 - Bracing member efficiency for square hollow section K and N gap connections
The concept of "connection efficiency", defined as the connection factored resistance divided by the full section yield load of the particular bracing member, is employed for these charts. That is, connection efficiency $= N/A(f, f_{r})$. However, the efficiencies given by the charts for all but the overlapped K connections - and the T, Y, X connection effective width checks - are termed $C_0$, $C_r$, or $C_y$ depending on the type of connection. These latter efficiencies need to be multiplied by three factors to obtain the final connection efficiency in each case.

The first factor, correcting for differing strengths between the chord and the bracing member, is $(f/f_{r})(t_0/t)$. This generally reduces to $t_0/t$, because the same grade of steel would normally be used throughout.

The second factor, adjusting for the angle between the bracing member and the chord, is $1/\sin \beta$ for square RHS T, X and gap K connections. One should note that such an angle function does not exist for square RHS overlap connections.

The third factor, correcting for the influence of compression chord longitudinal stresses on the connection efficiency, is $f(n)$ for RHS. $f(n)$ is defined in Table 2, Table 3, the List of Symbols and is plotted in Fig. 19. The function $f(n)$ is 1.0 for square RHS overlap connections. It should be noted that the chord stress ratio, $n$, is calculated using the maximum compression stress in the chord member, taking into account axial force and (where applicable) bending. Although this reduction factor was originally determined from K connection tests, it has been shown to be applicable to T connections [23] and also where the chord is subjected to high bending moment and no axial force [23].

In the design charts a lower bound of the various failure modes was generally drawn to result in $C_{x}$ functions which depend only on the type of connection. Simplifying, conservative assumptions, and narrower validity parameter ranges were sometimes necessary in the process. Still, use of the charts produces results close to those given by the formulae in Table 2.

Four charts are presented for T, Y and X connections (Figs. 11 to 14). The first applies to all three types of connection when they are loaded in tension; the second applies to T and Y connections, when loaded in compression; the third to X connections, when loaded in compression; and the fourth is an effective width failure mode check, necessary only when $\beta$ exceeds 0.85. The first three charts are identical for $\beta$ values up to 0.85. When $\beta$ exceeds 0.85 the behaviour of the chord side walls is different for the three situations, and three charts become necessary. [1] They show linear interpolations between known resistance values at $\beta = 0.85$ and $\beta = 1.0$. For gap K and N connections the efficiency is given by Fig. 15.

The range of overlap (defined graphically in Fig. 16) for square K and N connections is from 50% to 100% rather than from 25% as in Table 2. This avoids the more complex lower range where resistance varies constantly with amount of overlap. In the 80% to 100% overlap chart,

---

*Fig. 16 - Definition of overlap (O) in Figs. 17 and 18*

1) European buckling curve "a" is implicit in these charts [87]. However, the net difference is small if the curve from another code is used.
K and N overlap connections with 50% ≤ Qi < 80%: calculation example

Assume a 45° K connection with these members
ISO sizes) and \( t_{f} = 355 \text{ N/mm}^2 \)
- chord: 200 × 200 × 10.0 \( (A_{c} = 7450 \text{ mm}^2) \)
- compression bracing: 150 × 150 × 8.0 \( (A_{c} = 4450 \text{ mm}^2) \)
  (overlapping member = i)
- tension bracing: 140 × 140 × 8.0 \( (A_{c} = 4130 \text{ mm}^2) \)
  (overlapping member = i)
- eccentricity \( e = 0.30l_{0} = 60 \)

\[
g = \frac{(-60 + 100) \sin 90^\circ}{\sin 45^\circ \sin 45^\circ} - \frac{150}{2 \sin 45^\circ} - \frac{140}{2 \sin 45^\circ} \quad \text{(Eqn. 3.1)}
\]

\[
= \frac{-125}{2 \sin 45^\circ} \quad \text{(Eqn. 3.1)}
\]

\[
Q_{i} = \frac{q \times 100}{p} = 125 \times 100 = 63% \quad \text{(Eqn. 3.1)}
\]

\[
b_{i}/b_{i} = \frac{140}{150} = 0.93 \quad \text{(Eqn. 3.1)}
\]

\[
l_{i}/t_{i} = 8.0/8.0 = 1.0 \quad \text{(Eqn. 3.1)}
\]

\[
b_{i}/b_{0} = 20, \quad b_{i}/t_{i} = 17.5, \quad b_{i}/b_{i} = 18.8 \leq 30.4 \quad \text{(Eqn. 3.1)}
\]

\[
b_{i}/b_{0} = 0.75, \quad b_{i}/t_{i} = 0.70 \quad \text{(Eqn. 3.1)}
\]

\[
\frac{N_{i}^{*}}{A_{c}} = 0.38 + 0.36 = 0.74 \quad \text{(Eqn. 3.1)}
\]

\[
\frac{N_{i}^{*}}{A_{c}} \leq 0.74 \quad \text{(see Table 2)}
\]

Fig. 17 - Bracing member efficiency for square hollow section K and N overlap connections having 50% ≤ Qi < 80%
### K and N overlap connections with 80% ≤ \( Q_e \) < 100%: calculation example

**Connection and symbols**

![Connection Diagram](image)

**Ranges of validity**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>( b / t_0 )</td>
<td>≥ 0.25</td>
</tr>
<tr>
<td>( b / t_1 )</td>
<td>≤ ( 1.1 \sqrt{E / t_1} )</td>
</tr>
<tr>
<td>( b / r_2 )</td>
<td>≤ 40</td>
</tr>
<tr>
<td>( b / r_2 )</td>
<td>≤ 1.0</td>
</tr>
<tr>
<td>( b / t_0 )</td>
<td>≥ 0.75</td>
</tr>
<tr>
<td>( 80% \ ≤ \ Q_e \ &lt; 100% )</td>
<td></td>
</tr>
<tr>
<td>( -0.55 \ ≤ \ \epsilon / t_0 \ ≤ 0.25 )</td>
<td></td>
</tr>
<tr>
<td>( f_y )</td>
<td>≤ 355 N/mm²</td>
</tr>
<tr>
<td>( 30° \ ≤ \ \theta \ ≤ 90° )</td>
<td></td>
</tr>
</tbody>
</table>

Assume a 45° K connection with these members (ISO sizes) and \( f_y = 355 \) N/mm²:

- Chord: 200 × 200 × 10.0 (\( A_p = 7450 \) mm²)
- Compression bracing: 150 × 150 × 8.0 (\( A_p = 4450 \) mm²) (overlapping member = I)
- Tension bracing: 140 × 140 × 8.0 (\( A_p = 4130 \) mm²) (overlapping member = I)

**Eccentricity**

\[
\epsilon = -0.40 / t_0 = -80
\]

**Equation**: \( q = \frac{-80 + 100 \times \sin 90°}{\sin 45° \times \sin 45°} - \frac{150}{2 \sin 45°} - \frac{140}{2 \sin 45°} = 150 \times 100 \times 100 / 140 \sin 45° = 83\% \)

- **\( b / t_0 \)**: 140 / 150 = 0.93
- **\( l / t_1 \)**: 8.0 / 8.0 = 1.0
- **\( b / r_2 \)**: 20, **\( b / t_2 \)**: 17.5, **\( b / t_2 \)**: 18.8 ≤ 30.4
- **\( b / t_0 \)**: 0.75, **\( b / r_2 \)**: 0.70
- **\( N'_{tp} \)**
- **\( A_{tp} \)**: 0.87
- Hence, **\( N'_{tp} \)** ≤ 0.87 (see Table 2)

Fig. 18 - Bracing member efficiency for square hollow section K and N overlap connections having \( 80% \ ≤ \ Q_e \ < 100\% \)

40
Fig. 18, the subscript j applies to the overlapped member while subscript i refers to the overlapping member. Unlike circular hollow section overlapped connections, only the overlapped member need be checked for square RHS connections; however, there is a check on the connection efficiency for the overlapped bracing members, as noted in Table 2. Fig. [17] for partially overlapped connections requires entering twice, once for the overlap on the chord (when 0 and i terms are used) and a second time for the overlap on the other bracing member (when j and i terms are used). The two part-efficiencies are then added together for the total efficiency. Validity ranges are the same as for Table 2 except that overlap starts at 50%. It can be seen from the charts that efficiencies of K and N overlap connections always exceed 0.8 for full overlap (Fig. [18]) and 0.6 for partial overlap (Fig. [17]). Therefore, fully overlapped connections are usually stronger than partially overlapped ones.

![Function f(n)](image)

Fig. 19 - Function f(n) which describes the influence of chord stress on the total efficiency of square hollow section T, Y, X, K and N gap connections.

### 3.5 Reinforced connections

Instances may occur when a truss connection has an inadequate resistance, and a designer needs to resort to some form of connection reinforcement. Such a situation might arise if RHS material were ordered on the basis of member selection only, without connection capacity checks being performed. Alternatively, only one or a few connections of a truss may be inadequate due to the selection of a particular chord member, and so just these critical connections could be reinforced. The labour costs associated with connection reinforcement are significant, and the resulting structure may lose its aesthetic appeal, but in many cases it may be an acceptable solution.

#### 3.5.1 With stiffening plates

The most common method of strengthening RHS connections is to weld a stiffening plate (or plates) to the RHS chord member. It is particularly applicable to gap K connections with rectangular chord members, although an unstiffened overlap connection is generally preferable from the viewpoints of economy and fatigue. However, a gap connection with a stiffening plate eliminates the necessity for double cuts on the bracing members, and in
certain cases may prove more acceptable to the fabricator. The addition of a flat plate welded to the connecting face of the chord member greatly reduces local deformations of the connection and consequently the overall truss deformations are reduced. It also permits a more uniform stress distribution in the bracing members.

3.5.1.1 K and N connections

The type of reinforcement required depends upon the governing failure mode which causes the inadequate connection capacity. Two types of plate reinforcement – in one case to the chord connecting face and in the other to the chord side walls – are shown in Fig. 20. Both of these would be applicable to connections with RHS chord members and either CHS or RHS bracing members. An alternative to stiffening a connection with plates is to insert a length of chord material of the required thickness at the connection, the length of which would be the same as \( L_p \) given below. (This is equivalent to the use of a “joint can” in offshore steel structures.)

The capacity of gap K connections is typically controlled by either the chord face plastification criterion or the chord shear criterion, as summarized in Tables 2 and 3. When chord face plastification controls, the connection capacity can be increased by using flange plate reinforcement as shown in Fig. 20(a). This will usually occur when \( \beta \leq 1.0 \) and the members are square. When chord shear controls, the connection capacity can be increased by reinforcing with a pair of side plates as shown in Fig. 20(b). This failure mode will usually govern when \( \beta = 1.0 \) and \( h_0 < b_0 \).

![Diagram of flange plate reinforcement and side plate reinforcement](image)

Fig. 20 – Pratt truss connection with plate stiffening
(a) Flange plate reinforcement
(b) Side plate reinforcement

The first design guidance available for K connections stiffened with a flange plate, as shown in Fig. 20(a), was given by Shinouda [24]. However, this method was based on an elastic deformation requirement of the connection plate under specified (service) loads. A more logical limit states approach which is recommended for calculating the necessary stiffening plate thickness for gap K connections is to use the connection resistance expressions in Table 2 (for square or circular bracing members to square chord members), and Table 3 (for rectangular members) by considering \( t_p \) as the chord face thickness and neglecting \( t_0 [19] \). Also, the plate yield stress should be used. It is suggested that proportioning of the stiffening plate be based on the principle of developing the capacity of the bracing members \( (A_1 t_1) \). Dutta and Würker [25] consider that this will be achieved providing \( t_p \geq 2 t_0 \). Careful attention should be paid to the stiffening plate-to-chord welds which should have a weld throat size at least equal to the wall thickness of the adjacent bracing member [25]. The stiffening plate should have a minimum length \( L_p \) (see Fig. 20(a)), such that:

\[
L_p \geq 1.5 \left( h_1 \frac{h_0}{\sin \theta_1} + \frac{h_2}{\sin \theta_2} \right)
\]  

(3.7)
A minimum gap between the bracing members, just sufficient to permit welding of the bracing members independently to the plate is suggested. All-round welding is generally required to connect the stiffening plate to the chord member, and in order to prevent corrosion on the two inner surfaces.

In order to avoid partial overlapping of one bracing member onto another in a K connection, fabricators may elect to weld each bracing member to a vertical stiffener as shown in Fig. 21 (a). Another variation on this concept is to use the reinforcement shown in Fig. 21 (b). For both of these connections, $b_t \geq 2t$, and $2t_s$ is recommended. Designers should note that the K connection shown in Fig. 21 (c) is not acceptable, as it does not develop the strength of an overlapped K connection where one bracing member is welded only to the chord face. Also, it is difficult to create and ensure an effective saddle weld between the two bracing members.

If the capacity of a gap K connection is inadequate and the chord shear criterion is the governing failure mode, then as mentioned before one should stiffen with side plate reinforcement, as shown in Fig. 20 (b). A recommended procedure in this case for calculating the necessary stiffening plate thickness is to use the chord shear resistance expression in Table 3, by calculating $A_t$ as $2h_t (b_t + t_s)$. The stiffening plates should again have a minimum length, $L_p$ (see Fig. 20 (b)), given by Eqn. 3.7 and have the same depth as the chord member.

Fig. 21 – Some acceptable and unacceptable, non-standard truss K-connections

3.5.1.2 T, Y and X connections

Under tension or compression bracing loading, the capacity of a T, Y or X connection is typically controlled by either chord face yielding or chord side wall failure, as summarized in Tables 2 and 3. When chord face yielding controls, the connection capacity can be increased by using flange plate reinforcement similar to the connection shown in Fig. 20 (a). This will usually occur when $\beta \leq 0.85$. When chord side wall failure controls, the connection capacity can be increased by reinforcing with a pair of side plates similar to the connection shown in Fig. 20 (b). This failure mode will usually govern when $\beta = 1.0$.

