CIDECT

CONSTRUCTION WITH HOLLOW STEEL SECTIONS 7

DESIGN GUIDE

FOR FABRICATION, ASSEMBLY AND ERECTION OF HOLLOW SECTION STRUCTURES

D. Dutta, J. Wardenier, N. Yeomans, K. Sakae, Ö. Bucak, J. A. Packer

TÜV-Verlag

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CONSTRUCTION WITH HOLLOW STEEL SECTIONS

Edited by: Comité International pour le Développement et l'Etude de la Construction Tubulaire

Authors: Jaap Wardenier, Delft University of Technology
Dipak Dutta, Technical Commission of CIDECT
Noel Yeomans, Chairman CIDECT Joint and Fatigue Working Group
Jeffrey A. Packer, University of Toronto
Ömer Bucak, University of Munich
K. Sakae, Nippon Steel Metal Products

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Preface

In principle, the procedures for fabrication, assembly and erection of hollow section structures are the same as for structures in conventional open sections. However, there are a number of special properties and characteristics of hollow sections, which are to be accounted for in order to attain not only technical integrity but also economy. Due to their closed form, hollow sections necessitate special requirements for the processing of fabrication, assembly and erection of a structure, which have to be fulfilled to be able to compete with open sections as well as with concrete successfully. These methods as well as the equipment to carry them out have been described in this handbook explaining their merits and demerits in various design conditions. The aim is to demonstrate to designers the importance of proper initial conception of a design with hollow sections and show fabricators and site contractors the special qualities inherent to hollow sections while performing the jobs in their own fields.
This design guide is the seventh in a series, which CIDECT has published and also will publish in the coming years:

1. Design guide for circular hollow section (CHS) joints under predominantly static loading
2. Structural stability of hollow sections
3. Design guide for rectangular hollow section (RHS) joints under predominantly static loading
4. Design guide for structural hollow section columns exposed to fire
5. Design guide for concrete filled hollow section columns under static and seismic loading
6. Design guide for structural hollow sections in mechanical applications
7. Design guide for fabrication, assembly and erection of hollow section structures
8. Design guide for circular and rectangurlar hollow section joints under fatigue loading (in preparation)

We express our sincere thanks to Dipak Dutta, the main author, and chairman of the CIDECT Technical Commission till end 1994, and Prof. Dr. Jaap Wardenier of the Delft University of Technology, the Netherlands, Mr. Noel Yeomans of British Steel Tubes & Pipes, United Kingdom, Mr. Kazumi Sakae of Nippon Steel Metal Products, Japan, and Prof. Dr. Ömer Bucak of the "Fachhochschule" Munich, Germany, for their contributions and comments. We are also thankful to Dr.-Ing. Reinhard Bergmann of the University of Bochum, Germany, for his contribution to the chapter on "concrete filled hollow section columns", and Prof. Dr. Jeff Packer of the University of Toronto, Canada, for his contribution to the chapter on "blind bolting (Huck Ultra-Twist)". Further, we thank CIDECT member firms for their support.

Etienne Bollinger
Chairman of the
Technical Commission (1995-1997)
CIDECT

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1 Introduction

The modern approach to fabrication-led design is the key to the realisation of optimum construction of structural steelwork [12]. In order to obtain a technically secure, economic and architecturally pleasing structure, both the architect and design engineer must, from the very beginning, be aware of the effects of their design decisions on the fabrication, the assembly and the erection of the structure.

The architects, the design engineers, the fabricators and the erectors are experts in their own particular fields, but traditionally have worked separately. The architect and the design engineer are responsible for the conceptual lay-out, the sizing of the members and, perhaps, some initial detailing of the joints. All of which are generally aimed at the reduction of the material weight of the structure, often with little thought to the fabrication, assembly and erection.

The lack of communication between the various disciplines and subsequently, an inadequate interaction between them often leads to a situation where the impact of the design on the fabrication and erection, and vice-versa, is not properly taken into account. The practice of designing for minimum material weight is very often counter productive as an overall solution, because of the additional costs caused by complex fabrication and site erection imposed by the initial conceptual design. This can only be avoided by an effective dialogue between all of the disciplines involved, with each having some knowledge of the others' requirements for the realisation of a satisfactory and cost effective structure.

This concept of fabrication-led design is of great importance to designs in structural hollow sections, where, unlike bolted open section structures, the members are generally directly welded to each other and member sizing, therefore, has a direct effect on the joint capacity.

For a structure with connections between open and hollow sections made by bolting using gusset or head plates, a designer calculates and selects the member sizes suitable to transfer the applied loads independently of the detail design requirements of the connection. The fabricator is responsible for the final connection design, which he carries out based on his expert knowledge and experience regarding fabrication (Fig. 1.1).

In welded hollow section constructions with joints, where gusset plates are dispensed with in order to save fabrication costs and often to retain architectural attractiveness, members are directly welded to one another (Fig. 1.2). As the strength of the connection is no longer independent of the geometry and strength of the members, the joint performance needs to be considered by the designer himself at the time member sizes are determined. It is therefore important that the designer considers the joint behaviour right from the beginning. Designing members of, for example, a lattice girder based on the member loads only, may result in undesirable stiffening of joints afterwards. This means that the designer, due to the necessity of accounting for the joint strength along with the selection of the hollow section chord and bracing members, has to choose them in such a way that the main governing joint parameters e.g. diameter or width ratio, wall thickness ratio, chord diameter- or width-to-wall thickness ratio, gap between bracings, overlap of bracings and angle between bracing and chord axes, provide an adequate joint strength.

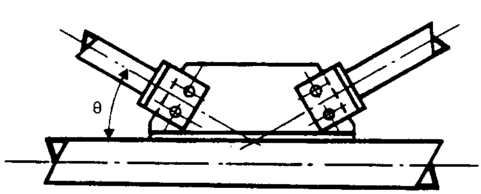


Fig. 1.1 – Bolted hollow section connection with gusset plates

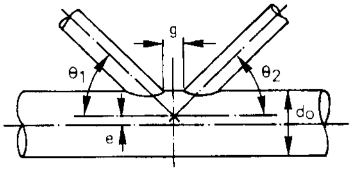


Fig. 1.2 – Welded connection with hollow sections directly welded to one another

As a consequence, the designer at the initial phase has to give thought to the fabrication of joints and the degree of repetition of detailing for optimising the overall cost of a structure made of circular and rectangular hollow sections.
A properly designed steel construction using structural hollow sections, taking into account all of the foregoing, will nearly always be lighter in terms of material weight than a similar construction made with open section profiles. As a result, although structural hollow sections are more expensive than open section profiles on a per tonne basis, the overall weight saving that can be gained by using them will very often result in a much more cost effective and therefore economic construction.
As an aid to the people involved in the construction of hollow section structures, the fabrication, the assembly and the site erection procedures specific to them have been described in this handbook. Particularly, they account for the shapes and sizes of the sections as well as the chemical compositions and physical properties of the steel grades and give recommendations discussing the merits and demerits related to application to various types of constructions.

2 Structural hollow steel grades and dimensional tolerances

The steel grades, dimensions and dimensional tolerances of circular hollow sections (CHS) and rectangular hollow sections (RHS), which includes square, are specified in various national and international standards. However, other hollow profiles such as triangular, hexagonal, octagonal and flat ovals can be produced by some manufacturers, but their availability is dependent upon the size of orders and there are generally no standards for them.
Structural hollow sections can be manufactured with the steel in either a hot or cold condition, and are specified as either "hot finished" and "cold formed" respectively. They can also be either welded with a longitudinal seam weld or seamless. Cold formed hollow sections are always welded, but hot finished hollow sections, athough mostly welded, can be seamless. It is always necessary for the designer to specify if the material is hot finished or cold formed[1]. This is because, although the mechanical properties of the steel and the section size and thickness may be the same, the nominal dimensional properties (area, section moduli, etc.) can be lower for cold formed sections than those for their hot finished equivalent due to the difference in corner radii.