For T, Y or X connections stiffened with side plate reinforcement, a recommended procedure for calculating the necessary stiffening plate thickness is to use the chord side wall resistance expression in Table 3, by replacing $t_s$ with $(b_t + t_s)$ for the side walls. The stiffening plates should have a length $L_p$ (see Fig. 20 (b)), such that for T and Y connections:

$$L_p \geq 1.5 \left( h_t / \sin \theta \right)$$

(3.8)

For T, Y and X connections stiffened with a flange plate there is a difference in behaviour of the stiffening plate depending on the sense of the load in the bracing member. With a tension
load in the bracing, the plate tends to lift off the chord member and behave as a plate clamped (welded) along its four edges. The strength of the connection thereby depends only on the plate geometry and properties, and not on the chord connecting face. Thus, for tension bracing loading, if one applies yield line theory to the plate-reinforced T, Y or X connection with rectangular members, the connection factored resistance can be reasonably estimated by:

\[ N^* = \left[ \frac{t_p}{f_p} \left( 1 - \beta_b \sin \theta_i \right) \right] \left[ \left( \frac{b_p}{f_p} \sin \theta_i \right) + 4 \left( 1 - \beta_b \sin \theta_i \right) \right] \] (3.9)

where

- \( t_p \) = thickness of the stiffening plate,
- \( f_p \) = yield strength of the stiffening plate,
- \( \beta_b \) = width ratio of bracing member to plate = \( b_b/B_p \),
- \( b_p \) = bracing member depth to plate width ratio = \( h_i/B_p \), and
- \( B_p \) = width of plate.

The similarity between Eqn. 3.9 and the factored resistance of unstiffened T, Y and X connections, based on chord face yielding as given by Table 3, is clearly evident. In order to develop the yield line pattern in the stiffening plate implicit in Eqn 3.9, the length of the plate \( L_p \) should be at least:

\[ L_p \geq \left( \frac{h_i}{\sin \theta_i} \right) + \sqrt{\left( B_p - b_b \right) \left( B_p - b_b \right)} \] (3.10)

Also, the plate width \( B_p \) should be such that a good transfer of loading to the side walls is achieved; for example \( B_p = b_b \) (see Fig. 20).

For T, Y and X connections stiffened with a flange plate, and under compression bracing loading, the plate and connecting chord face can be expected to act integrally with each other. This type of connection has been studied by Korol et al.\[27\], also using yield line theory. Hence for \( \beta_b \leq 0.85 \) (a reasonable upper limit for application of yield line analysis also employed for unreinforced connections), the following plate design recommendations are made to obtain a full strength connection:

I. \( B_p = \) flat width of chord face
II. \( L_p \geq 2b_b \) (increase proportionately if \( \theta_i \neq 90^\circ \) to allow for greater bracing member "footprint")
III. \( t_p \geq 4t_c - t_b \)

The application of the above guidelines, for compression-loaded T, Y and X connections, should ensure that the connection capacity exceeds the bracing member capacity, provided chord side wall failure by web crippling is avoided\[27\].

3.5.2 With concrete filling

A less visible alternative to adding stiffening plates to the exterior of an RHS is to fill the hollow section chord with concrete or grout. Filling the chord members of an RHS truss, either along the full length of the chord or just in the vicinity of critical connections, has two main disadvantages: the concrete will increase the dead weight of the structure, and it involves a secondary trade with its associated costs. On the other hand, the strength of certain connections may increase, and if the members are completely filled, there are further benefits of enhanced member capacity and improved fire endurance. Concrete-filling of chord members can be done in the fabrication shop by tilting the truss and using a concrete or grout with a high water-to-cement ratio. If flange plates exist in the chord member, the filling can be restricted to just the distance from thetruss and to the flange plate.

The connections which benefit most from concrete-filling are X connections with the bracing members loaded in compression; that is, connections at which a compression force is being transferred through the RHS. Examples of such connections are truss reaction points, truss...
connections at which there is a significant external concentrated load, and beam-to-RHS column moment connections, as illustrated in Fig. 22. An experimental study of concrete-filled RHS members subjected to transverse compression loads has been made by Packer and Fear [29], who concluded that:

I. Concrete-filling of RHS greatly enhances their performance under transverse compression. The RHS provides some confinement for the concrete which allows it to reach bearing capacities greater than its crushing strength, as determined by cylinder compression tests.

II. For limit states design, the factored resistance of a concrete-filled RHS, compression-loaded, X connection could be taken as:

\[ N_e^* = (\phi_t t_A / \sin \theta_I) \sqrt{(A_p / A_t)} \]  

(3.11)

where

- \( \phi_t \) = resistance factor for concrete in bearing (0.6 in Canada, for example)
- \( t_A \) = crushing strength of concrete by cylinder tests,
- \( A_t \) = bearing area over which the transverse load is applied, and
- \( A_p \) = dispersed bearing area.

\( A_t \) should be determined by dispersion of the bearing load at a slope of 2:1 longitudinally along the chord member, as shown in Fig. 23. The value of \( A_t \) is limited by the length of concrete, and \( \sqrt{(A_p / A_t)} \) cannot be taken to be greater than 3.3.

III. The following are also recommended for general design application of Eqn. 3.11:

\[ h_0 / b_0 \leq 1.4 \]

\[ L_x \geq (h_0 / \sin \theta_I) + 2 h_0 \]

where \( L_x \) = length of concrete in RHS chord member.

IV. It has also been shown that shrinkage of the concrete (or grout) away from the RHS inside walls does not have a negative impact on the strength of a concrete-filled connection.

\[ A_1 = h_0 - b_1 \]

\[ A_2 = (h_0 + 2w_0) b_1 \]

Fig. 23 – Recommended method for determining bearing capacity of concrete-filled RHS loaded in transverse compression
3.5.3 Design Example

The 45°X connection given in Fig. 24 is subjected to the factored loads shown. The resistance of the connection will be checked to see if it is adequate. The members are hot-formed hollow sections with dimensions conforming to ISO/DIS 657-14 [223]. The steel grade is Fe510, conforming with ISO 630 [30], with a minimum specified yield strength of 355 N/mm².

Refer to Design Chart, Fig. 13 with \( \beta = 1.0 \), for a “Chord Side Wall Failure” check.

\[
N_t^* = C_{t_1}(t_{n1})(f_n)(A_n)(f_{y1})
\]

With \( \beta = 150/150 = 1.0 \) and \( b_{t_1}/t_o = 150/10 = 15 \), \( C_{t_1} = 0.49 \)

\[
n = (-1200)/5450 (0.355) = -0.620
\]

\[
\therefore f_n = 1.3 + (0.4/1.00) (-0.620), \text{ but } n \leq 1.0
\]

\[
\therefore = 1.0
\]

\[
\therefore N_t^* = 0.49 (1.0) (1.0) (5450) (0.355) = 948 \text{ kN} \leq 1200 \text{ kN} \therefore \text{Inadequate.}
\]

Also refer to design chart, Fig. 14 for an “Effective Width” check on the bracing member.

With \( (f_{y1}t_1)/t_o = 15 \) and \( b_{y1}/t_o = 15 \), \( N_t^* = 0.815 A_{t_1} f_{y1} \)

\[
\therefore N_t^* = 0.815 (5450) (0.355) = 1580 \text{ kN} > 1200 \text{ kN} \therefore \text{O.K.}
\]

Hence, the connection is still inadequate due to chord side wall capacity and must be reinforced, either by using plate reinforcement or concrete-filling. The chord should also be checked for shear (through the chord near the bracing member toes), as noted at the bottom of Table 3, but reinforcement is required anyway.

(a) Side plate reinforcement

A pair of side plates will be added to the chord side walls, with the side plates also having a yield strength of 355 N/mm².

From Table 3, for “Chord Side Wall Failure” with \( \beta = 1.0 \),

\[
N_t^* = 0.8 f_{y1} (t_{n1} + t_o) [2t_{n1}/\sin \theta_1 + 10t_o]
\]

Assume that a plate thickness of 10 mm (same as the chord), is chosen. Assuming that the chord side wall and plate act independently, they will have approximately the same compression resistance. Hence, it can be seen that the connection resistance will double when reinforced in this manner.

Hence \( N_t^* = 1900 \text{ kN} > 1200 \text{ kN} \therefore \text{O.K.} \)

For the length of the side plates, \( L_p \), the intent of Eqn. 3.8 for T and Y connections is that the plates extend 50% further than the bracing member “footprint”. Applying this same guidance to the X connection of Fig. 24, with two offset bracing member “footprints”.

\( L_p \geq 1.5 (150/\tan 45^\circ + 150/\sin 45^\circ) = 543 \text{ mm} \)

\( \therefore \) Make the stiffener plates 600 mm long \times 150 mm high \times 10 mm thick, and weld all around the plate perimeter.

---

Fig. 24 – RHS X connection design example
(b) Concrete filling

Fill the chord member of the connection shown in Fig. 24 with concrete having a crushing strength,

\[ f'_c = 40 \text{ N/mm}^2, \]
\[ N_i^f = (\phi_c f'_c A_1 / \sin \theta_1) \sqrt{(A_2/A_1)} \]
\[ = 0.6 (0.040) (31,820) (1.767) / \sin 45^\circ \]
\[ = 1908 \text{ kN} > 1200 \text{ kN} \therefore \text{O.K.} \]  \hspace{1cm} (3.11)

In this calculation \[\sqrt{(A_2/A_1)}\] was limited to 1.767 because \[A_2\] was interpreted to be (total footprint length shown on Fig. 24 + 500 mm) (150 mm) = 199,300 mm². Hence \[A_2/A_1 = 3.12.\]

For this chord member \[h_0/b_0 = 1.0\] is within the limit of experimental verification. An appropriate minimum length of concrete would be the “total footprint” (see Fig. 24) plus 2 \[h_0 = 0.7 \text{ metres.}\]
Warehouse rack structure
3.6 Cranked-chord connections

"Cranked-chord" connections arise in certain Pratt or Warren trusses such as the one shown in Fig. 25 and are characterized by a crank or bend in the chord member at the connection nodding point. The crank is achieved by butt (groove) welding two common sections together at the appropriate angle, and the intersection of the three member centre-lines is usually made coincident. The uniqueness of this cranked-chord connection lies both in its lack of a straight chord member and the role of the chord member as an "equal width bracing member". An experimental research program with square and rectangular members has revealed that unstiffened, welded, cranked-chord RHS connections behave generally in a manner dissimilar to RHS T or Y connections, despite their similar appearance (they all have a single bracing member welded to a uniform-size chord member). Instead, cranked-chord RHS connections have been shown to behave as overlapped K or N connections, and their capacity can be predicted using the current recommendations given in Table 2, subject to the limits of application in Table 2a[31]. Cranked-chord connections can be interpreted as overlapped K connections as shown in Fig. 26, wherein one chord member can be given an imaginary extension and the cranked-chord member is considered to be the overlapped bracing member.

![Diagram of a cranked-chord connection in a Pratt truss](image1)

Fig. 25 – Cranked-chord connection in a Pratt truss

![Diagram of a cranked-chord connection represented as an overlapped K or N-connection](image2)

Fig. 26 – Cranked-chord connection represented as an overlapped K or N-connection

3.6.1 Design example

The 45° cranked-chord connection given in Fig. 27 is subjected to the factored loads shown. The resistance of the connection will be checked to see if it is adequate. The RHS members have dimensions conforming to ISO/DIS 657-14[29], the steel grade is Fe510, conforming with ISO 630[30] with a minimum specified yield strength of 355 N/mm².

![Diagram of a RHS cranked-chord connection design example](image3)

Fig. 27 – RHS cranked-chord connection design example

Imagine the horizontal chord member extending as shown in Fig. 26 and both of the other members joining on to the top of the extended chord member.
Overlap (see Fig. 16) = q/p × 100% = 112.3 × 100/150 = 75%

Check range of validity for an overlapped K connection in Table 2a:

\[ \frac{b}{b_0} = 0.83 \text{ and } 1.00 \geq 0.25 \quad \therefore \text{O.K.} \]
\[ \frac{b_f/b_0} = 18 \leq 35 \quad \therefore \text{O.K.} \]
\[ \frac{b_f}{b_i} = 18 \leq 40 \quad \therefore \text{O.K.} \]
\[ \frac{b_i}{b_0} = 0.83 \geq 0.75 \quad \therefore \text{O.K.} \]
\[ O_i = 75\% \geq 25\% \]
\[ e = 0 \quad \therefore \text{O.K.} \]

\[ \therefore \text{From Table 2, for } 50\% \leq 80\%, \]
\[ N_i^* = \frac{f_{pB}}{2} \left[ 2h_i - 4l_i + b_b + b_{min} \right] \]

where \( b_b = \frac{10}{18}(1.00) = 53.3 \text{ mm} = b_{min} \), also.
\[ N_i^* = (0.355)(10) \left[ 300 - 40 + 2(83.3) \right] \]
\[ = 1510 \text{ kN} \geq 1200 \text{ kN} \quad \therefore \text{O.K.} \]

Alternatively, one could use Fig. 17 to calculate \( N_i^* \)

For \( b_f/b_0 = 18 \) and \( f_{pB}/f_{pB} = 1.0 \), partial efficiency = 0.37

For \( b_f/b_i = 18 \) and \( f_{pB}/f_{pB} = 1.0 \), partial efficiency = 0.37

\[ \therefore \text{Total efficiency} = 0.37 + 0.37 = 0.74. \]

or \( N_i^* = 0.74(0.355)(5450) = 1430 \text{ kN} \geq 1200 \)

(somewhat more conservative than direct use of equations).

Check (see Note to Table 2) on bracing efficiencies:

the efficiency of the overlapped bracing \( j \) should not be greater than that of the overlapping bracing, hence:

\[ N_j^* = \frac{f_{pA}}{f_{pA}} = 1510 \cdot \frac{0.355 \cdot 5450}{0.355 \cdot 6650} \]
\[ = 1840 \text{ kN} \geq 1700 \text{ kN} \]

\[ \therefore \text{still O.K.} \]
4 Truss design examples

4.1 Uniplanar truss

Truss layout and member loads

An example has been selected to illustrate the use of the connection design methods given in Chapter 3, as well as the truss design principles described in Chapter 2. A Warren truss consisting of square RHS is presented since that configuration is often the preferred solution. A Warren configuration with low bracing member angles, such as used here, keeps the number of connections to a minimum. All members chosen are hot-formed hollow sections with dimensions conforming to ISO/DIS 857-14 [29]. The steel grade throughout is Fe510 conforming with ISO 693 [30], with a minimum specified yield strength of 355 N/mm².