2.1 Steel grades

On an international basis the steel grades are specified by the International Standards Organisation (ISO) in the following standards:

ISO 630 Structral steels[2]
ISO 4951 High yield strength steel bars and sections
ISO 4952 Structural steel with improved corrosion resistance

It should be noted that in some countries quite different national standards may apply and material manufactured in these may not conform to the ISO specification.
Appendix A shows the chemical compositions of the structural steels given in ISO 630 [38], while their mechanical properties are contained in Table 2.1.
The chemical composition and mechanical properties of cold formed hollow sections comply with those recommended by ISO 630 [38]. According to ISO 4019 [39], which mainly deals with the dimensions and sectional properties of cold formed hollow sections, the effects of the cold work in modifying the mechanical properties of the steel shall be taken into account when assessing the mechanical properties of the cold formed section.
In the course of the European product harmonization, the chemical composition and the mechanical properties of non-alloy and fine grain steels for the hot finished (HF) and cold formed (CF) structural hollow sections (circular, square and rectangular) have been standardized in EN 10210-1 [62] and EN 10219-1 [64] respectively. The chemical compositions are shown in Appendix B, while Table 2.2 contains the mechanical properties. Although the steel designations in the CEN (European) standards differ from those in ISO, the chemical compositions and mechanical properties are nearly identical.

---

1) Cold formed hollow sections with subsequent heat treatment to obtain equivalent metallurgical conditions to those obtained by normalizing rolling are deemed to meet the requirements of the standard for hot finished hollow section [62]

2) It applies to steel plates with thicknesses of 3 mm and over, wide strip in coils wider than or equal to 600 mm wide, and greater than 6 mm in thickness, wide flats, bars and "hot-rolled" (synonymous with "hot-finished") sections including hollow sections generally used in the as-delivered condition and normally intended for bolted, riveted or welded structures.

Table 2.1 – Mechanical properties of structural steels acc. to ISO 630 [38].

Steel designation	Quality	Min. yield strength (N/mm^2)			Tensile strength (N/mm^2)	Min. elongation $L_0 = 5.65\sqrt{S_0}$ %	Impact test strength	
		$t \leq 16$ mm	$t > 16$ mm $t \leq 40$ mm	$t > 40$ mm $t \leq 63$ mm			Test temp. (°C)	Energy min. (joule)
Fe 360	B	235	225	215	360 – 460	25	+ 20	27
	C	235	225	215		25	0	27
	D	235	225	215		25	– 20	27
Fe 430	B	275	265	255	430 – 530	22	+ 20	27
	C	275	265	255		22	0	27
	D	275	265	255		22	– 20	27
		$t \leq 16$ mm	16 – 35 mm	35 – 50 mm				
Fe 510	B	355	345	335	490 – 630	21	+ 20	27
	C	355	345	335		21	0	27
	D	355	345	335		21	– 20	27

Table 2.2a – Mechanical properties of hot finished and cold formed structural hollow sections of non-alloy steels acc. to EN 10210-1 (HF) and EN 10219-1 (CF)

Steel designation	Min. yield strength (N/mm^2)			Tensile strength (N/mm^2)		Min. elongation in % $L_0 = 5.65\sqrt{S_0}$				Impact strength	
	$t \le 16$ mm	$t > 16$ mm $t \le 40$ mm	$t > 40$ mm $t \le 65$ mm	$t < 3$ mm	$t \ge 3$ mm $t \le 65$ mm	$t \le 40$ mm	$t > 40$ mm $t \le 65$ mm	$t \le 40$ mm	$t > 40$ mm $t \le 65$ mm	Temp. (°C)	Energy (joule)
						Longitudinal		Tranverse			
S 235 JRH + HF	235	225	215	360 – 510	340 – 470	26	25	24	23	20	27
S 275 JOH + HF	275	265	255	430 – 580	410 – 560	22	21	20	19	0	27
S 275 J2H + HF										– 20	27
S 355 JOH + HF	355	345	355	510 – 680	490 – 630	22	21	20	19	0	27
S 355 J2H + HF										– 20	27
				identical to the values above		Nominal thickness $t \le 40$ mm*				identical to the values above	
S 235 JRH + CF	235	225*	–			24					
S 275 JOH + CF	275	265*	–			20					
S 275 J2H + CF											
S 355 JOH + CF	355	345*	–			20					
S 355 J2H + CF											

* Thickness over 24 mm is available in CHS only

For cross sectional properties there may be significant diferences between hot finished and cold formed hollow sections, see Appendix C

Table 2.2b – Mechanical properties of hot finished and cold formed structural hollow sections of fine grain steels acc. to EN 10210-1 (HF) and EN 10219-1 (CF)

Steel designation	Min. yield strength (N/mm^2)			Tensile strength (N/mm^2)	Min. elongation in % $L_0 = 5.65\sqrt{S_0}$		Impact strength	
	$t \leq 16$ mm	$t > 16$ mm $t \leq 40$ mm	$t > 40$ mm $t \leq 65$ mm	$t \leq 65$ mm	$t \leq 65$ mm long.	$t \leq 65$ mm trans.	Temp. (^oC)	Energy (joule)
S 275 NH + HF	275	265	255	370 – 510	24	22	– 20	40
S 275 NLH + HF							– 50	27
S 355 NH + HF	355	345	355	470 – 630	22	20	– 20	40
S 355 NLH + HF							– 50	27
S 460 NH + HF	460	440	430	550 – 720	17	15	– 20	40
S 460 NLH + HF							– 50	27
				$t \leq 40$ mm*	$t \leq 40$ mm*			
S 275 NH + CF	275	265	–	identical to the values above	24		identical to the values above	
S 275 NLH + CF								
S 355 NH + CF	355	345	–		22			
S 355 NLH + CF								
S 460 NH + CF	460	440	–		17			
S 460 NLH + CF								

* Thickness over 24 mm is available in CHS only

For cross sectional properties there may be significant differences between hot finished and cold formed hollow sections, see Appendix C

Various other national standards [42–49, 53–57] contain steel designations, which vary from those given by ISO or CEN. However, the steel specifications are, in general, comparable.

Structural hollow sections can also be produced in special steels with yield strengths of 640 N/mm^2 and higher. In recent times, seamless hollow sections with yield strengths of 770 and 790 N/mm^2 in quenched and tempered fine grain steels have become available. They are mostly applied to construct mobile cranes, where the reduction of the dead weight is of high importance. However, in order to obtain economy in production, an order for a relatively large quantity has to be placed.

It is important to understand the interaction of yield strength f_y, the ultimate tensile strength f_u, the elongation and ductility while selecting appropriate steel grades for particular applications with specific requirements (Fig. 2.1). In most cases, the basis of design is the yield strength of a member, which avoids excessive deformation. There are also other cases e.g. statically indeterminate structures, where the yielding of members or yielding at particular locations in a structure provides redistribution of loads. Sufficient deformation or rotation capacity is necessary in the latter case. An adequate difference between the yield strength and the ultimate tensile strength is specified in some codes in order to ensure that a structure acts in a ductile manner. ENV 1993-1-1 [66] prescribes the following minimum value for the ratio:

$$\frac{f_u}{f_y} \geq 1.2 \quad \text{(based on the nominal values of } f_y \text{ and } f_u\text{)}$$

Ductility is measured by the Charpy V tests, in which a small steel specimen with standardised dimensions and a standardised V-notch is subjected to a shock load in a particular temperature environment. Charpy-V-value represents the minimum failure energy the test specimen can sustain at a particular temperature, expressed in Joule (Fig. 2.2). These values of the steels standardised by ISO (see Table 2.1) and CEN (see Tables 2.2a and 2.2b) satisfy the requirement of minimum 27 Joules prescribed by Eurocode 3 [66] in its Appendix C.