![Diagram of Warren truss with applied loads and resulting member forces](image)

Fig. 28 – Example Warren truss showing applied loads and resulting member forces (in kN)

Fig. 28 shows the truss and factored loads along with member axial forces, determined by a pinjointed analysis. The trusses are spaced at 12 metre intervals and the top (compression) chord is considered to be laterally supported at each purlin position. The span-to-depth ratio is 15, which is around the optimal upper limit considering service load deflections and overall costs (see Section 2.1).

Design of members

For member selection one could use either member resistance tables for the compression members, with the appropriate effective length, or the applicable strut buckling curve/equation. In practice one would also pay attention to the availability of member sizes selected. For this truss design example, compression member resistance has been determined in accordance with Eurocode 3 using buckling curve “a”. The resistance has also been calculated assuming $\gamma_r = 1.0$ (i.e. no partial safety factor or resistance factor, since this factor will be different for various countries (1.0 and higher)). Since the connections at the truss ends are generally critical, the chord walls selected should not be too thin, as a single size member will be used for the top chord and another single size member selected for the bottom chord.

Top chord

Use a continuous section with an effective length, for both in-plane and out-of-plane buckling, of $0.9L = 0.9 \times 6000 = 5400$ mm, as noted in Section 2.3.1, Eqns. 2.1 and 2.2

Maximum force $= 1148$ kN (compression).

Possible section sizes are shown in Table 4 below, along with their compressive resistances. As noted in Section 2.6, use $b_t/t_b$ ratios which are between 15 and 25, so select the $180 \times 180 \times 6.0$ RHS at this stage.
### Table 4 – Possible section sizes for top (compression) chord

<table>
<thead>
<tr>
<th>f_v0</th>
<th>N_1</th>
<th>KL</th>
<th>possible sections</th>
<th>A_0</th>
<th>b_0/t_0</th>
<th>λ</th>
<th>x</th>
<th>x f_v0 A_0</th>
</tr>
</thead>
<tbody>
<tr>
<td>355</td>
<td>1148</td>
<td>5.4</td>
<td>200 x 200 x 8.0</td>
<td>6050</td>
<td>25.0</td>
<td>0.93</td>
<td>0.71</td>
<td>1530</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>180 x 180 x 8.0</td>
<td>5410</td>
<td>22.5</td>
<td>1.04</td>
<td>0.64</td>
<td>1230</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>200 x 200 x 6.3</td>
<td>4820</td>
<td>31.7</td>
<td>0.92</td>
<td>0.72</td>
<td>1220</td>
</tr>
</tbody>
</table>

**Bottom chord**

### Table 5 – Possible section sizes for bottom (tension) chord

<table>
<thead>
<tr>
<th>f_v0</th>
<th>N_1</th>
<th>possible sections</th>
<th>A_0</th>
<th>b_0/t_0</th>
<th>f_v0 A_0</th>
</tr>
</thead>
<tbody>
<tr>
<td>355</td>
<td>1215</td>
<td>150 x 150 x 6.3</td>
<td>3560</td>
<td>23.8</td>
<td>1260</td>
</tr>
<tr>
<td></td>
<td></td>
<td>160 x 160 x 6.6</td>
<td>3480</td>
<td>26.6</td>
<td>1240</td>
</tr>
<tr>
<td></td>
<td></td>
<td>180 x 180 x 6.0</td>
<td>3460</td>
<td>36.0</td>
<td>1230</td>
</tr>
</tbody>
</table>

It is best, for connection capacity, to keep the tension chord as compact and stocky as possible; so select the **150 x 150 x 6.3 RHS** at this stage.

**Diagonals**

By aiming for gap connections, reference to the chart in Fig. 15 shows that the highest connection efficiency will be achieved when the ratio \( f_v0 b_0/t_1 \) is maximized. So try to select bracing members such that \( f_v0 b_0/t_1 \geq 2.0 \), which in this case implies \( t_1 \leq 3.15 \text{ mm} \), or near this thickness if possible. For the compression bracing members use an effective length of 0.75 L (Eqn. 2.3, Section 2.3.1) = 0.75 \( \sqrt{(2.4^2 + 3.0^2)} \) = 2.88 m.

**Compression diagonals**

### Table 6 – Possible section sizes for compression diagonals

<table>
<thead>
<tr>
<th>f_v1</th>
<th>N_1</th>
<th>KL</th>
<th>possible sections</th>
<th>A_1</th>
<th>b_1/t_1</th>
<th>λ</th>
<th>x</th>
<th>x f_v1 A_1</th>
</tr>
</thead>
<tbody>
<tr>
<td>355</td>
<td>432</td>
<td>2.881</td>
<td>120 x 120 x 4.0</td>
<td>1850</td>
<td>30.0</td>
<td>0.82</td>
<td>0.79</td>
<td>516</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>100 x 100 x 5.0</td>
<td>1890</td>
<td>20.0</td>
<td>1.00</td>
<td>0.67</td>
<td>448</td>
</tr>
<tr>
<td>355</td>
<td>259</td>
<td>2.881</td>
<td>100 x 100 x 3.6</td>
<td>1360</td>
<td>27.8</td>
<td>0.96</td>
<td>0.68</td>
<td>332</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>90 x 90 x 3.6</td>
<td>1240</td>
<td>25.0</td>
<td>1.10</td>
<td>0.69</td>
<td>293</td>
</tr>
<tr>
<td>355</td>
<td>96</td>
<td>2.881</td>
<td>70 x 70 x 3.2</td>
<td>850</td>
<td>21.9</td>
<td>1.42</td>
<td>0.41</td>
<td>123</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>80 x 80 x 3.2</td>
<td>978</td>
<td>13.9</td>
<td>1.23</td>
<td>0.51</td>
<td>178</td>
</tr>
</tbody>
</table>
Tension diagonals

Table 7 – Possible section sizes for tension diagonals

<table>
<thead>
<tr>
<th>f_d² N/mm²</th>
<th>N_2 kN</th>
<th>possible sections mm x mm x mm</th>
<th>A_d mm²</th>
<th>b_d/b_2</th>
<th>f_d² A_d kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>355</td>
<td>432</td>
<td>90 x 90 x 3.6</td>
<td>1240</td>
<td>25.0</td>
<td>440</td>
</tr>
<tr>
<td>355</td>
<td>259</td>
<td>70 x 70 x 3.2</td>
<td>850</td>
<td>21.9</td>
<td>302</td>
</tr>
<tr>
<td>355</td>
<td>66</td>
<td>30 x 30 x 2.5</td>
<td>272</td>
<td>12.0</td>
<td>97</td>
</tr>
</tbody>
</table>

Member selection

The number of sectional dimensions depends on the total tonnage to be ordered. In this example, only two different sections will be selected for the bracing members. A comparison of the members suitable for compression diagonals and tension diagonals shows that the following are most convenient:

Bracings: 120 x 120 x 4.0 RHS
120 x 80 x 3.2 RHS Note: 80 x 80 selected rather than 70 x 70 to conform to b_d ≥ 0.77 (b_d + b_y)/2 limit.

Top chord: 180 x 180 x 8.0 RHS
Bottom chord: 150 x 150 x 6.3 RHS

For square chords and θ_d = θ_y = 38.7°, Fig. 10 shows that gap connections are possible providing b_d ≤ 1.0, so the members selected allow gap connections.

The location of the sections selected, along with connection numbers, is shown in Fig. 29.

Commentary

At connections 1 and 4 the top chord member is welded to a flange plate for connecting to a column and a neighbouring chord member, respectively. At connection 1, a gap of 4b_d is chosen between the toe of the tension bracing member and the plate. This connection is checked as a K (or specifically N) connection, rather than Y, because the flange plate provides similar restraint to the chord face as a neighbouring compression bracing member of the same size as the tension bracing. Connection 4 is also checked as a K connection since the plates (see Fig. 29) again stiffen the connection, despite the loading being similar to an X connection. Hence all connections are checked as K (or N) connections and the chart in Fig. 15 can be used.

Table 8 gives the connection resistance calculations and it can be seen that all connections are adequate. This was possible due to an astute selection of member sizes, in which the ratio (f_d²/b_d/b_y) was kept as high as possible. Furthermore, realizing that a large bracing member
<table>
<thead>
<tr>
<th>connection</th>
<th>chord</th>
<th>bracings</th>
<th>connection parameters</th>
<th>actual efficiency</th>
<th>connection efficiency</th>
<th>remains</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>mm x mm x mm</td>
<td>mm x mm x mm</td>
<td>$b_1$, $b_2$, $b_3$, $t_1$, $t_2$, $t_3$</td>
<td>$\frac{N}{V}$</td>
<td>$\frac{A_V}{k_1}$</td>
<td>$N_{C_{max}}$</td>
</tr>
<tr>
<td>1</td>
<td>190 x 190 x 8.0</td>
<td>125 x 125 x 4.0</td>
<td>0.67, 0.56, 0.36, 0.36, 0.36, 0.36</td>
<td>0.08</td>
<td>0.86</td>
<td>0.34</td>
</tr>
<tr>
<td>2</td>
<td>190 x 190 x 8.0</td>
<td>125 x 125 x 4.0</td>
<td>0.67, 0.56, 0.36, 0.36, 0.36, 0.36</td>
<td>0.08</td>
<td>0.86</td>
<td>0.34</td>
</tr>
<tr>
<td>3</td>
<td>190 x 190 x 8.0</td>
<td>125 x 125 x 4.0</td>
<td>0.67, 0.56, 0.36, 0.36, 0.36, 0.36</td>
<td>0.08</td>
<td>0.86</td>
<td>0.34</td>
</tr>
<tr>
<td>4</td>
<td>190 x 190 x 8.0</td>
<td>125 x 125 x 4.0</td>
<td>0.67, 0.56, 0.36, 0.36, 0.36, 0.36</td>
<td>0.08</td>
<td>0.86</td>
<td>0.34</td>
</tr>
<tr>
<td>5</td>
<td>190 x 190 x 8.0</td>
<td>125 x 125 x 4.0</td>
<td>0.67, 0.56, 0.36, 0.36, 0.36, 0.36</td>
<td>0.08</td>
<td>0.86</td>
<td>0.34</td>
</tr>
<tr>
<td>6</td>
<td>190 x 190 x 8.0</td>
<td>125 x 125 x 4.0</td>
<td>0.67, 0.56, 0.36, 0.36, 0.36, 0.36</td>
<td>0.08</td>
<td>0.86</td>
<td>0.34</td>
</tr>
<tr>
<td>7</td>
<td>190 x 190 x 8.0</td>
<td>125 x 125 x 4.0</td>
<td>0.67, 0.56, 0.36, 0.36, 0.36, 0.36</td>
<td>0.08</td>
<td>0.86</td>
<td>0.34</td>
</tr>
</tbody>
</table>

Table 8: Check for connection resistance

54
would be adjacent to a much smaller bracing member at connections 2, 3 and 6, the 80 × 80 RHS was selected instead of the 70 × 70 RHS to satisfy \( b_i \geq 0.77 \left( b_i + b_2 \right)/2 \). Along the compression chord, all connections have zero noding eccentricity, which is usually the first choice of designers providing a sufficient gap results. On the tension chord, a noding eccentricity has been introduced at all the connections; but this does not influence the design of the tension chord or the connections (see Table 1, Section 2.2).

**Purlin connections**

Depending on the type of purlins, various purlin connections are possible. If light gauge purlins are used, such as cold-formed channel shapes for example, a popular form of purlin cleat is a section of angle welded to the top face of the chord member, extending across the full width of the RHS. The purlin would then be bolted to the outstanding leg of the angle. In this example, long span purlins are used; so they are likely to be I-sections, in which case angle cleats could be welded to each side of the RHS chord member and the purlin bolted through its flange to the outstanding leg of the angle as shown in Fig. 30.

![Fig. 30 – Possible purlin cleat connection at truss connection no. 2](image)

Arched roof trusses for a multi-use hall
4.2 Arch-formed truss

The connections of arched trusses can be designed in a similar way to those of straight chord trusses. If the arched chords are made by bending at the connection location only, as shown in Fig. 31 (a), the chord members can also be treated in a similar way to those of straight chord trusses provided that the bending radius remains within the limits to avoid distortion of the cross section [26, 32]. If the arched chords are made by continuous bending, the chord members have a curved shape between the connection locations, as shown in Fig. 31 (b). In this case the curvature should be taken into account in the member design by treating the chord as a beam-column. (Moment = axial force × eccentricity.)

Fig. 31 - Arched truss
5 Bolted connections

5.1 Flange-plate connections

RHS flange-plate connections have generally been bolted along all four edges of the plates; however the option of bolting along only two edges has been investigated during the 1980s and shown to be effective.

5.1.1 Bolted on two sides of the RHS

Preliminary tests on flange-plate connections bolted along two sides of the RHS as in Fig. 32 were performed by Mang [33] and Kato and Mukai [34], followed by a more extensive study by Packer et al. [35]. The latter showed that one could, by selecting specific connection parameters, fully develop the tensile resistance of the member by bolting along only two sides of the tube. This form of connection lends itself to analysis as a 2-dimensional prying problem, but the application of traditional prying models developed for T-stubs was found to not correlate well with the test results. One of the main reasons was attributed to the location of the hogging plastic hinge lines, which tended to form within the width of the tube as shown on Fig. 32.