Another aspect of the characterization of the mechanical properties is described by the strength and ductility of hollow sections while loaded in the thickness direction (Z quality, Fig. 2.3). If a crack occurs, i.e. lamellar tearing, it can be avoided by a low Sulphur content or by joining Sulphur with other elements e.g. Calcium.

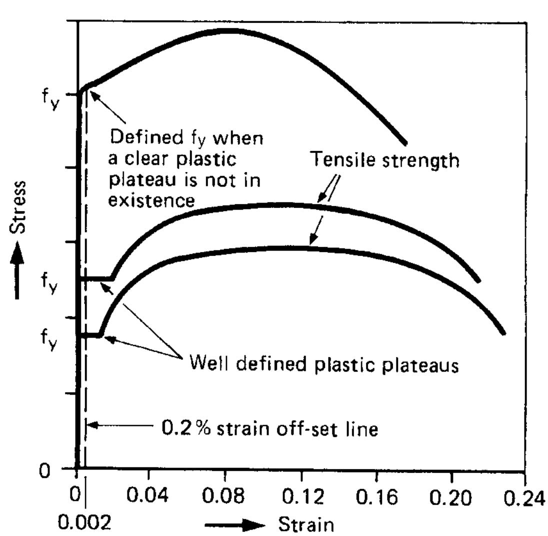


Fig. 2.1 – Tensile stress-strain curve

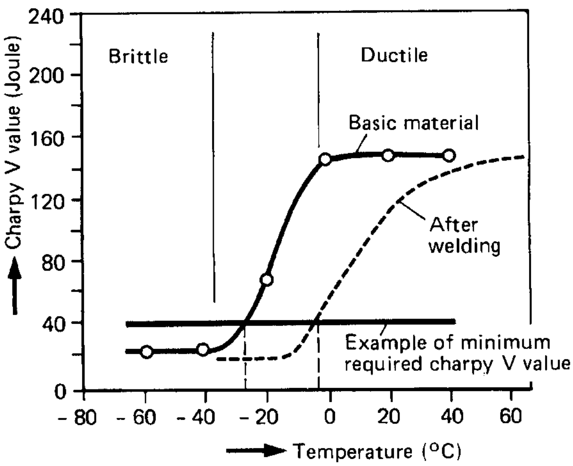


Fig. 2.2 – Charpy V values in relation to temperature

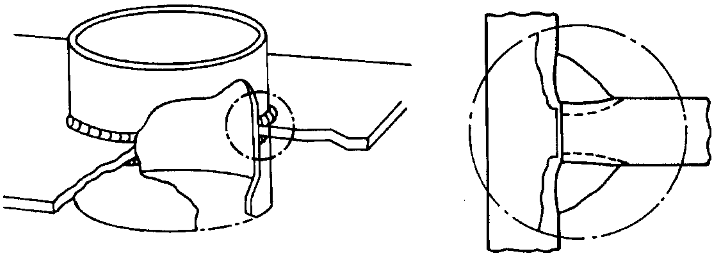


Fig. 2.3 – Lamellar tearing

2.1.1 Welding considerations for the materials

Principally the chemical compositon of a steel grade determines its weldability. In general, the non-alloy steels described above are not heat-treated to increase their strength, which can be affected by welding. They also contain a low percentage of alloy elements making any special measure for welding unnecessary. Decisive for the weldability of the non-alloy steels are their Carbon contents and their steel purity illustrated by the Sulphur, Phosphor and Nitrogen contents.

The fine grain steels obtain their favourable weldability and increased strength of material by a relatively low Carbon content and by the addition of the alloys e.g. Manganese, Silicon, Niobium, Vanadium, Aluminium, Titanium, Chromium, Nickel and Molybdenum respectively. The weldability is improved not only by the lower percentage of Carbon (≤ 0.20%) but also by the fine grain microstructure of the material, which lowers the susceptibility to brittle fracture. The higher strength of these steels are mainly attained by the high Manganese content together with the low Carbon percentage.

In practice, it is simple and also usual to take the Carbon Equivalent Value CEV as the decisive criterium for the weldability of a steel type. On the basis of the ladle analysis, CEV is calculated using the following formula recommended by IIW (International Institute of Welding:

$$CEV = C + \frac{Mn}{6} + \frac{Cr + Mo + V}{5} + \frac{Ni + Cu}{15}$$

The lower the CEV, the better is the weldability, particularly in combination, with the lowest possible Carbon content necessary to reach the required strength of material.
Cold cracking in the welded zone forms the main risk which increases with the increasing product thickness, increasing strength and increasing Carbon Equivalent Value. These cracks may occur due to the following factors in combination:

- the amount of diffusive Hydrogen in the weld metal
- a brittle structure of the heat affected zone
- significant tensile stress concentration in the welded joint

The most effective measure against cold cracking is pre-heat treatment, specially applied to high strength steels with large wall thicknesses. To avoid cold cracking the CEV, the heat input, the thicknesses to be connected and the hydrogen content of the electrodes should be considered to determine the preheating temperature. From the fabrication point of view preheating should be avoided with the consequence that the CEV of the steel and the hydrogen content of the electrode should be low. This is the reason why European application standards and national regulations give restrictions to the CEV.

2.2 Sectional dimensions and properties

The scope of application of a certain profile in a structure depends largely on the sizes of the profile available in the market. Agreements between the manufacturers and the users of the hollow sections regarding the sizes and geometrical tolerances should therefore make the basis for the national and international standards, which regulate the production programmes.
The following ISO standards describe the ranges of the dimensions of the hot rolled (finished) and cold formed hollow sections:

ISO 657/14 [40]	Hot formed structural hollow sections	
	CHS:	External diameter 21.3 to 457 mm
		Wall thickness 2.3 to 40 mm
	RHS (square):	20 × 20 to 400 × 400 mm
		Wall thickness 2 to 25 mm
	RHS (rectangular):	50 × 30 to 500 × 300 mm
		Wall thickness 2.6 to 25 mm

The harmonized European Standards EN 10210-2 "Hot finished structural hollow sections of non-alloy and fine grain steels – tolerances, dimensions and sectional properties" and EN 10219-2 "Cold formed structural hollow sections of non-alloy and fine grain steels – tolerances, dimensions and sectional properties" are now used in most countries of Europe. They show the ranges of the dimensions of the hollow sections and their sectional properties i.e. cross sectional area A, second moment inertia I, radius of gyration i, elastic modulus W_{el}, plastic modulus W_{pl}, torsional inertia constant I_t, torsional modulus constant C_t, Superficial area per metre length A_s and nominal length per tonne. Appendix C contains the formulae used to calculate the geometrical properties.

EN 10210-2	Hot finished structural hollow sections	
	CHS:	External diameter 21.3 to 1219 mm
		Wall thickness 2.3 to 50 mm
	RHS (square):	20 × 20 to 400 × 400 mm
		Wall thickness 2 to 20 mm
	RHS (rectangular):	50 × 25 to 500 × 300 mm
		Wall thickness 2.5 to 20 mm

EN 10219-2	Cold formed structural hollow sections	
	CHS:	External diameter 21.3 to 1219 mm Wall thickness 2 to 30 mm
	RHS (square):	20 × 20 to 400 × 400 mm Wall thickness 2 to 16 mm
	RHS (rectangular):	40 × 20 to 400 × 300 mm Wall thickness 2 to 16 mm

The sizes given in the standards in Canada and U.S.A. [44–46, 49] do not exactly comply with those given in the above mentioned European standards, because their origin lies in the imperial units system (inch, pound).

Fig. 2.4 gives a comparison of the sectional properties of various cross sections, which underlines the superiority of the hollow sections under torsion, compression and multi-axial bending to the open sections. Starting from the identical weights per metre, the specific suitability of hollow sections as structural elements is clearly demonstrated for various types of loading:

– Higher buckling load resistance indicated by larger second moment of area (moment of inertia) about the weak axis I_{min}

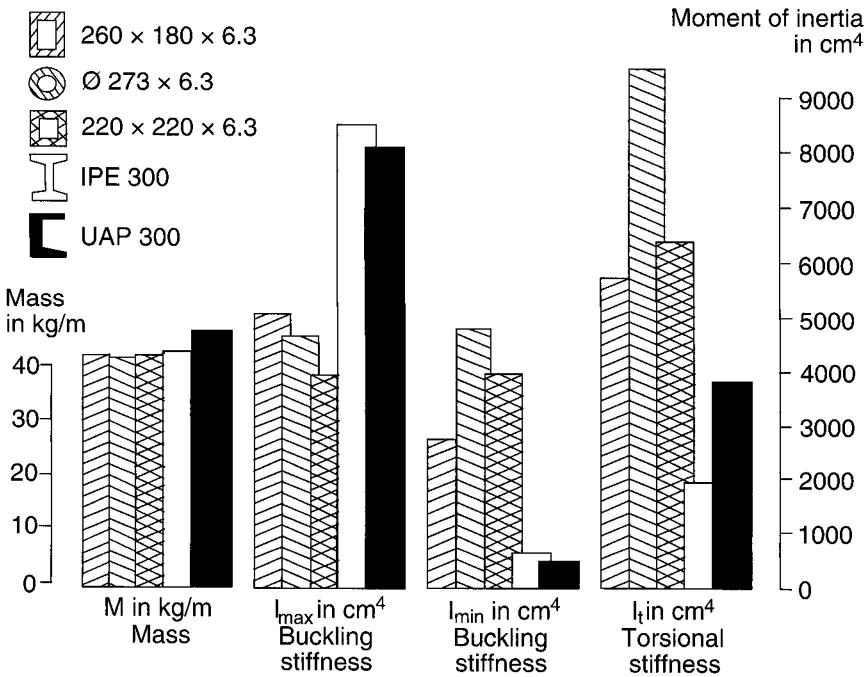


Fig. 2.4 – Comparison of various cross sectional properties

– Under torsion load, the advantage of closed sections, especially circular, are particularly notable. The torsional moment of inertia of hollow sections is 200 to 500 times larger than that of open sections. At identical torque, the torsional angle is only a fraction of that typical of open sections (Fig. 2.5).

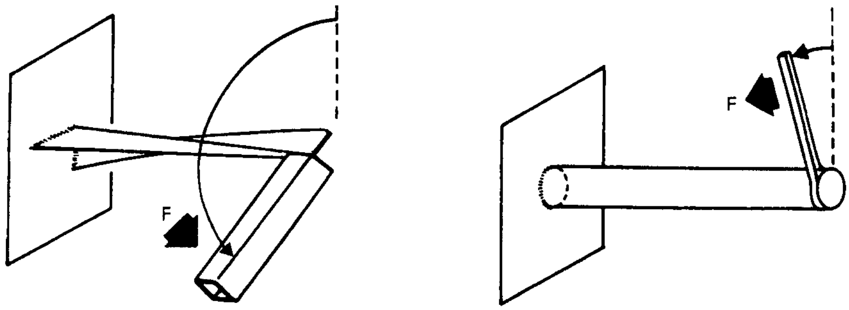


Fig. 2.5 – Hollow sections under torsion

– Under uniaxial bending, the UB and UC sections are more economical than hollow sections due to their higher moment of inertia about the major axis I_{max}. Under biaxial or multiaxial bending, the hollow sections represent optimum cross sections, as they exhibit comparatively high static values about both cross sectional main axes.

Another aspect of the advantage of the application of hollow section under bending load is manifested, when lateral buckling is to be accounted for. The design stress for open sections may be reduced by lateral buckling, while lateral instability is not at all critical for circular hollow sections and for rectangular hollow sections with $b/h > 0.25$ (normally used).
The plastic design of hollow sections under bending leads to higher economy while using the compact sections with the limiting d/t or b/t values given by ENV 1993-1-1 [66].
The production programmes of the manufacturers in different countries may vary from one another depending on the manufacturing methods. Special agreements between the manufacturers and the users regarding the dimensions and tolerances are also possible from case to case. One of the most favourable aspects for the application of hollow sections is, however, the closely stepped range of dimensions, both in side lengths and wall thicknesses, plus the availability of large mill lengths. This enables the design engineer to select the sizes which perfectly fit the application profile both technically and economically.
Not all manufacturers of hollow sections produce all the section sizes shown in the above standards. Some also produce additional section sizes. Much larger sections, which are particularly suited to multi-storey buildings and offshore applicatications, are also available from some manufacturers; an idea of the larger sections available is given below:

Japan:

Cold roll-formed RHS (square) by ERW	Cold press-formed RHS (square) by SAW	
300 × 300 × 6 – 19 mm	300 × 300 × 9 – 22 mm	700 × 700 × 12 – 40 mm
350 × 350 × 9 – 22 mm	350 × 350 × 9 – 25 mm	750 × 750 × 16 – 40 mm
400 × 400 × 9 – 22 mm	400 × 400 × 9 – 32 mm	800 × 800 × 16 – 40 mm
450 × 450 × 9 – 22 mm	450 × 450 × 9 – 36 mm	850 × 850 × 16 – 40 mm
500 × 500 × 9 – 22 mm	500 × 500 × 9 – 40 mm	900 × 900 × 16 – 40 mm
550 × 550 × 12 – 22 mm	550 × 550 × 9 – 40 mm	950 × 950 × 19 – 40 mm
	600 × 600 × 9 – 40 mm	1000 × 1000 × 19 – 40 mm

United Kingdom:
Hot finished RHS (square)

350 × 350 × 19 – 25 mm	550 × 550 × 16 – 40 mm
400 × 400 × 22 – 25 mm	600 × 600 × 25 – 40 mm
450 × 450 × 12 – 32 mm	650 × 650 × 25 – 40 mm
500 × 500 × 12 – 36 mm	700 × 700 × 25 – 40 mm

2.2.1 Dimensional tolerances

Tables 2.3 and 2.4 contain comparisons of the dimensional tolerances of the circular and rectangular (including square) hollow sections given in the CEN [63, 65] and ISO [39, 40] standards. They show very small deviations from one another. However, considerable variations do exist between the tolerances given in various national standards applied in different countries. This is due to the difference of the mass and length tolerances depending on the manufacturing facilities in various mills.
It is worth mentioning that the dimensional tolerances of hollow sections are in general lower than those for open sections.