A modified T-stub design procedure was consequently proposed [36] and verified against a set of possible failure mechanisms which were based on the observed failure modes. The design procedure involved a redefinition of various parameters in the T-stub design method of Struijk and de Back [37] which is now adopted by many structural steelwork codes. To reflect the observed location of the inner (hogging) plastic hinge line, and also represent the connection behaviour illustrated by the more complex analytical models, the distance b was adjusted to b' as shown on Fig. 32, where:

\[ b' = b - (d/2) + t \]  \hspace{3cm} (5.1)

The term \( \alpha \) has been used in Struijk and de Back's T-stub prying model to represent the ratio of the bending moment per unit plate width at the bolt line, to the bending moment per unit plate width at the inner (hogging) plastic hinge. Thus, for the limiting case of a rigid plate
\( \alpha = 0 \), and for the limiting case of a flexible plate in double curvature with plastic hinges occurring both at the bolt line and the edge of the T-stub web \( \alpha = 1.0 \). Hence, the term \( \alpha \) in Struik and de Back’s model was restricted to the range \( 0 < \alpha \leq 1.0 \). For bolted RHS flange-plate connections, this range of validity for \( \alpha \) was changed to simply \( \alpha \geq 0 \). This implies that the sagging moment per unit width at the bolt line is allowed to exceed the hogging moment per unit width, which is proposed because the RHS member tends to yield adjacent to the hogging plastic hinge and participate in the general failure mechanism. Thus, a suitable design method for this connection would be to initially estimate the number, grade and size of bolts required, knowing the applied tensile force \( N \) and allowing for some amount of prying. In general, the applied external load per bolt should be only 60% to 80% of the bolt tensile resistance in anticipation of bolt load amplification due to prying. Hence, determine a suitable connection arrangement. The bolt pitch \( p \) should generally be about 3 to 5 bolt diameters (although closer pitches are physically possible if wanted), and the edge distance a about 1.25 b. Prying decreases as a is increased up to 1.25 b, beyond which there is no advantage. Hence, maximum effective a for use in Table 9, is 1.25 b. Then proceed as shown in Table 9.

### Table 9 – Design steps for RHS flange-plate connections with bolts along two sides

<table>
<thead>
<tr>
<th>commentary</th>
<th>design equations</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. From a layout of the connection, calculate ( \delta ), the ratio of the net area at bolt line to gross area at the tube face</td>
<td>( \delta = 1 - d'/p ) (5.2)</td>
</tr>
<tr>
<td>2. Determine a trial flange-plate thickness ( t_p )</td>
<td>( \frac{K P r (1 + b)}{K P r (1 + b)} \leq t_p \leq \left( \frac{K P r}{2} \right)^{0.5} ) (5.3) where ( P_r = N/t ) = external factored tensile load/bolt load and ( K = 4b/(0.9 t_p) ) (5.4)</td>
</tr>
<tr>
<td>3. Calculate the ratio ( \alpha ) necessary for equilibrium, assuming bolts are loaded to their tensile resistance</td>
<td>( \alpha = \left[ \frac{K T_r}{L^2} - 1 \right] \left[ \frac{a + d/2}{2} (\alpha + b + t_p) \right] ) (5.5) where ( a \leq 1.25b; T_r = ) bolt tensile resistance</td>
</tr>
<tr>
<td>4. Calculate the connection factored resistance, ( N^* ) by using ( \alpha ) from Eqn. 5.5, except set ( \alpha = 0 ) if ( \alpha &lt; 0 ).</td>
<td>( N^* = \frac{L^2 (1 + \delta a)}{n/K} ) (5.6) where ( n = ) number of bolts ( N^* ) must be ( \geq N )</td>
</tr>
</tbody>
</table>
| 4a. If required, the total bolt tension, including prying \( T_r \), can also be calculated. \( \alpha \) from Eqn. 5.5 is not necessarily the same as that from Eqn 5.5 which assumed bolts were loaded to their full tensile resistance. Eqn. 5.8 becomes Eqn. 5.3 for \( \alpha = 0 \) or 1.0 | \( T_r = P_r \left[ \frac{1 + (b/2a')}{(\delta/2)(1 + \delta a)} \right] \) (5.7) where \( a' = a \) (but \( a' 

This design method should be restricted to the range of flange-plate thicknesses over which it has been validated experimentally and analytically[35, 36] namely 12 to 26 mm. It should be borne in mind that when a connection with bolts in tension is subject to repeated loads, the flange must be made thick enough and stiff enough so that deformation of the flange is virtually eliminated (\( \alpha \leq 0 \)). Most structural steelwork specifications require that bolts with tensile loads be pretensioned, a requirement that is essential for fatigue situations. Spacers placed between the plates in line with the RHS sides parallel to the bolt lines, as shown in Fig. 32, can avoid prying action and improve fatigue performance considerably[36].

58
5.1.2 Bolted on four sides of the RHS

CIDECT research programs on flange-plate connections bolted along all four sides, as in Fig. 33, have been undertaken by Mang [33] and Kato and Mukai [39], but a reliable connection design procedure has not yet evolved. Kato and Mukai proposed a complex model based on yield line theory with an estimate of the prying force. Depending on the relative strengths of the flange plate to the bolts, the ultimate strength of the connection was determined by one of six failure modes. Failure modes 1 to 3 involved failure of the flange plates while modes 4 to 6 involved bolt failure. Two recent connection tests of this type [40], however, have indicated that the model overestimated the strength of the connections by about 25%, so further investigation of this connection is still warranted.

Fig. 33 – Four and eight bolt configurations for bolting on all sides of an RHS

Kato and Mukai summarized, for connections on 150 and 200 mm square RHS, that the flange plate thickness should not be less than the bolt diameter for connections with four bolts, nor less than the bolt diameter plus 3 mm for connections having eight bolts. They used 16, 20 and 24 mm diameter bolts in their tests and decided that the "yield load" of the connection was approximately 0.8 times the sum of the original tensions in the bolts. Kato and Mukai also recommended that the external load per bolt be limited to no more than 75% of the bolt’s factored resistance T.

Kato and Mukai’s method for proportioning flange-plate thickness does not consider the plate yield strength. This fact and the evidence that some connection strengths are overestimated suggest that a more conservative approach be taken pending additional experimental work and comprehensive recommendations.

5.1.3 Design example for bolted flange-plate connection

For the unplanar truss design example in Section 4.1, the bottom chord splice to the right of connection 7 (see Fig. 29) can be made readily with flange plates bolted along two sides as shown in Fig. 34.

\[
egin{align*}
N_0 &= 1215 \text{ kN} \\
b &= 35 \text{ mm} \\
a &= 27 \text{ mm} \\
p &= 75 \text{ mm} \\
d &= 30 \text{ mm} \\
t_0 &= 6.3 \text{ mm} \\
 2a &= 45 \text{ mm} \\
a_{(\text{effective})} &= 43.8 \text{ mm} \quad (a < 1.25 b)
\end{align*}
\]

Fig. 34 – Tension chord splice in unplanar truss design example
Refer to Section 5.1.1 and make a trial arrangement. Load is 1215 kN.
Try 27 mm diameter high grade bolts, having a factored tensile resistance \(T_f\) of 300 kN

If 6 bolts are used, \(P_r = 202.5 \text{ kN and } P_r / T_f = 0.68\)

From Table 9, 
\[
\delta = 1 - \frac{d'}{d}\text{ (Eqn. 5.2)}
\]
\[
b' = b - d/2 + b_t \text{ (Eqn. 5.1)}
\]
\[
= 35 - 13.5 + 6.3 = 27.8 \text{ mm}
\]
\[
K = 4b'/(0.9 f_y p) \text{ (Eqn. 5.4)}
\]
\[
= 4(27.8)/(0.9(0.355)) = 4.64
\]

Plate with a yield strength of 355 N/mm\(^2\) has been assumed

\[
t_{min} = (KP)_{(1 + b/b')^{0.8}} \text{ (Eqn. 5.3)}
\]
\[
= (4.64(202.5)/(1 + 0.60))^{0.8} = 24.2 \text{ mm}
\]

\[
t_{max} = (KP)_{(b/b')^{0.3}} \text{ (Eqn. 5.3)}
\]
\[
= (4.64(202.5))^{0.3} = 30.6 \text{ mm}
\]

Try a 28 mm plate

Determine \(\alpha\), the ratio of the “sagging” plate moment at the edge of the bolts to the “hogging” plate moment within the tube.

\[
\alpha = \frac{[K(T_{r} / t_{b}) - 1] [(a + d/2)/(a + b + b_t)]}{[K(T_{r} / t_{b}) - 1] [43.8 + 13.5] / 0.60 [43.8 + 35 + 6.3]}
\]
\[
= 0.87
\]

Calculate the splice tensile resistance.

\[
N_s = \frac{1}{2} \frac{1 + \delta}{n/K}
\]
\[
= 28 \times 1 + 0.6 (0.87) / 6 / 4.64
\]
\[
= 1543 \geq 1215 \quad \text{ connection is adequate.}
\]

For general interest, calculate the actual total bolt tension, including prying force.

\[
T_r = P_r\left[1 + (b'/a') \left(\delta_a/(1 + \delta_a)\right)\right] \text{ (Eqn. 5.7)}
\]
\[
a' = \frac{b \times 0.75}{d} + d/2 = 43.8 + 13.5 = 57.3 \text{ mm}
\]
\[
\alpha = \frac{[K(T_{r}/t_{b}) - 1] / 8}{[K(T_{r}/t_{b}) - 1] / 6}
\]
\[
= (4.64(202.5)/(28^2))^{0.8} - 1] / 0.6 = 0.33
\]
\[
T_r = 202.5 \left[1 + (27.8/57.3)(0.60)(0.33) / (1 + (0.60)(0.33))\right]
\]
\[
= 219 \text{ kN} > 202.5 \text{ kN} (P_r)
\]
\[
< 300 \text{ kN} (T_f)
\]

5.2 RHS to gusset-plate connections

RHS bracing members can be field bolted to gusset plates which have been shop welded to RHS chord members, thus producing bolted shear connections as shown in Fig. 35. If

![Fig. 35 - Bolted RHS gusset-plate connections](image)
dynamic loading is a design consideration, this type of connection has an advantage over bolted flange-plate connections in that flange plates must be proportioned to eliminate all prying when fatigue loads are present. In general static load applications, however, the gusset-plate connections is less aesthetically pleasing and often more expensive than its flange-plate counterpart. An important limitation to the use of RHS gusset-plate connections is the need to have closely matching member widths. Equal width members are connected directly as in Fig. 35 (a), but the gussets often need to be spread slightly by jacking after welding is complete in order to allow field assembly (welding contraction tends to pull the gussets inwards). Small width differences can be adjusted by the use of filler plates welded on the sides of the bracing member. Larger differences allow the further option of extra plates, Fig. 35 (b) which can be more convenient in the field.

5.2.1 Net area, effective net area, and reduced effective net area

The concept of gross area, net area, effective net area, and reduced effective net area can be used to describe various failure modes for a tension member with holes or openings and these concepts will be utilized herein. The three basic checks are:

\[ T_s = \phi A_{ns} f_y \]  
(yielding on gross area) (5.9)

\[ T_s = 0.85 \phi A_{nse} f_y \]  
(rupture on effective net area) (5.10)

\[ T_s = 0.85 \phi A_{rse} f_y \]  
(rupture on effective net area reduced for shear lag) (5.11)

where \( \phi \) is a reduction factor which can be taken as 0.9 (Note: \( \phi = 1/\gamma_{su} \)). The 0.85 factor represents a suitable minimum margin between factored loads and the factored ultimate resistance for failure modes established by fracture of the tension members. The 0.85 \( \phi \) term = 0.77 is comparable to that used in other limit state codes (e.g. Eurocode 3 [21] requires a factor of 1/\( \gamma_{su} \) = 0.80).

The effective net areas \( A_{nse} \) is the sum of individual net areas \( A_n \) along a potential critical section of the member. Such a critical section may comprise net area segments loaded in tension, segments loaded in shear, and segments with a combination of the two loads. The method given provides a means of checking against "block shear" failures, whereby a chunk of material tears away from the piece by a combination of shear and tension ruptures.[21][41]

An illustrative example which includes all three types of net area segments is the gusset plate Y connection in Fig. 36, where the "block shear" area is calculated from the proposed failure line AM.

The tension segment, perpendicular to the load (AB), has \( A_{ns} = w_n t = (g_1 - d'/2) t \)

Shear segments parallel to the load (GH, JK, LM), have in total \( A_{ns} = 0.8 t_n L_t = 0.8 (L - 2.5 d') t \)

The 0.6 factor relates shear strength to tensile strength, so that sections failing in shear and tension can be "added together".

![Fig. 36 - Calculation of effective net area \( A_{net} \) for a gusset plate](image.png)
Each inclined segment (CD or EF) has
\[ A_n = (w_n + s^2/4g)l + (g - d)l + (s^2/4g)l \]

The effective net area \( A_{n_e} \) of the gusset plate for the potential critical section being examined is therefore the sum of all the net area segments above \( A_{n_1} + A_{n_2} + 2A_{n_2} \).

For bolted connections the effective net area, reduced for shear lag, \( A_{n_e} \), is the effective net area \( A_{n_0} \) multiplied by a shear lag factor. This factor comes into operation when a member is connected by some but not all of its cross-sectional elements, if the critical net section includes elements which are not connected. The critical net section in this instance may include net area segments \( A_n \), which are perpendicular to the load or inclined to it, but not those which are parallel to it. (This is not a check against block shear tear out.) The shear lag factor to be applied to \( A_{n_0} (A_{n_e} = \text{shear lag factor} \times A_{n_0}) \) can be taken as:

- 0.90 when shapes like I-sections (or tees cut from them) are connected only by their flanges with at least three transverse rows of fasteners (flange width at least two thirds the depth),
- 0.85 for all other structural shapes (e.g. RHS) connected with three or more transverse rows of fasteners,
- 0.75 for all members (e.g. RHS) connected with two transverse rows of fasteners.

For example, if the bracing member in Fig. 36 was an RHS and it was bolted to gusset plates on two sides as shown in the figure with each side having eight bolts, in three rows, then the reduced effective net area \( A_{n_0} \) would be 0.85 \( A_{n_0} \). In this instance the effective net area \( A_{n_0} \) would be the lesser of the failure paths AB-CD-EF-GN and AB-CF-GN (see Fig. 36).

The effective net area reduced for shear lag, \( A_{n_0} \), also applies to welded connections, when a member is not welded all around its cross-section. An example is Fig. 35 (b) where bolting plates were welded to the sides of the bracing member. For welds parallel to the direction of load (as welds would be in Fig. 35 (b), along the corners of the RHS), the shear lag factor is a function of the weld lengths and the distance between them. The distance between such welds would be \( b \). The shear lag factor to be applied to \( A_n (A_{n_0} = \text{shear lag factor} \times A_n) \) is:

- 1.00 when the weld lengths \( L \) along the RHS corners are \( \geq 2b \),
- 0.87 when the weld lengths \( L \) along the RHS corners are \( 1.5b \leq L < 2b \),
- 0.75 when the weld lengths \( L \) along the RHS corners are \( b \leq L < 1.5b \).