Table 2.3 – Comparison of dimensional tolerances for circular hollow sections according to ISO [39, 40] and CEN [63, 65] standards

	ISO 657-14 [40] hot	ISO 4019 [39] cold	EN 10210-2 [63] hot	EN 10219-2 [65] cold
outside dimensions	± 1% of diameter with a minimum of ± 0.5 mm	same as [40]	± 1% of diameter with a minimum of ± 0.5 mm and a maximum of ± 10 mm	same as [63]
thickness	– 12.5% with a miminum of – 0.4 mm	± 10% with a minimum of ± 0.2 mm, outside the weld area	– 10%[1)2)]	for D ≤ 406.4 mm T ≤ 5 mm: ± 10% T > 5 mm: ± 0.50 mm for D > 406.4 mm, ± 10% with a maximum of ± 2 mm
mass	+ 10% – 6% on individual lengths + 8.5% – 4% on lots of 10 tonnes	–	± 6% on individual lengths	same as [63]
straightness	0.2% of total length	same as [40]	0.2% of total length	same as [63]
length (exact)	for ≤ 6000 mm, + 10 mm – 0 mm for > 6000 mm, + 15 mm – 0 mm	same as [40]	for ≥ 2000 mm to 6000 mm, + 10 mm – 0 for > 6000 mm, + 15 mm – 0	for < 6000 mm, + 5 mm – 0 for > 6000 mm to ≤ 10000 mm, + 15 mm – 0 for > 10000 mm, + 5 mm + 1 mm/m – 0

1) The positive deviation is limited by the tolerance on mass.
2) For seamless sections thicknesses of less than 10% but not less than 12.5% of the nominal thickness may occur in smooth transitional areas over not more than 25% of the circumference.

Table 2.4 – Comparison of dimensional tolerances for square and rectangular hollow sections according to ISO [39, 40] and CEN [63, 65]

	ISO 657-14 [40] hot	ISO 4019 [39] cold	EN 10210-2 [63] hot	EN 10219-2 [65] cold
outside dimensions	± 1% with a minimum of ± 0.5 mm	same as [40]	± 1% with a minimum of ± 0.5 mm	H, B < 100 mm, ± 1% with a minimum of ± 0.5 mm 100 ≤ H, B ≤ 200 mm, ± 0.8% H, B > 200 mm, ± 0.6%
thickness	– 12.5% with a miminum of – 0.4 mm	± 10% with a miminum of ± 0.2 mm, outside the weld area	– 10% 1)2)	T ≤ 5 mm: ± 10% T > 5 mm: ± 0.5%
mass	+ 10% – 6% on individual lengths + 8.5% – 4% on lots of 10 tonnes	–	± 6% on individual lengths	± 6% on individual lengths
straightness	0.2% of total length	same as [40]	0.2% of total length	0.15% of total length
length (exact)	for ≤ 6000 mm, + 10 mm – 0 mm for > 6000 mm, + 15 mm – 0 mm	same as [40]	for ≥ 2000 mm to ≤ 6000 mm, + 10 mm – 0 for > 6000 mm, + 15 mm – 0	for < 6000 mm, + 5 mm – 0 for > 6000 mm to ≤ 10000 mm, + 15 mm – 0 for > 10000 mm, + 5 mm + 1 mm/m – 0
squareness of sides	$90^o \pm 1^o$	$90^o \pm 2^o$	$90^o \pm 1^o$	$90^o \pm 1^o$

	ISO 657-14 [40] hot	ISO 4019 [39] cold	EN 10210-2 [63] hot	EN 10219-2 [65] cold
outside corner radius	2 T to 3 T (= r_{max})	3 T (= r_{max})	3 T maximum at each corner	for T ≤ 6 mm, 1.5 T to 2.4 T for 6 < T ≤ 10 mm, 2.0 T to 3.0 T for T > 10 mm, 2.4 T to 3.6 T
concavity/convexity	± 1%	same as [40]	1%	max 0.8% with a minimum of 0.5 mm
twist	2 mm + 0.5 mm/m length	same as [40]	same as [65]	2 mm plus 0.5 mm/m length

1) The positive deviation is limited by the tolerance on mass.
2) For seamless sections thicknesses of less than 10% but not less than 12.5% of the nominal thickness may occur in smooth transition areas over not more than 25% of the circumference.

3 Structural hollow section fabrication methods

As described in chapter 2, structural hollow sections have a very advantageous strength to weight ratio when compared to open section profiles, such as I-, H-, C- and L-sections and, because of their lower external area, they also require a much smaller weight of protection material, whether they are fire or corrosion coatings. As a result, construction with structural hollow sections can often result in the most economic form of structure, when the overall reduced weight and cost of protection and maintenance are taken into account.

Various procedures can be used in the fabrication of tubular structures. They include cutting (sawing or flame cutting), flattening, bending, bolting, welding and nailing and these will be discussed in the following sections of this chapter.

Several jointing techniques are available, such as welding, bolting and adhesive bonding. Welding is the most often used method for building up subassemblies or modules in the fabrication shop, because, due to the closed nature of a hollow section one cannot usually get to the inside of the section to tighten up bolts and nuts. Adhesive bonding would also be appropriate for the same reason, but, at the present moment of time, it has not been proved to be economically feasible or structurally consistent from the strength point of view [13].

The joining together of subassemblies or modules on site is usually carried out by welding or by bolting; but, because of the relative costs, bolting is often the preferred method. Until recently, bolting could be difficult because of the closed nature of the hollow section. But now several blind bolting systems have been developed [20–23, 33, 34] and they can be very useful especially for beam to SHS column type connections.

As for all steel structures, the fabrication of hollow section structures in workshops should preferably be organized in such a way that the material will pass through a one-way system from receipt to final dispatch. Prior to the start of the actual fabrication procedure, the hollow sections as building elements are to be taken into temporary stock, where they can be easily identified and moved. In modern factories computerized records hold details of member sizes, lengths, steel grades and qualities and the hollow sections to be used for a particular structure are specified by an identification mark.

After the material is transported from the temporary storage in stacks to the fabrication shops by a conveyor or a lifting device, the following steps take place in general:

a. Marking
b. Cutting to length by sawing or flame cutting
c. Flattening (if necessary)
d. Bending (if necessary)
e. Edge preparation for welds (for welded structures) This can also be done together with b.
f. Drilling holes (for bolted structures)
g. Welding or bolting or combination of both, assembling the members or constructional parts
h. Shot blasting
   This may also be done prior to g., as shot blasting may be difficult after the assembly, especially for large structures.
i. Finishing with primer coating (two or more layers depending on requirements)
j. Painting for protection against external corrosion or with intumescent paints for protection against fire or combinations of various steps, should be taken into consideration.

a. and b. can be combined and if flame cutting is used, cutting to length can be easily combined with other cut-outs. For members to be welded together, the end cutting should preferably include the end preparation or bevelling (if required) for welding. Measuring of the actual dimensions (automatic tolerance compensation) is essential for obtaining the required cut-out and/or bevelling.

In the case of welding end plates or cleats to beams or columns, the conveyor system should be such that this can be easily incorporated into the fabrication flow, e.g. parallel. The same applies to bending or straightening of hollow sections.

3.1 Cutting

The fabrication of a structure starts in general with the end preparation of the members, which involves primarily the various processes of cutting. In the case of structural hollow sections, the methods, which are most frequently applied, are sawing and flame cutting. In light fabrications, often a simultaneous cutting and crimping are carried out in one operation by means of a punch in a press. Depending on the required shape of the end of a member, the operation involves square cutting, mitre cutting, profile shaping and cropping.

3.1.1 Plane cut by sawing

Sawing is mainly used for the end preparations that fit into single planes, which is specially the case for intersection surface of RHS joints. This applies whether the cuts are square or at an angle.

The cutting tool is either a heavy duty circular saw with hydraulic feed or a heavy band saw or a power hacksaw. For further details, see [6].

It is also possible to affect double cutting operation with a swivel head cutter (Fig. 3.1). Further, cutting plants capable of operating simultaneously at both ends of hollow sections have been developed to give greater output. Special attention is needed here to avoid reciprocal twist.