The minimum length of welds \( L \) is the distance between them.

Another failure mode of the gusset plate which must be checked is yielding across an effective dispersion width of the plate, which can be calculated using the Whitmore [42] effective width concept illustrated in Fig. 37. For this failure mode (for two gusset plates):

\[ N^* = \phi \frac{2L}{y} (g + 1.15 \Sigma \phi) \]

where \( \phi = 0.9 \) \hspace{1cm} (5.12)

The use of \( N^* \) indicates that this check applies to both tension and compression load cases. If the member is in compression, buckling of the gusset plate must also be prevented. The term \( \Sigma \phi \) represents the sum of the bolt pitches in a bolted connection or the length of the weld in a welded connection.

![Diagram](image)

Fig. 37 - Whitmore criterion for gusset-plate yielding

62
6 RHS to RHS moment connections

6.1 Vierendeel connections

6.1.1 Introduction to Vierendeel trusses

Arthur Vierendeel first proposed Vierendeel trusses in 1896. They are comprised of chord members connected to bracing members which are nearly always at 90° to the chords. The typical design premise with Vierendeel trusses has been to assume full connection rigidity, but this is very rarely the case with RHS to RHS Vierendeel connections. Unlike triangulated Warren or Pratt trusses, in which the connections approach a pinned condition at their ultimate limit state and cause the bracing members to be loaded by predominantly axial forces, Vierendeel connections have bracing members subjected to substantial bending moments as well as axial and shear forces. Until very recently, most of the testing performed on Vierendeel connections has been on isolated connection specimens with a lateral load applied to the vertical bracing member while the connection is in an inverted T position as shown in Fig. 38. Thus, the connection strength and moment-rotation behaviour have been assessed mainly by researchers under moment plus shear loading.

Fig. 38 – RHS Vierendeel connection types [51]

(a) Unreinforced
(b) With bracing plate stiffeners
(c) With chord plate stiffener
(d) With haunch stiffeners
(e) With truncated pyramid stiffeners
Square and rectangular RHS single chord connections loaded by in-plane bending moments have been studied by Duff [43], Redwood and Cuite [44], Mehrotra and Redwood [45], Lazar and Farg [47], Mehrotra and Goyal [48], Staples and Harrison [49], Brookenbough [50], Korol et al. [51], Korol and Mansour [52], Giddings [53], Kanatani et al. [54], Korol et al. [55], Korol and Mirza [55], Mang et al. [56], Davies and Panjeeshah [56], Szlendak and Brodka [57, 58, 59], Szlendak [60, 61], and Kanatani et al. [62]. Researchers concur that both the strength and flexural rigidity of an unstiffened connection decrease as the chord slenderness ratio $b_t/b_b$ increases, and as the bracing to chord width ratio $b_r/b_b$ (or $I$) decreases. Connections with $\beta = 1.0$ and a low $b_r/b_b$ value approach full rigidity, but all other unstiffened connections can be classed as semi-rigid. For such semi-rigid connections, Fig. 38 (b) to (e) gives a variety of means of stiffening which have been used to achieve full rigidity. From these alternatives, Figs. 38 (c) and (d) are recommended since the resistance of Fig. 38 (b) is limited by the effective width criterion and Fig. 38 (e) is rather expensive to fabricate.

### 6.1.2 Connection behaviour and strength

The connection ultimate moment capacity in tests is typically recorded, and Korol et al. [51] even developed an empirical formula for estimating the maximum connection moment, but this moment typically occurs at excessively large connection deformations. Thus, for all practical design purposes, the moment capacity of a connection can be determined in a manner similar to that used for axially-loaded RHS T connections, whereby the strength is characterized by an ultimate bearing capacity or by a deformation or rotation limit [19]. This design approach is more apparent if one considers the possible failure modes for such connections, which are shown in Fig. 39. The failure modes represented in Fig. 39 presume that neither the welds nor the members themselves are critical (e.g. local buckling of the bracing is precluded). Cracking in the chord (chord punching shear) has not actually been observed in any test, and chord shear failure is strictly a member failure, so analytical solutions for failure modes (b) and (e) are not considered herein.

![Fig. 39 - Possible failure modes for RHS connections loaded by in-plane bending moments][19]

- (a) Chord face yielding
- (b) Cracking in chord
- (c) Cracking in bracing member
- (d) Crippling of the chord side walls
- (e) Chord shear failure

For Mode (a), the moment capacity of connections with low to moderate $\beta$ values can be determined by the yield line model in Fig. 40. Neglecting the influence of membrane effects and strain hardening, the moment capacity is given by Wardenier [19]:

$$M_{\text{cr}} = 0.5 f_{y} b_{b} \left( 1 + \frac{4 h_{b}/b_{b}}{\sin \theta_{s} \sqrt[3]{1 - \beta}} + \frac{2 (h_{b}/b_{b})^{2}}{\sin^{2} \theta_{s} (1 - \beta)} \right) f_{y}$$

(6.1)

for $\beta \leq 0.85$.

---

[19]: Reference or Figure Number

---

64
Fig. 40 – Yield line mechanism for chord face yielding under in-plane bending (failure mode (a))

The term $f(n)$ is a function to allow for the reduction in connection moment capacity in the presence of large compression chord forces, according to de Koning and Wardenier.[63]

$$f(n) = 1.3 + \left( \frac{0.4}{\beta} \right) n, \quad \text{but } n \geq 1.0, \quad (6.1 \text{ a})$$

where $n = \frac{\text{axial compression load in the chord (negative) expressed as a fraction of the chord yield load}}{M_0/\rho_y + \frac{S_y}{\rho_y}}$.

For tension chords, $f(n) = 1.0$. Eqn. 6.1 a is shown graphically in Fig. 41.

Fig. 41 – Connection strength reduction factor $f(n)$, as a function of the compressive load in the chord expressed as a fraction of the chord yield load $\left( n = \frac{N_y/A_y}{f_y} + \frac{M_0}{S_y} \right)$.

For chord tension loading $f(n) = 1.0$

Nearly all Vierendeel connections have the bracing to chord angle $\theta = 90^\circ$, which simplifies Eqn. 6.1 to:

$$M_0 = \frac{f_y \rho_y h}{2 h_y/b_s \sqrt{1 - \beta} + h_y/b_s} \left( \frac{1}{1 - \beta} \right) f(n)$$

$$\text{for } \beta \leq 0.85.$$
For Mode (c), an effective width approach is used to relate the reduced capacity of the bracing member (considered to be the same on the tension and compression flanges of the bracing member) to the applied bracing moment as follows:

\[
M_{c} = f_{c} \left( 1 - \frac{b_{1} t_{1} (h_{1} - t_{1})}{b_{1} t_{0}} \right)
\]  
(6.3)

The term \(b_{1}\) in Eqn. 6.3 is the effective width of the bracing member flange, and is given by:

\[
b_{1} = \frac{10 b_{0} t_{0}}{f_{m} t_{0}} \quad \text{but} \quad b_{1} \geq b_{0},
\]  
(6.4)

For Mode (d), a chord side wall bearing or buckling capacity can conservatively be given by Eqn. 6.5 [19] which is illustrated in Fig. 42.

\[
M_{c} = 0.5 f_{c} t_{0} (h_{1} + 5 t_{0})^{2}
\]  
(6.5)

This moment is derived from stress blocks of twice (two walls) \(f_{c} t_{0} (h_{1}/2 + 2.5 t_{0})\) acting as a couple at centres of \(h_{1}/2 + 2.5 t_{0}\). Since the compression is very localized, tests [63] have shown that buckling is less critical for moment-loaded T connections than axially loaded T connections. Hence, within the parameter range of validity given, the chord yield stress can be used instead of the buckling stress for T connections. For X connections this is reduced by 20% to be consistent with Table 3.

Hence, for design purposes an estimate of the connection moment capacity can be obtained from the lower of the \(M_{c}^{*}\) values obtained from Eqns. 6.2, 6.3, and 6.5. It can be seen that the moment capacity predicted by Eqn 6.2 tends towards infinity as \(\beta\) tends towards unity, and so this failure mode, which corresponds to a state of complete connection plastification and hence high connection deformations, is not critical in the high \(\beta\) range. This accounts for the \(\beta \leq 0.85\) limit attached to Eqn. 6.2 and, for high \(\beta\) values, the web crippling failure criterion expressed by Eqn. 6.5 will likely govern. A summary of the design equations for in-plane moment loading is given in Table 10.

From the above expressions for \(M_{c}^{*}\) it can be seen that full width (\(\beta = 1.0\)) unstiffened RHS Vierendeel connections are capable of developing the full moment capacity of the bracing member, providing \(b_{0}/t_{0}\) is sufficiently low. For \(h_{0} = b_{0} = h_{1} = b_{1}\) and \(h_{0}/t_{0} \leq 16\), the chord side wall web crippling capacity is given by Warden (19):

\[
M_{c}^{*} = 12 f_{c} t_{0} b_{0} h_{1}
\]  
(6.6)

Since the plastic moment capacity of a square RHS bracing member is given approximately by

\[
M_{pl} = 1.5 b_{1} t_{1} f_{m}
\]  
(6.7)

then, \(M_{c}^{*} = \frac{8 f_{c} t_{0}}{b_{0}/t_{0}} \frac{f_{m} t_{0}}{f_{m} t_{1}}\)  
(6.8)

66
Table 10 – Factored resistance of welded T and X connections of rectangular hollow sections under in-plane and out-of-plane bending moments

<table>
<thead>
<tr>
<th>type of connection</th>
<th>factored connection resistance</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>T and X connections under</strong></td>
<td></td>
</tr>
<tr>
<td><strong>in-plane bending moments</strong></td>
<td></td>
</tr>
<tr>
<td>$\beta \leq 0.85$</td>
<td>basis: chord face yielding</td>
</tr>
<tr>
<td>$M_{w}^r = f_p t_{l} \left[ h_{l} \left( \frac{1}{2h_{l}/b_{l}} + \frac{2}{\sqrt{1-\beta}} + \frac{h_{l}/b_{l}}{(1-\beta)} \right) f(n) \right]$</td>
<td>(6.2)</td>
</tr>
<tr>
<td>$0.85 &lt; \beta \leq 1.0$</td>
<td>basis: effective width</td>
</tr>
<tr>
<td>$M_{w}^r = f_p \left[ Z_{l} - \left( 1 - \frac{b_{l}}{b_{l}} \right) b_{l} t_{l} (h_{l} - t_{l}) \right]$</td>
<td>(6.3)</td>
</tr>
<tr>
<td>$0.85 &lt; \beta \leq 1.0$</td>
<td>basis: chord side wall failure</td>
</tr>
<tr>
<td>$M_{w}^r = 0.5 t_{l} b_{l} (h_{l} + 5 t_{l})^2$</td>
<td>(6.5)</td>
</tr>
<tr>
<td><strong>T and X connections under</strong></td>
<td></td>
</tr>
<tr>
<td><strong>out-of-plane bending moments</strong></td>
<td></td>
</tr>
<tr>
<td>$\beta \leq 0.85$</td>
<td>basis: chord face yielding</td>
</tr>
<tr>
<td>$M_{w}^{r, \theta} = f_p t_{l} \left[ \frac{h_{l}}{2} \left( 1 + \beta \right) \cdot \frac{\sqrt{2b_{l}b_{l} \left( 1 + \beta \right) \left( 1 - \beta \right) } }{(1-\beta)} \right] \cdot f(n)$</td>
<td>(6.12)</td>
</tr>
<tr>
<td>$0.85 &lt; \beta \leq 1.0$</td>
<td>basis: effective width</td>
</tr>
<tr>
<td>$M_{w}^{r, \theta} = f_p \left[ Z_{l} - 0.5 t_{l} (b_{l} - b_{l})^2 \right]$</td>
<td>(6.13)</td>
</tr>
<tr>
<td>$0.85 &lt; \beta \leq 1.0$</td>
<td>basis: chord side wall failure</td>
</tr>
<tr>
<td>$M_{w}^{r, \theta} = f_p t_{l} (h_{l} + 5 t_{l}) (b_{l} - t_{l})$</td>
<td>(6.14)</td>
</tr>
</tbody>
</table>

**functions**

- $f(n) = 1.0$ for $n \geq 0$ (tension)
- $f(n) = 1.3 + \frac{0.4}{\beta} \cdot n$ for $n < 0$
- $n = \frac{N_{w}}{A_{w} f_{y}} + \frac{M_{w}}{S_{w} f_{y}}$
  - $f_{p} = 355$ N/mm²
  - $b_{l}/t_{l}$ and $h_{l}/t_{l} \leq 35$
  - $b_{l}/t_{l} \leq 1.1 \sqrt{E} f_{y},$
  - $\beta \leq 90^\circ$
  - $f_{y} = 200$ MPa

**range of validity**

- $f_{p} = 355$ N/mm²
- $b_{l}/t_{l}$ and $h_{l}/t_{l} \leq 35$
- $b_{l}/t_{l} \leq 1.1 \sqrt{E} f_{y},$
- $\beta \leq 90^\circ$
So, for the same steel grades used throughout a truss $\beta = 1$, dimensional ratios of $b_0/t_0 = 16$ and $t_0/t_1 = 2$ will produce a connection with a moment capacity approximately equal to the plastic moment capacity of the bracing [19] when hot-formed material is used. In this case, the bracing member cross-section is fully effective ($b_0 = b_1$ in Eqns 6.3 and 6.4). The above is similar to the recommendation by Korol et al. [51] for hot-formed sections that $b_0/t_0$ be less than 16 with $\beta = 1$ for full moment transfer to be assumed at the connection.

Any resistance factor ($\phi$) or partial safety factor ($\gamma_M$) is already included, where necessary, in the above resistance expressions of $M^c$ for their use in a limit states design format. Even though a rigorous evaluation of the proposed equations against all available experimental data has not yet been performed, experience suggests that these proposed equations will prove to be practical lower bounds on the connection moment capacity. The expressions for $M^c$ also have a limited range of validity which corresponds to the limits of the test data against which the equations have been checked. This validity range is: $b_0/t_0 \leq 35$, $h_0/t_0 \leq 35$, $\theta_i = 90^\circ$, $f_{yi}$ nominal $\leq 355$ N/mm$^2$, and the compression bracing member is restricted to plastic design sections (see Table 10).
The welds in RHS moment connections are loaded in a highly non-uniform manner and should also be capable of sustaining significant connection rotations. To enable adequate load redistribution to take place, the fillet weld sizes should be at least as large as those now specified for axially-loaded RHS truss connections to develop the capacity of the bracing member (see Section 2.2).

The previous expressions for moment capacity are based on moment loading only, whereas in Vierendeel trusses significant axial loads may also exist in the bracing members. The effect of the axial load on the connection moment capacity depends on the critical failure mode, and so a complex set of interactions is developed. Consequently, it is conservatively proposed that a linear interaction relationship be used to reduce the in-plane moment capacity of a Vierendeel connection as follows:

\[ \frac{N_1}{N_1^*} + \frac{M_{ip}}{M_{ip}^*} \leq 1.0 \]  

(6.9)

\(M_{ip}\) and \(N_1\) are the applied bending moment and axial load respectively in the bracing member, \(M_{ip}^*\) is the lower of the values obtained from Eqns. 6.2, 6.3 and 6.5 (Table 10), and \(N_1^*\) is the connection resistance with only an axial load applied to the bracing member (Table 3).
The resistance of an RHS T connection under bracing member axial load is given and discussed in Section 3.2, but is reproduced below for the most relevant case of $\beta = 1$. There are two failure modes to be checked: web crippling of the chord member side walls is again the likely governing failure mode, and can be estimated by $[23]$

$$N'_c = f_{y} t_{y} (2h_{c} + 10t_{y}) \quad (6.10)$$

where $f_{y}$ is determined as for Eqn. 6.5. The value for $f_{y}$ in Eqn. 6.10 assumes that the bracing member is in compression; if the bracing is in axial tension $f_{y} = f_{yt}$, which corresponds to chord wall tensile yielding. The other failure mode to check, for an RHS T connection with $\beta = 1$, is premature failure of the bracing member or connecting weldment. This is also termed an “effective width” check on the bracing member, and is expressed by $[23]$

$$N'_c = f_{y} t_{y} (2h_{c} - 4t_{y} + 2b_{c}) \quad (6.11)$$

where $b_{c}$ is given by Eqn. 6.4. Thus the connection resistance as an axially-load RHS T connection, for $\beta = 1$, will be given by the lower of the $N'_c$ values from Eqns. 6.10 and 6.11.

### 6.1.2.1 In-plane bending moments for T and X connections

The design criteria for RHS T connections with the bracing member subjected to an in-plane bending moment ($M_{ho}$) are described in Section 6.1.2. These are summarized in Table 10. For RHS X connections, subject to equal and opposite (self-equilibrating) in-plane bending moments ($M_{ho}$) applied to the bracing members, connection resistance formulae are also given in Table 10. These are the same as for RHS T connections except that a reduced bearing strength is used for the chord side wall failure mode.

In the case of the stiffened connection shown in Fig. 38(c), the effect of the stiffening can be treated in a similar way to that of axially-loaded, plate-reinforcement T connections (i.e. modify the formulae in Table 10 in a similar way to Section 3.5.1.2). For haunched connections with $\beta > 0.65$ as shown in Fig. 38(d), recommended minimum haunch dimensions are shown on the figure and connection resistance should be checked using Eqn. 6.5 in Table 10 with a modified value of $h_{c}$. For haunched connections with $\beta \leq 0.65$ use Eqn. 6.2 with a modified value of $h_{c}$.

### 6.1.2.2 Out-of-plane bending moments for T and X connections

For RHS T connections with the bracing member subjected to an out-of-plane bending moment ($M_{ho}$) as shown in Fig. 43, there is very little test evidence available to support any design models. However, one can postulate analogous failure modes to those described above for in-plane moment loading, which has been done for AWS [16].

(a) For $\beta \leq 0.85$, design would likely be governed by chord face yielding as shown in Fig. 43.

For this yield line mechanism,

$$M_{ho}^{*} = f_{y} t_{y} \left[ \frac{h_{c} (1 + \beta)}{2 (1 - \beta)} + \sqrt{\frac{2 b_{c} b_{y} (1 + \beta)}{(1 - \beta)}} \right] \cdot f_{n} \quad (6.12)$$

where $f_{n}$ is given by Eqn. 6.1a. It should be noted that for this failure all deformation takes place in the chord face and the chord will therefore not distort as a rhombus.

(b) For $0.85 < \beta \leq 1.0$, design would likely be governed by the more critical failure mode between: Reduced bracing member capacity (or an “effective width” failure mode), and chord side wall bearing or buckling capacity (see Fig. 44).

For “effective width” failure: $M_{ho}^{*} = f_{y} t_{y} \left[ Z_{1} - 0.5 t_{y} (b_{c} - b_{y}) \right] \quad (6.13)$

70
Fig. 43 – T connection subject to an out-of-plane bending moment, showing the chord face yielding failure mode for $\beta \leq 0.85$

$Z_1$ in Eqn. 6.13 is the plastic section modulus about the correct axis of bending, and plastic design sections should be selected for the bracing member. The term $b_s$ is defined by Eqn. 6.4.

For chord side wall failure: $M_{ce} = f_y t_0 (h_1 + 5t_0) (b_0 - t_0)$  \hspace{1cm} (6.14)

The term $f_y$ is the bearing strength of the chord side walls for T connections, and can be assumed to be equal to $f_{yc}$. These design provisions for RHS T connections subject to out-of-plane bending are summarized in Table 10.

Fig. 44 – T connection subject to an out-of-plane bending moment, showing the basis of design models for:
(a) Effective width failure mode
(b) Chord side wall failure mode

One could see that the design criteria for RHS X connections, subject to equal and opposite (self-equilibrating) out-of-plane bending moments ($M_{cd}$) applied to the bracing members, are again the same as those given above for T connections with one exception. The difference would be that when determining the resistance for the chord side wall failure mode, $f_y$ should be reduced to 0.8 $f_{yc}$. The design formulae are also covered by Table 10.

### 6.1.3 Connection flexibility

In the foregoing it was shown that unstiffened RHS connections with $\beta = 1$ and certain $b_0/t_0$ and $t_y/t_0$ values could achieve the full moment capacity of the bracing member, but it should be remembered that any connection moment resistance calculated ($M_{ce}$) must be reduced to
take account of the influence of axial load in the bracing member (see Eqn. 6.9). Such connections, which still develop a moment resistance exceeding the moment capacity of the bracing member, can be considered as fully rigid for the purpose of analysis of the Vierendeel truss. All other connections (which covers most possible connection combinations) should be considered as semi-rigid.

To analyse a frame, which is connected by semi-rigid connections, one needs the load-deformation characteristics of the connections being used, and these can be obtained by either reliable finite element analysis or from laboratory tests.

6.1.4 Design example

The Vierendeel truss shown in Fig. 45 is to be designed for a factored panel load P of 17 kN. All the connection points are laterally braced, perpendicular to the truss, by secondary members. The top and bottom chord members will be the same, and one section size will be used for all vertical (bracing) members. A statically admissible set of moments and shears follows in Fig. 46. Members will be designed using plastic analysis. All members chosen are hot-formed hollow sections with dimensions conforming to ISO/DIS 657-14 [29]. The steel grade throughout is Fe510 conforming with ISO 630 [30], with a minimum specified yield strength of 355 N/mm². Reductions in plastic moment capacity due to axial force or shear force can be shown to be negligible [84].

Fig. 45 – Example Vierendeel truss

Fig. 46 – Forces and moments within Vierendeel truss (forces and moments shown applied to nodes)
Select 150 × 150 × 10 RHS for the chord.

Confirm that this section is Class 1 (suitable for plastic design) at the worst axial load condition.

Maximum moment = 1.875 P = 31.9 kNm
Plastic moment of resistance = Zc × f'c (282) (0.355) = 100.1 kNm > 31.9 ∴ O.K.

Note: The resistance above has been calculated assuming γM = 1.0 (i.e. no partial safety factor or resistance factor), to be consistent with Chapter 4.1. Designers should introduce the appropriate partial safety factor/resistance factor for member design.

Therefore, 150 × 150 × 10 RHS is suitable for the chords.

Select 150 × 150 × 6.3 RHS for the vertical members.

Again, confirm that this section in Class 1 (suitable for plastic design) at the worst axial load condition.

Maximum moment = 3 P = 51.0 kNm
Plastic moment of resistance = (191) (0.355) = 67.8 kNm ≥ 51.0 ∴ O.K.
(This again ignores any partial safety factor/resistance factor to be consistent with member design elsewhere.)

Therefore, 150 × 150 × 6.3 RHS is suitable for the vertical members.

Plastic collapse mechanism

Fig. 47 illustrates the collapse mechanism. Let λ' be the additional multiplication factor by which the already factored loads have to be increased to cause plastic collapse. By the principle of Virtual Work,

\[
17λ'(3θ + 6θ + 6θ + 6θ + 3θ) = 100.1 (4θ) + 67.8 (8θ)
\]

\[
\therefore \lambda' = 2.31
\]

Therefore, adequate reserve capacity exists for ultimate strength as \(λ' \geq 1.0\).

Fig. 47 - Plastic collapse mechanism for Vierendeel truss

Connection capacity check

As \(λ' = 1.0\), the moment resistance of the connection could be limited by Mode (c), cracking in the bracing member, or Mode (d), chord side wall buckling (see Table 10).

Mode (c)

\[
M_{cc} = f'c \left( Z_c - (1 - b_y/b_t) b_t (h_y - t_y) \right)
\]

(6.3)

\[
b_y = \left[ \frac{10}{(b_y/t_y)} \right] (b_y/t_y) b_t
\]

(6.4)

= 158.7 ∴ use \(b_y = b_t\),

∴ \(M_{cc} = 0.355 \times 191 = 67.8 \text{ kNm} \geq 51.0 \therefore \text{O.K.} \)
Mode \( d \)

\[
M_c^2 = 0.5 f_{t_d} (h_1 + 5 t_d)^2 \quad (6.5)
\]

\[f_{t_d} = f_{t_d} \text{ for T connections (Table 10)}\]

\[
\therefore M_c^2 = 0.5 (0.355)(10)(150 + 50)^2 = 71 \text{ kNm} \geq 51.0 \quad \therefore \text{O.K.}
\]

\[\therefore \text{The limiting moment resistance is } 67.8 \text{ kNm.}\]

Now check that the moment and axial force interaction is satisfied according to:

\[
N_c^2 + \frac{M_c^2}{M_c^2} = 1.0 \quad (6.9)
\]

\[N_c^2 = f_{t_d} (2 h_1 + 10 t_d) = 0.355 (10)(300 + 100) = 1420 \text{ kN.}\]

or \[N_c^2 = f_{t_d} (2 h_1 - 4 t_1 + 2 b_d)\]

\[b_d = \frac{10 (b_1 / f_{t_d})}{(t_d / f_{t_d})} b_1 = b_1 \text{ as before}\]

so \[N_c^2 = 0.355 (8.3)(300 - 25.2 + 300) = 1286 \text{ kN}\]

\[\therefore \text{governing value of } N_c^2 = 1286 \text{ kN}\]

Hence, one should check the connections to the outside posts (maximum axial compression force of 1.75 \( P = 29.8 \text{ kN.} \)) and the connections to the most critical interior vertical (having a maximum moment of 3 \( P = 51 \text{ kNm.} \)).

For outside posts:

\[
(29.8/1286) + (31.9/67.8) = 0.49 \leq 1.0 \quad \therefore \text{O.K.}
\]

For interior verticalls:

\[(8.5/1286) + (51.0/67.8) = 0.76 \leq 1.0 \quad \therefore \text{O.K.}\]

Therefore, connection resistance is adequate and truss is satisfactory.

The members would also be suitable by elastic design procedures, and even with the introduction of partial safety factor (resistance factor) applied to member resistance. By either design method, the chord thickness is still enhanced to provide adequate connection strength, the end connections (at A, B, M and N) can be made by welding the vertical posts to the chord to form T connections, and then adding capping plates to the ends of the chord sections.

6.2 Knee connections

Research on mitred RHS knee connections (such as those in Fig. 48) has been performed by Mang et al. [565, 66] at the University of Karlsruhe. Their recommendations have also been reported by Wardenier [19] CIDEC [11] Dutta and Würker [25] and Eurocode 3 [21]. They cover both stiffened and unstiffened knee connections, and are intended for use in corner connections of rigid frames.

The original test results and moment vs. rotation diagrams are not widely available, but CIDEC [11] makes its design recommendations applicable for "flexurally-rigid frame corners." However, it could be expected that the rotation capacity of some unstiffened connections might be low, and in structures in which reasonable rotational capacity is required, a stiffened knee connection should be used [19]. In the Karlsruhe tests, simple unstiffened knee connections tended to fail by excessive deformation of the lateral RHS cross-
wall in compression. On the other hand, for connections with a stiffening plate, excessive deformations appeared only for very thin walled members. For thicker hollow sections, complete plastification was reached in the course of the tests. In view of the uncertain moment vs. rotation properties it is suggested to use, for unstiffened connections, only RHS members which satisfy plastic design requirements for rigid frames.

Analysis of the test results showed that, for design purposes, it was possible to estimate the total flexural and axial load capacity of the connection by applying a reduction factor to the material yield stress. Thus, adequate connection strength will be obtained for both stiffened and unstiffened 90° mitre connections providing Eqns. 6.15 and 6.16 are satisfied:

\[ \frac{N_i}{N_{yi}} + \frac{M_{ei}}{M_{ye}} \leq \alpha, \quad \text{for } i = 1 \text{ and } 2 \text{ (see Fig. 48)} \]

(6.15)

Where \( N_i \) is used here to refer to the axial resistance of member \( i \), either in compression or tension as applicable, \( M_{ei} \) refers to the moment resistance of member \( i \). The term \( \alpha \) is a stress reduction factor which can be taken as 1.0 for mitre connections with stiffening plates. For mitre joints without stiffening plates, \( \alpha \) is a function of the cross-sectional dimensions and is given in Figs. 49 and 50. For connections without stiffening plates, \( N_i \) should also not exceed 0.2 \( N_{yi} \).