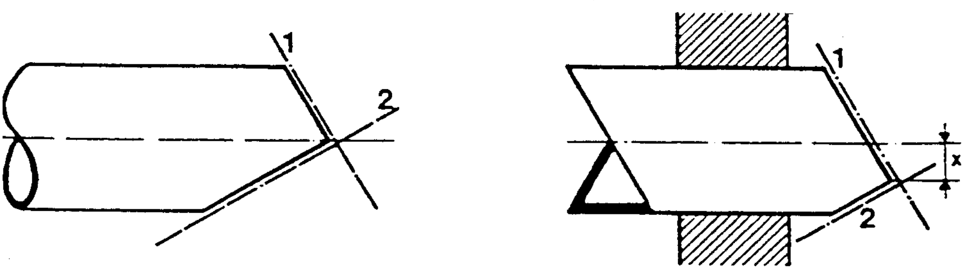


Fig. 3.1 – Double cut of hollow section

The direct joining of circular hollow sectons necessitates a "profiled" cut, often referred to as a "saddle" (Fig. 3.2). The profiled multiplanar intersection curves can, however, be substituted by a number of plane cuts (Fig. 3.3) using a sawing procedure, depending on the relative diameters of the tubes used in the joints. It is necessary to know about the gap between the ends of the bracings and the chord surface, so that this can be bridged by welding.

The following parameters govern the size of the weld gap between the ends of the bracing and the chord surface:

1. Number of plane cuts
2. Ratio of the diameter of the bracing to the diameter of the chord, $\frac{d_{1,2}}{d_0}$
3. Wall thickness of the bracing, $t_{1,2}$
4. Angle of inclination of the bracing axis to the chord axis, $\theta_{1,2}$

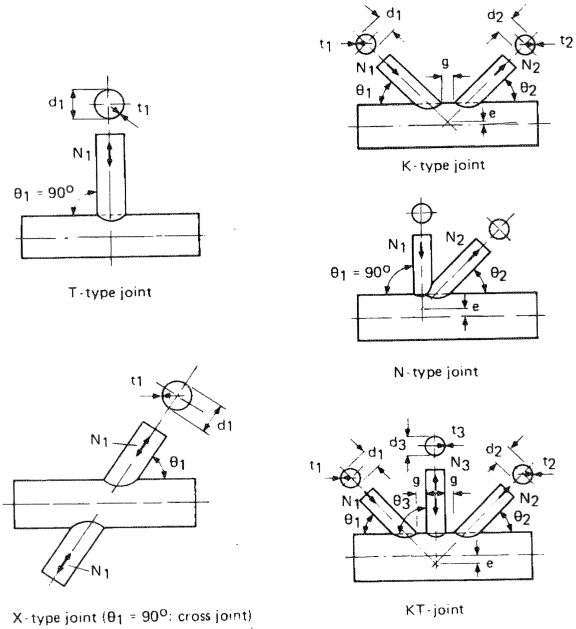


Fig. 3.2 – Welded joint types in CHS

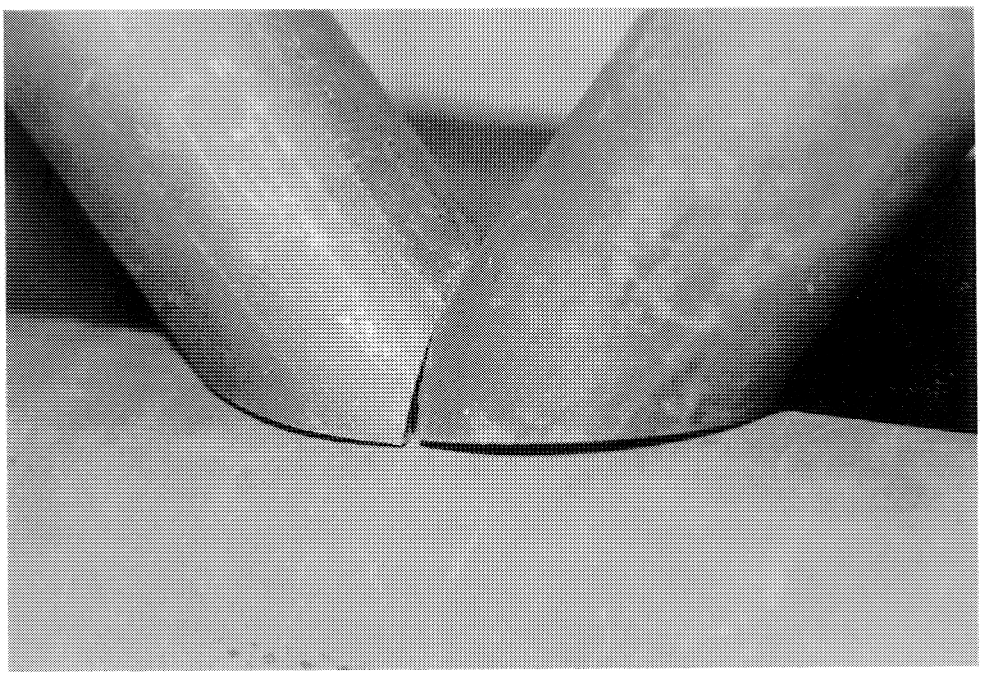


Fig. 3.3 – Node manufactured by plane cuts by sawing before welding (a plate is to be inserted between bracing ends for convenient welding)

The simplest procedure is the one with a single cut. This can however only be applied to joints with very small $d_{1,2}/d_0$ ratios.

When a CHS bracing member is joined to a CHS chord with a substantially larger diameter (Fig. 3.4), the former can only be cut flat at the end, provided that

$g_1 \leq t_r$

with t_r being the smaller of the two values t_0 and $t_{1,2}$. A further condition, which is of more general nature, is:

$g_2 \leq 3$ mm

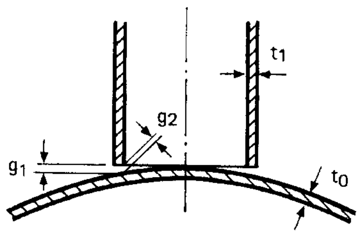


Fig. 3.4 – CHS joint with a single plane cut at the bracing end

Table 3.1 shows the recommended limiting combinations of bracing and chord diameters for the joint given in Fig. 3.4 under the condition $g_2 \leq 3$ mm.

Table 3.1

d_0 mm	d_1 mm
33.7	26.9
42.4	26.9
48.3	26.9
60.3	33.7
76.1	33.7

d_0 mm	d_1 mm
88.9	33.7
101.6	42.4
114.3	42.4
139.7	48.3
168.3	48.3

Large d_1/d_0 ratios lead to large weld gaps and significant attention has to be paid to welding in order to avoid any negative influence on the load bearing capacity of the joint. The real disadvantage of large weld gaps lies in the high costs for welding both members together.

In these cases, the weld gap can be minimized making an appropriate "profile" cut by means of either

a) two single plane cuts followed by grinding or cutting (shearing off) of the "point" area (see Fig. 3.5)

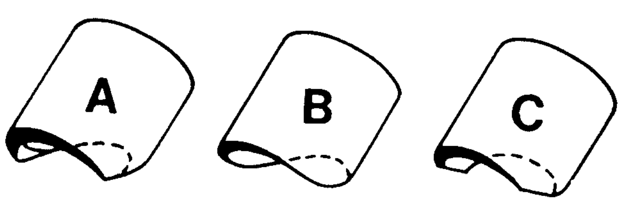


Fig. 3.5 – Operations to smooth down the "Point" area
A: by grinding off the internal angles B: by profile grinding C: by shearing off

or b) two or three successive single plane cuts using the equations for the cutting angles β_g and β_d in Fig. 3.7.