The shear force acting at the connection \( V \) should also meet the requirement:

\[ \frac{V}{V_y} \leq 0.5 \]

(6.16)

where \( V_y \) is the shear yield load in the member under consideration. This can be taken as the yield stress in pure shear \( (f_y/\sqrt{3}) \) multiplied by the cross-sectional area of the RHS webs \( (2h_t \ell) \). If Eqn. 6.16 is not satisfied, the connection strength could still be deemed adequate, providing the combined stress does not produce failure according to the Von Mises failure criterion; in doing this check, the normal stresses (axial and bending) should be increased by a factor of 1/\( \alpha \).

75
For stiffened knee connections, the plate size should comply with \([21]:\)

\[ t_p \geq 1.5t_i \quad (i = 1 \text{ or } 2), \quad \text{and} \quad t_p \geq 10 \text{ mm}. \]  

(6.17)

The fabrication details shown in Fig. 48 are recommended. The weld size can be considered to be adequate when the throat thickness \(a\) is equal to the connected wall thickness, plus the factor \(\alpha\) (for unstiffened knee connections) is \(\leq 0.71\). This rule pertains to RHS having \(f_{yi} = 355 \text{ N/mm}^2\). If RHS with \(f_{yi} = 235 \text{ N/mm}^2\) are used for unstiffened knee connections, this can be adjusted to \(\alpha \leq 0.84\).
If mitred knee connections are used with an obtuse angle between the RHS members (i.e. $\theta > 90^\circ$ in Fig. 48), the same design checks can be undertaken as for right-angle connections, since obtuse angle knee connections behave more favourably than right-angle ones [1]. For unstiffened knee connections with $90^\circ < \theta < 180^\circ$, this strength enhancement can be used to advantage in Eqn. 6.15 by increasing the value of $\alpha$ as follows:

\[
\alpha = 1 - \left( \sqrt{2} \cos \frac{\theta}{2} \right) \left( 1 - \alpha_{\theta = 90^\circ} \right)
\]  

(6.18)

$\alpha_{\theta = 90^\circ}$ is the value obtained from Fig. 49 or Fig. 50.

An alternative form of connection reinforcement (other than a transverse stiffening plate) is a haunch on the inside of the knee. This haunch piece needs to be of the same width as the two main members, and can easily be provided by taking a cutting from one of the RHS sections. Provided the haunch length is sufficient to ensure that the bending moment does not exceed the section yield moment ($S_{fy}$) in either main member, the connection resistance will be adequate and does not require checking [1].

Quadrangular multiplanar truss for a pipeline bridge
Exhibition centre under construction; RHS box columns and girders supporting CHS space – frame panels

Triangular trusses of a baseball training centre
Fig. 49 – Stress reduction factors $\alpha$ for RHS subjected to bending about the major axis in 90° unstiffened mitred knee connection [65]

Fig. 50 – Stress reduction factors $\alpha$ for RHS subjected to bending about the minor axis in 90° unstiffened mitred knee connection [65]
7 Multiplanar welded connections

Multiplanar connections are frequently used in tubular structures such as towers, space frames, off-shore jacket structures, triangular trusses, quadrangular trusses and many other applications. Design rules for such connections, however, are given by specifications or codes only for CHS, and only by AWS [16] and CIDECT [67] at present.

RHS KK connections

Even less attention has been devoted to multiplanar connections between RHS compared to connections between CHS. Initial tests by Coutie at al. [68] on RHS multiplanar connections (KK) found a small decrease in the strength of the in-plane K connection due to out-of-plane loaded bracing members, as found for CHS connections. Bauer and Redwood [69] for KK connections to the single RHS chord of a triangular truss as shown in Fig. 51, deduced that there was little interactive effect produced by identical loading (same sense) on an adjacent wall of the chord. Bauer and Redwood’s study concentrated on connections with low to medium width ratios (β) between the bracing members and chord, for which the yield line method represented an ideal form of analysis. On this basis, it was suggested that in cases where the angle between bracing member planes (θ in Fig. 52) was less than 90° leading to an increase in the effective value of β at the chord face, and when the bracing members were attached to the chord face off-centre (as shown in Fig. 52), then the strength of a triangular truss tension chord face will be greater than that of a planar truss chord face with the same size members. As further failure modes may exist over a wider range of connection parameters than those studied by Bauer and Redwood [69], and as uniplanar, gap, K connection strength is traditionally assessed nowadays on the basis of ultimate strength rather than predicted yield strength, for simplicity it is suggested that a reduction factor of 0.9 be applied to the uniplanar K connection design formulae in Tables 2 and 3. This is the same reduction factor as recommended by CIDECT for CHS KK connections [67]. This recommendation for RHS KK connections is made for $60° \leq \theta \leq 90°$, where $\theta$ is shown on Fig. 52 [21]. In addition, it is suggested that one always performs a chord shear check for gap KK connections (see Table 11), even for square RHS members [21].

Fig. 51 – RHS triangular truss with double compression chords

Fig. 52 – Elevation view of KK connection to triangular truss tension chord
Triangular trusses, such as illustrated in Fig. 51, have several advantages over single plane trusses, such as the increased lateral stability offered by twin, separated, but connected compression chords. They are frequently used as exposed structures and considered equivalent in appearance, but less expensive, than space frames. In general, purlins are also not necessary with triangular trusses as the usual practice is to space the top chords of the trusses at a distance suitable for the roof deck, and then fasten the roof deck directly to the flat surfaces of the RHS top chords.

**RHS TT and XX connections**

It has been found that for CHS TT connections there is no change in the T connection strength due to out-of-plane loaded braces, while for CHS XX connections there is a significant increase in strength for pairs of bracings loaded in the same sense (or significant decrease for pairs of bracings loaded in the opposite sense) [67]. For RHS TT and XX connections however, Davies and Morita [70] have shown theoretically that very little difference exists between the
design strengths of planar and multiplanar connections, for 90° TT and XX connection shapes. Although lacking in experimental evidence at this stage, it is recommended that a correction factor of 0.9 be applied to the uniplanar T and X connection resistances (Tables 2 and 3) to account for out-of-plane loaded bracings.

A summary of the correction factors for multiplanar RHS connections is given in Table 11.

**Table 11 – Correction factors for RHS multiplanar connection resistances**

<table>
<thead>
<tr>
<th>type of connection</th>
<th>correction factor to uniplanar connection resistance from Table 2 or Table 3</th>
</tr>
</thead>
</table>
| KK \(60° \leq \varphi \leq 90°\) | 0.9 \[
\frac{N_{0 \text{(gap)}}}{A_0 f_y} + \frac{V}{A_0 f_y / \sqrt{3}} \leq 1.0
\] |
| TT, XX \(60° \leq \varphi \leq 90°\) | 0.9 |
8 Other uniplanar connections

8.1 Trusses with RHS bracing members framing into the corners of the chord

With some multiplanar (or even planar) RHS trusses it is possible to have the truss bracing members framing into the corners of an RHS chord member, as shown in Fig. 53. This necessitates very careful profiling of the bracing member end, particularly where corner radii are large, into so called “bird-mouth” or “bill-shaped” joints. Such a member arrangement has been used occasionally in North America, for example in the Minneapolis Convention Center Roof and in the Minneapolis/St. Paul Twin Cities Airport Skyway. It has also been used in Japan, where in this case a robot was developed to profile the ends of the bracing members. By framing into the corners of the RHS chord member a high connection strength and stiffness is achieved, regardless of the bracing to chord member width ratio. Ono et al. [71] have undertaken an experimental study of such square RHS T and K connections and have found that these connections are much stronger than their conventional RHS counterparts with the chord and bracings rotated through 45° about the member axis. All of the T connections tested (25) had the bracing loaded in compression, and the K connections (16) had all bracing members inclined at 45° to the chord. The orientation of the bracing and chord member is shown in Fig. 53, and one should also note that the bracing member is axially rotated. Ono et al. concluded that the connection ultimate strengths could be given by:

For T connections:

\[ N_{\text{tu}} = 0.5 f_y \left( \frac{1}{0.211 - 0.147 \left( \frac{b_2}{b_0} \right) + 1.794 - 0.942 \left( \frac{b_2}{b_0} \right)} \right) f'(n') \]  

(8.1)

For K connections:

\[ N_{\text{tu}} = \frac{0.5 f_y}{\sqrt{1 + 2 \sin^2 \alpha}} \left( 4 \alpha \right) \left( \frac{d_2}{d_0} \right) f'(n') \]  

(8.2)

where the effective area coefficient \( \alpha \) is given for 45° K connections in Fig. 54. \( f'(n') \) is a function used for CHS connections to allow for the influence of normal stresses in compression chords, and is given by [67]:

\[ f'(n') = 1 + 0.3 n' - 0.3 n'^2, \quad \text{for} \ n' < 0 \]  

(compression)  

(8.3)

and \( f'(n') = 1.0, \quad \text{for} \ n' \geq 0 \) (tension)  

(8.3a)

where \( n' = f_{uy} / f_{yc} \).  

(8.3b)
As these equations are based on a regression analysis of the test data, one should be careful to ensure that they are only applied within the approximate bounds of parameter ranges examined in the tests (i.e. \(16 \leq b_t/t_o \leq 42\) and \(0.3 \leq b_t/b_o \leq 1.0\) for T connections; \(16 \leq b_t/t_o \leq 44, 0.2 \leq b_t/b_o \leq 0.7\) and \(\theta = 45^\circ\) for K connections). Resistance factors or partial safety factors are necessary for application of Eqs. 8.1 and 8.2 to limit states design, giving the factored resistance expressions (with rounding of the constants), below:

For T connections:

\[
N_t' = 0.9 \frac{f_y}{5} \left( \frac{1}{0.21 - 0.15 (b_t/b_o)} + \frac{b_t/t_o}{1.79 - 0.94 (b_t/b_o)} \right) f(n')
\]  
(8.4)

For K connections:

\[
N_t' = \frac{0.9 \frac{f_y}{5} \sigma_0}{\sqrt{1 + 2 \sin^2 \theta}} (4 \alpha) (b_t/t_o) f(n')
\]  
(8.5)

with \(\alpha\) given by Fig. 54.

8.2 Trusses with flattened and cropped-end CHS bracing members

For statically loaded hollow section trusses of small to moderate span, cropping – a procedure in which a CHS bracing member is simultaneously flattened and sheared – can simplify fabrication and reduce cost. The procedure is faster than sawing or profiling, the conventional methods of preparing CHS bracing members for welding to RHS and CHS chords, respectively, and it simplifies the welding process. Typical cropped-bracing Warren truss connections to an RHS chord member are shown in Fig. 55. Note that the flattened ends of the bracing member can be aligned in the direction of the truss or transverse to it. For all trusses with flattened or cropped-bracing members an effective length factor of 1.0 should be used for design of the bracing members.

Flattening the CHS bracings in the plane of the truss does not provide as good a structural performance, nor the economies of fabrication, compared to out-of-plane flattening. [72]
Although this has been argued for CHS chord members, the out-of-plane flattening of CHS bracing members and welding to RHS chord members is the basis of the “Strarch” roof system [73]. At this stage no design guidance is available for such connections. As shown in Fig. 56, various types of flattening can be performed on the CHS bracing members. 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. 56 [77]. For d/t ratios exceeding 25 the flattening will reduce the bracing member compressive strength [77]. For welded connections the length of the flat part should be minimized for compression bracing members to avoid local buckling in the flattened region.

Considerable research has been performed on in-plane cropped-end CHS bracings to RHS trusses [75] and 45 Warren isolated connections [76] have been reported. The latter tests had the geometry shown in Fig. 57, in which the toes of the flattened bracing members just met at
Separated double chord truss construction
the chord face, with no overlap or gap between them. That particular geometry has been found to be economical and practical for cropped-bracing connections to RHS chords. For this connection configuration in Fig. 57, with symmetrical bracing members, Morris and Packer [77] showed that the connection resistance was given by:

$$N_{y1}^* = 0.4 N_{y1} \left( 1 + 0.22 \frac{b_0}{t_0} \right) \left( 1 + 1.71 \frac{d_1}{b_0} \right)$$

(8.6)

where

$$N_{y1} = \frac{t_0^2 f_{y0}}{\sin \theta_1} \left( \frac{\pi}{2} + \frac{(b_1' + 2h_1')}{b_0' - b_1'} + \frac{1.32}{t_0} \sqrt{\frac{f_{y1}}{f_{y0}} \cdot \tan \theta_1 \cdot b_0' \cdot t_1} \right) \cdot f(n).$$

(8.7)

$$b_0' = b_0 - t_0$$

$$b_1' = \text{width of flattened bracing member (with full cropping and flattening, this can be assumed to be } 2t_1. \text{ If fillet welding is used, this effective contact width can be increased to include the fillet weld leg dimensions).}$$

$$h_1' = \frac{\pi (d_1 - t_1) + t_1}{2 \sin \theta_1}$$

and

$$\theta_1' = \text{slope of bracing member face at the cropped end, relative to the chord (see Fig. 57).}$$

Conservatively, a value of $\theta_1' = \theta_1$ can be used.

Eqns. 8.6 and 8.7 apply to symmetrical connections where $\theta_1 = \theta_2$, $d_1 = d_2$, $t_1 = t_2$, $d_1/b_0 \geq 0.3$ and $b_0/t_0 \leq 32$. 

87
8.3 Double chord truss connections

Limitations on the largest available RHS member size have restricted the application range of RHS structures. For very long span roof trusses, such as sports centres and auditoria, the use of double RHS chord members will enable longer clear spans than those available from single chord trusses. Immediate advantages of double chord RHS trusses include not only their greater span capacity, but also more efficient and stiffer connections compared to some single chord trusses. Enhanced lateral stiffness can reduce lateral bracing requirements as well as facilitate handling and erection of the structural components.