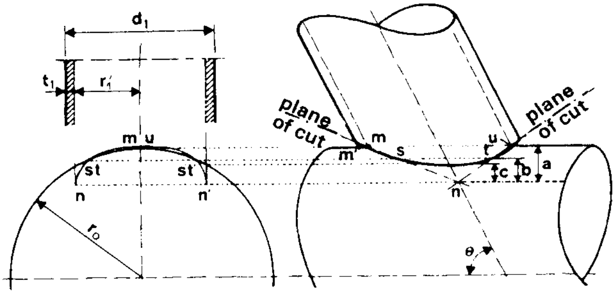
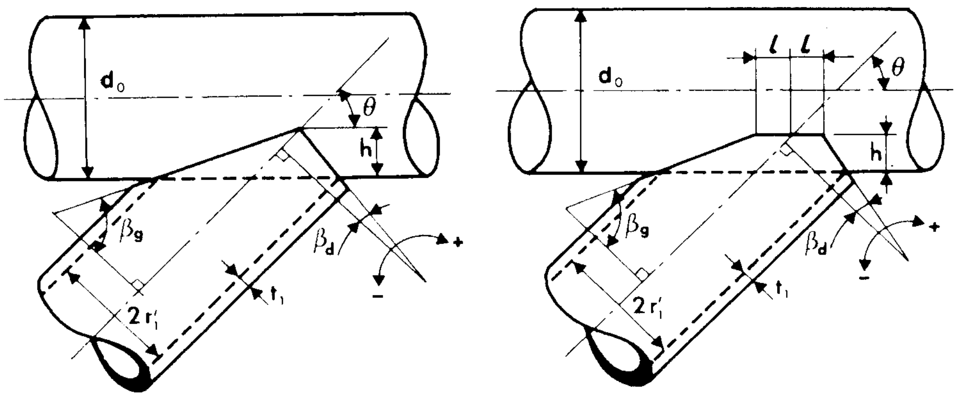


Fig. 3.6 – Plane cuts of bracings ends for CHS joints (Method A)



2 cuts

$$h = \frac{d_0}{2} - \sqrt{\frac{d_0^2}{4} - r_1'^2}$$

$$\alpha_g = \mathrm{arctg}\left(\frac{h \sin\theta}{r_1' + h\cos\theta}\right)$$

$$\alpha_d = \mathrm{arctg}\left(\frac{h \sin\theta}{r_1' - h\cos\theta}\right)$$

3 cuts

$$\ell = \sqrt{r_1'^2 - (r_1' - t_1)^2}$$

$$h = \frac{d_0}{2} - \sqrt{\frac{d_0^2}{4} - (r_1' - t_1)^2}$$

$$\alpha_g = \mathrm{arctg}\left(\frac{h \sin\theta}{r_1' + h\cos\theta - \ell\sin\theta}\right)$$

$$\alpha_d = \mathrm{arctg}\left(\frac{h \sin\theta}{r_1' - h\cos\theta - \ell\sin\theta}\right)$$

2 cuts and 3 cuts: $\beta_g = 90° - \theta + \alpha_g$ $\qquad \beta_d = -90° + \theta + \alpha_d$

Fig. 3.7 – Plane cuts of bracing ends for CHS joints (Method B)

Method A (Fig. 3.6):

"a" in Fig. 3.6 is determined according to the equation

$$a = \frac{r_1'^2}{2r_0 - r_1'}$$

where, r_1' = internal radius of the bracing = $\frac{d_1 - 2t_1}{2}$

r_0 = external radius of the chord = $\frac{d_0}{2}$

The value "a" is constant whatever the angle of inclination θ.

Starting from the point "n" determined by the value "a", the lines "n-m" and "n-u" are drawn. These define the cutting planes, whose inclinations have to be measured. After both cuts are made, the edges are trimmed as required, so that the bracing fits neatly on the chord. The smoothing down of the "point area" can be carried out by either of the three operations shown in Fig. 3.5.

Method B (Fig. 3.7):

This method is valid for two as well as three cuts. "h" in Fig. 3.7 is a constant value irrespective of the angle θ and can be calculated using the formulae given with Fig. 3.7.

The intermediate values α_g and α_d can also be calculated using the corresponding formulae.

The required cutting angles β_g and β_d can then be determined.

The detailed calculation procedure for determining the maximum gap of weld is given in [18].

3.1.2 Flame cutting

This cutting procedure is specially applicable to circular hollow sections for profile cuts, which can be performed either manually or by means of automatic machines. Manual flame cutting is mainly used for cutting on site or for cutting larger sized sections. Automatic flame cutting machines have been developed and perfected by the machine tool producers during the last decades and are usually applied in the workshops. They can cut and profile shape the ends of CHS to any combination of diameters and inclination angles within their ranges.

3.1.2.1 Manual flame cutting

In this procedure the flame cutting torch is held by hand and follows the line of the cut on the hollow section with or without a guide. The path of the cut can be marked directly on the hollow section or on a template such as a thin metal sheet. The cut is located with reference to the heel (A) and toe (B) points (see Fig. 3.8).

The theoretical contact lines are:

- heel point A side, on the outside of the bracing member
- toe point B side, on the inside of the bracing member

It is not necessary to make a chamfer up to 5 mm wall thickness of the bracing member, but for wall thickness more than 5 mm, the edges have to be chamfered for welding.

Fig. 3.9 a – c shows the transition of chamfer on the intersection curve from point to point.

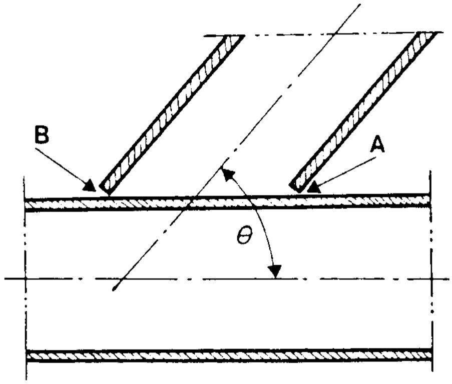


Fig. 3.8 – Reference points, heel A and toe B for profile cut of CHS

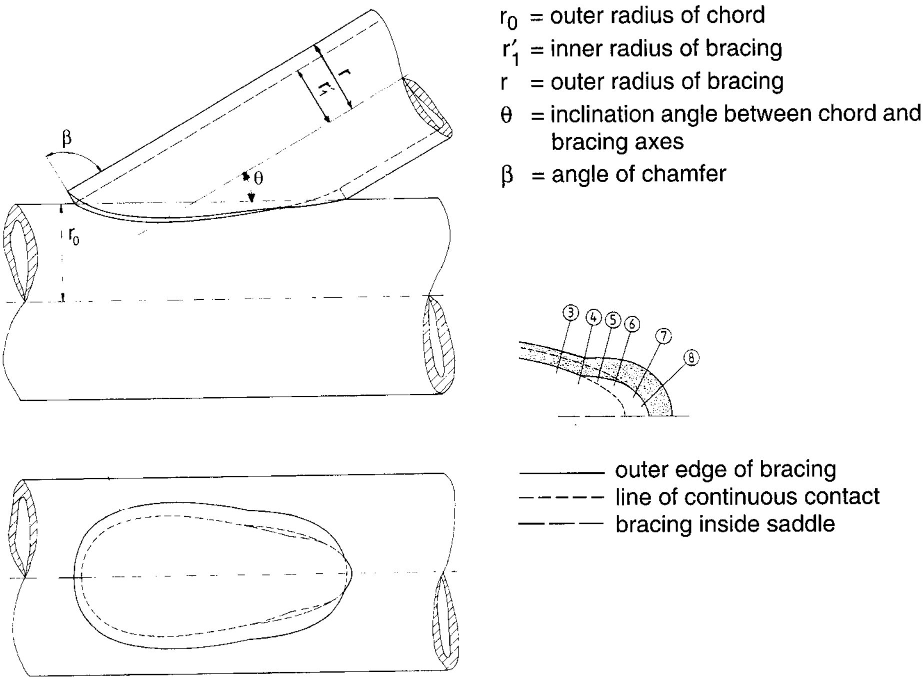
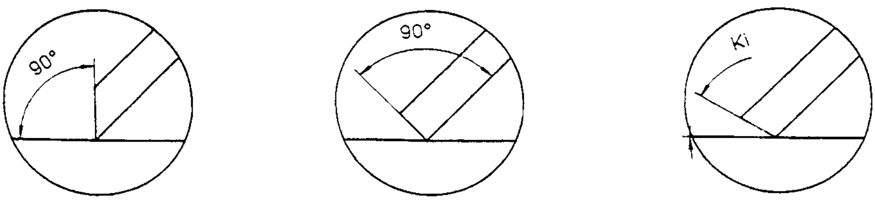