Research has been undertaken in Canada [78, 79, 80, 81, 82, 83] on isolated connections and trusses of the types shown in Fig. 58. The two separated chord trusses types require that all the bracing members have the same width; in such cases the bracing member sizes can be varied by changing the bracing member wall thickness (t) or depth (h). For the separated chord bolted connection (Fig. 58(b)), it is recommended that tie bars be used between the RHS chord members on the outside of the truss as they significantly increase the truss stiffness by maintaining the alignment of the sections. However, connections without these tie bars are almost as strong, just more flexible.

![Diagram of double chord truss connections](image)

**Fig. 58 - Types of RHS double chord connections**
(a) Separated chord welded connection
(b) Separated chord bolted connection
(c) Back-to-back chord connection

For RHS double chord trusses it is recommended that a pin-jointed analysis be used with effective length factors (K) as given in Section 2.3.1, when designing the compression members. A microcomputer program DCTRUS 
* for the analysis and design of RHS double chord trusses is available [82, 83]. Connection resistance expressions have only been proposed for the separated chord welded connection, and these can be simplified to:

*This software program has been made basing on Canadian specification [411]
\[ N_y^* = \frac{l_x A_y}{\sqrt{3} \sin \theta} \]  
(8.8)

where \( A_y = 2.6 h_y b_y \) for \( h_y/b_y \geq 1 \)  
(8.8a)

and \( A_y = 2 h_y b_y \) for \( h_y/b_y < 1 \)  
(8.8b)

Eqsns. 8.8a and 8.8b take into account the reduced effectiveness of the chord outer side walls in resisting shear forces, at different chord aspect ratios.

The interaction of axial force and shear force in the gap region of the double chord connection should also be checked. The connection eccentricity has been found to have little effect on the connection strength and pin-jointed analysis is recommended for the truss analysis, so that moments acting on the connection can be ignored. The axial force/shear force interaction can be checked in a manner similar to that used in Table 3, such that:

\[ N_{y\text{min}} = 2 (A_y - A_i) V_{y0} + A_y V_p \left[ 1 - \left( \frac{V}{V_p} \right)^2 \right]^{0.5} \]  
(8.9)

In Eqn. 8.9, \( A_i \) is the area of one chord member, \( A_y \) is given by Eqns. 8.8a and 8.8b, \( V \) is the vertical shear force applied to the connection (N sin \( \theta \), assuming no "purlin load"). \( V_p \) is given by:

\[ V_p = \frac{l_x A_y}{\sqrt{3}} \]  
(8.10)

A recent economic comparison of single chord and double chord RHS trusses [83] has shown that for short spans single chord trusses were the lightest and most economical, being around 20% less expensive than back-to-back double chord trusses. (Back-to-back double chord trusses are generally the heaviest and most expensive option for welded trusses.) Thus, for long spans separated double chord welded connections are preferable and will again prove more economical than back-to-back connections.

8.4 Plate to RHS connections

Plates are sometimes welded to the faces of RHS members for bracing connections, purlin cleat connections, hanger connections, and a pair of plates can even be used to represent the flanges of an I-section beam to RHS column moment connection. For the latter, the connection moment resistance can be obtained by multiplying the plate axial force resistance by the beam depth (h_b – t_b).

Plates can be welded longitudinal or transverse to the RHS member axis. A longitudinal plate will always have a very low \( \beta \) value and hence be an extremely flexible connection, with the axial resistance governed by the formation of a yield line mechanism which represents a control on the connection deformation. This is reflected in the design resistance given in Table 12, but this plate orientation is not recommended.

A transverse plate with a low to medium \( \beta \) value will also develop a yield line mechanism in the RHS connection face. For large \( \beta \) values, but where \( \beta \) is still less than 1 – 1/\( \gamma \), punching shear failure of the chord face is the most probable failure mode, and for somewhat lesser \( \beta \) values a combination of flexural failure and punching shear will likely occur. This combined failure mode has been studied by Davies and Packer [84] but is too complicated for routine design. However the bracing (plate) load capacity can also be reduced by a non-uniform stress distribution in the plate, which is termed a bracing "effective width" failure criterion [85, 86]. Moreover, it has been shown [83] that bracing effective width always governs over chord face yielding as a critical failure mode for \( \beta \leq 0.85 \), so the chord face yielding failure mode is omitted in Table 12 and a bracing effective width check is included for all \( \beta \) values. At \( \beta = 1.0 \),
the plate will bear directly on the RHS side walls and so side wall failure is the pertinent failure mode for which the connection must be designed. If the bracing is loaded in compression the RHS chord side wall buckling stress can still be taken as the yield stress ($f_{y}$) as the compression is very localized. The foregoing design criteria are all summarized in Table 12.

<table>
<thead>
<tr>
<th>Table 12 – Factored resistance of plate to RHS connections</th>
</tr>
</thead>
<tbody>
<tr>
<td>type of connection</td>
</tr>
<tr>
<td>---------------------</td>
</tr>
<tr>
<td>longitudinal plate</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>transverse plate</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>functions</td>
</tr>
<tr>
<td>$b_{wo} = \frac{10}{b_y/t_2} \cdot b_1$, $b_1 \leq b_1$</td>
</tr>
<tr>
<td>$b_y = \frac{10}{b_y/t_2} \cdot \bar{b}_1$, $b_1 \leq b_1$</td>
</tr>
<tr>
<td>range of validity</td>
</tr>
<tr>
<td>$b_y/t_2 \leq 30$</td>
</tr>
</tbody>
</table>
9 List of symbols

a
Throat thickness of a fillet weld; edge distance of plate from centre of bolt hole (see Fig. 32)
a' = a (effective) + d/2; a (effective) = a, but ≥ 1.25 b (see Fig. 32)
A1
Bearing area over which a transverse load is applied = h1 b1
A2
Dispersed bearing area = (h1 + 2 w2) b1
A3
Cross-sectional area of member i (i = 0, 1, 2, 3)
A4
Gross cross-sectional area of member = A3
Ar
Effective net cross-sectional area of tension member
Ar e
Effective net cross-sectional area reduced for shear lag
Ae
Effective shear area of the chord (see Table 3 and Fig. 7)
b
Distance from bolt line to the hollow section face (see Fig. 32)
b' = b - d/2 + t1 (see Fig. 32)
b2
Effective width of a bracing member (see Tables 2 and 3, and Fig. 8)
b2w0
Effective width for overlapping bracing member connected to overlapped bracing member (see Tables 2 and 3, and Fig. 8)
bw
Effective punching shear width (see Table 3 and Fig. 8)
b0
External width of square or rectangular hollow section (RHS) member i (90° to plane of truss) (i = 0, 1, 2, 3)
b0 X,T,X
Uncorrected connection efficiency, for K, T, and X-type connections respectively, expressed as a proportion of the yield load (A fy,i) for a particular bracing member.
d
Nominal bolt diameter
d'
Bolt hole diameter
dh
External diameter of circular hollow section (CHS) for member i (i = 0, 1, 2, 3)
e
Nodding eccentricity for a connection – positive being towards the outside of the truss (see Fig. 1)
E
Modulus of elasticity
tc
Crushing strength of concrete
f a
Axial stress in member i (i = 0, 1, 2, 3)
f m
Buckling stress according to steelwork specification, using a column slenderness ratio of K L/r
f m u, f m h
Yield stress of member i (i = 0, 1, 2, 3) or j
f p
Yield stress of plate
f p u
Ultimate tensile stress of member i (i = 0, 1, 2, 3)
f(n)
Functions in the connection resistance formulae which incorporate the influence of normal stresses in compression chords
f(n') Function in "bird-mouth" connection resistance formulae which incorporates the influence of prestress in compression chords
f0
Maximum applied axial stress in chord (or maximum stress due to axial force and bending moment where moment is taken into account)
g
Gap between the bracing members (ignoring welds) of a K, N, or KT connection, at the face of the chord (see Fig. 5); bolt gauge, or distance between bolt lines (see Fig. 36)
g'
Gap divided by chord wall thickness, g' = g/tc
h0
External depth of square or rectangular hollow section (RHS) member i (in plane of truss) (i = 0, 1, 2, 3)
i
Subscript to denote member of connection; i = 0 designates chord; i = 1 refers in general to the bracing for T, Y and X connections, or it refers to the compression bracing member for K, N, and KT connections; i = 2 refers to the tension bracing member for K, N and KT connections; i = 3 refers to the vertical for KT connections; i = 3 refers to the overlapping bracing member for K and N type overlap connections
Moment of inertia of member

Subscript to denote the overlapped bracing member for K and N-type overlap connections

Effective length factor

Length of member

Length of concrete in RHS chord member

Net length

Length of plate

In-plane bending moment applied to bracing member

Connection resistance for in-plane bending, expressed as a bending moment in bracing member

Out-of-plane bending moment applied to bracing member

Connection resistance for out-of-plane bending, expressed as a bending moment in bracing member

Plastic moment capacity of member i

Moment resistance of member i

Bending moment in chord member

\[
\frac{f_{ip}}{f_{po}} = \frac{N_{ip}}{N_{po}} + \frac{M_{ip}}{S_{ip}f_{po}}
\]

Number of bolts

Axial force applied to member i \((i = 0, 1, 2, 3)\)

Reduced axial load resistance, due to shear, in the cross section of the chord at the gap

Axial resistance of member i

Overlap, \(O_r = q/p \times 100\%\) (see Fig. 16)

Length of projected contact area between overlapping bracing member and chord without presence of the overlapped member (see Fig. 16): length of flange-plate attributed to each bolt, or bolt pitch (see Fig. 32 and Fig. 37); subscript to denote a plate

External tensile load applied to a bolt

Length of overlap between bracing members of a K or N connection at the chord face (see Fig. 16)

Radius of gyration

Bolt spacing (see Fig. 36)

Elastic section modulus of member i

Thickness of hollow section member i \((i = 0, 1, 2, 3)\) or j

Thickness of plate

Tensile force applied to member or component

Tensile resistance of member or component or bolt

Ultimate tensile capacity of a bolt

Applied shear force

Shear yield capacity of a section (see Table 3)

Net width

Dispersion width (see Fig. 23)

Plastic section modulus of member i

Non-dimensional factor for the effectiveness of the chord flange in shear; ratio of the equilibrating moment per unit plate width at the bolt line to the flange moment at the inner plastic hinge; axial force/bending interaction factor for mitred knee connections; effective area coefficient for "bird mouth" connections
Width or diameter ratio between bracing member(s) and chord

\[ \beta = \frac{d_1}{b_0} \cdot \frac{b_0}{b_0} (T, Y, X) \]

\[ \beta = \frac{d_1 + d_2}{2b_0} \cdot \frac{b_1 + b_2 + h_1 + h_2}{4b_0} (K, N) \]

\[ \beta = \frac{d_1 + d_2 + d_3}{3b_0} \cdot \frac{b_1 + b_2 + b_3 + h_1 + h_2 + h_3}{6b_0} (K, T) \]

\[ \beta_p \] Width or diameter ratio between bracing member and plate, \( \beta_p = b_i/B_p \)

\[ \phi \] Connection resistance factor (inverse of partial safety factor \( \gamma_u \))

\[ \gamma \] Angle between planes of multiplanar joints

\[ \gamma \] Half width to thickness ratio of the chord \( \gamma = \frac{b_0}{2t_0} \)

\[ \eta \] Bracing member depth to chord width ratio \( \eta = \frac{h}{b_0} \)

\[ \eta_p \] Bracing member depth to plate width ratio \( \eta_p = \frac{h}{B_p} \)

\[ \theta_i \] Included angle between bracing member \( i (i = 1, 2, 3) \) and the chord

\[ \lambda \] Slenderness of a member under compression [67]

\[ x \] Reduction factor for buckling curves [87]

Note: When mechanical or geometric properties of members in the List of Symbols are used in limit states design equations, or in conjunction with design charts, the nominal or specified values are to be used.
10 References


[23] Zhao, X.-L., and Hancock, G. J.: Tubular T-joints subject to combined actions. 10th. International Specialty Conference on Cold-Formed Steel Structures, St. Louis, Missouri, U.S.A., October 1990, Proceedings pp. 545-573.


[56] Davies, G., and Panjehshahi, E.: Tee joints in rectangular hollow sections (RHS) under combined axial loading and bending. 7th International Symposium on Steel Structures, Gdansk, Poland, 1984.


[82] Luft, R. T.: DCTRUS - A computer program to aid in the analysis and design of separated HSS double chord Warren trusses. Civil Engineering Department, McMaster University, Hamilton, Canada, 1990.


Acknowledgements for photographs:
The authors express their appreciation to the following firms for making available the photographs used in this Design Guide:

British Steel
Mannesmann-Röhrenwerke
Nippon Steel Metal Products
Rautaruukki
Mannstaedt Werke
SIDERCAD
Valex
Voest Alpine Krems
Van Leeuwen
Hoesch Rohr
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 recommendations, 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.

The organization of CIDECT comprises:

- President: J. C. Ehlers (Federal Republic of Germany)
  Vice-President: C. L. Bijl (The Netherlands)

- A General Assembly of all members meeting once a year and appointing an Executive Committee responsible for administration and executing of established policy

- Technical Commission and Working Groups meeting at least once a year and directly responsible for the research and technical promotion work
Present members of CIDECT are:

(1992)

- Altos Hornos de Vizkaya S.A., Spain
- British Steel PLC, United Kingdom
- Hoechst Rohr AG, Federal Republic of Germany
- ILVA Form, Italy
- IPSCO Inc., Canada
- Laminors de Longtain, Belgium
- Mannesmannrohren-Werke AG, Federal Republic of Germany
- Mannstädter Werke GmbH, Federal Republic of Germany
- Nippon Steel Metal Products Co. Ltd., Japan
- Rautaruukki Oy, Finland
- Sonnichsen A/S, Norway
- Tubemakers of Australia, Australia
- Van Leeuwen, The Netherlands
- Valexy, France
- Verenigde Zuilenfabrieken (VBF), The Netherlands
- VOEST Alpine Krems, Austria

Cidec Research Reports can be obtained through:

Mr. D. Dutta
Office of the Chairman of the CIDECT Technical Commission
c/o Mannesmannrohren-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

Care has been taken to ensure that all data and information herein is factual and that numerical values are accurate. To the best of our knowledge, all information in this book is accurate at the time of publication.

CIDECT, its members and the authors assume no responsibility for errors or misinterpretation of the information contained in this book or in its use.