Fig. 3.9a



Version 1: chamfer 90^{o} to the chord

Version 2: chamfer 90^{o} to the bracing

Version 3: variable chamfer

Fig. 3.9b – Welding chamfer at the bracing inside saddle

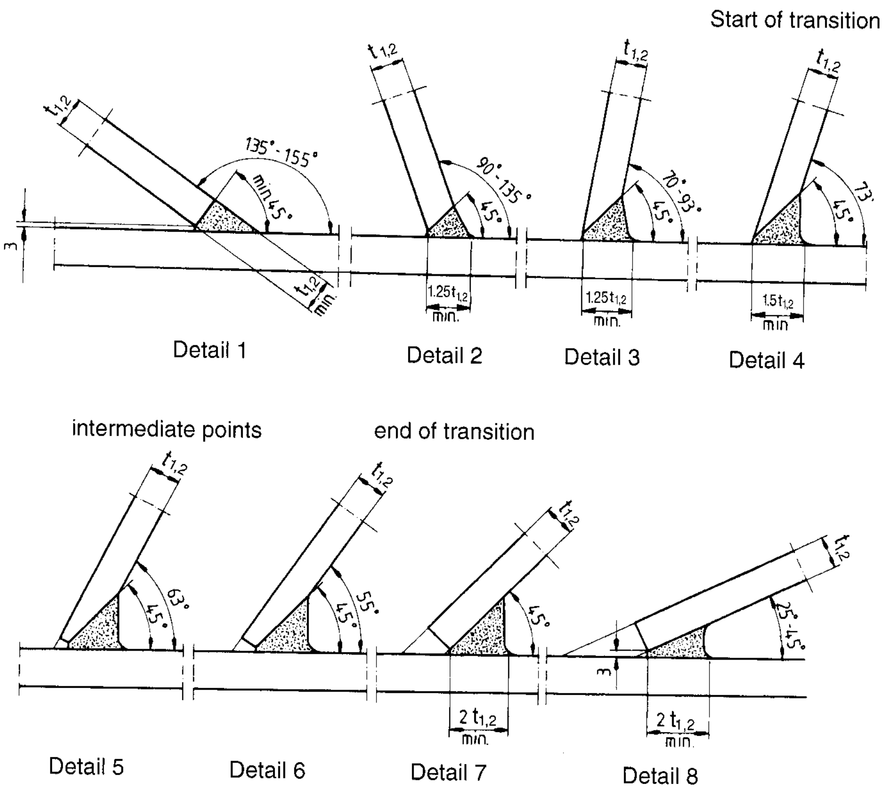


Fig. 3.9c – Welding details

For the manual cutting procedure the chamfering of the edge can be performed with a hand grinder or a flame cutter depending on the cutting quality required.

The following job sequences are followed for manual flame cutting:

- Determination of an intersection curve
- Making a template
- Marking a tube
- Flame cutting by hand
- Making the weld edges correcting the intersection curves for the internal and external diameters
- Final edge correction by means of grinding

The usual procedure for making templates for profile shaping is marking the ends of CHS as given in Fig. 3.10.

Manual flame cutting is not very precise and the cut surface will usually require grinding to ensure precision.

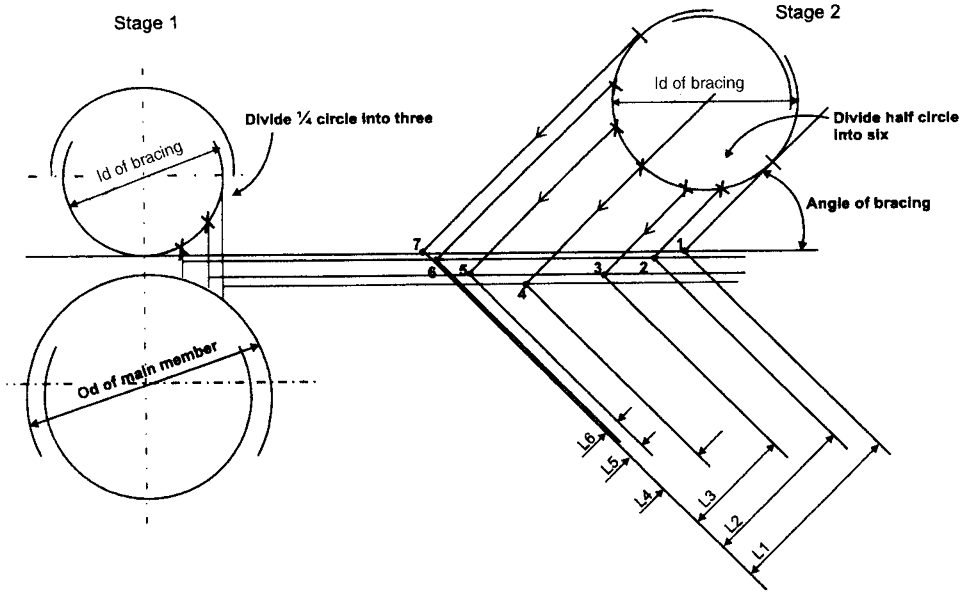


Fig. 3.10a – Making a template for profile shaping of CHS ends

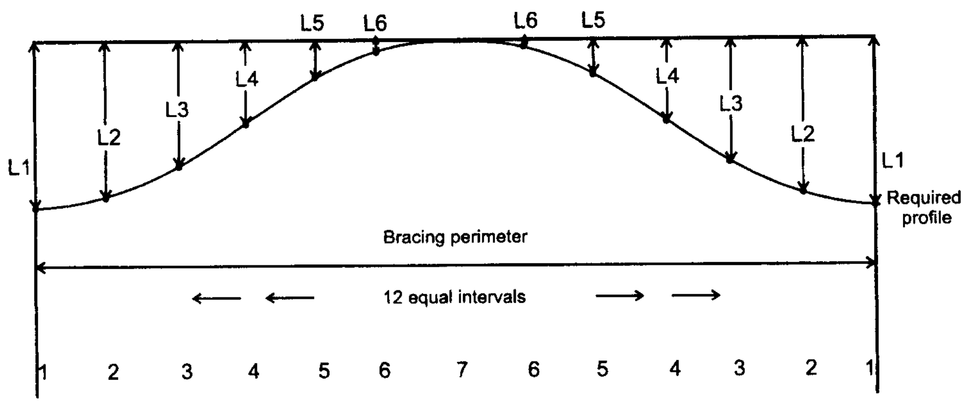


Fig. 3.10b – Making a template for profile shaping of CHS ends

3.1.2.2 Automatic flame cutting

The working principle of automatic flame cutting machines is based mainly on two systems:

1. The burner moves horizontally while the workpiece (CHS) only rotates. The burner also tilts up and down to produce the chamfer.
2. The workpiece (CHS) is stationary while the burner moves in both translation and rotation and also dips to cut the chamfer.

The automatic flame cutting machine of the firm Müller, Opladen, Germany, introduced in the fifties, has pioneered in this field with a machine controlled by a lever arrangement. The adjustment of the levers is easy to make without calculations, since the values for the CHS diameters, the bevel, mitre angles and the eccentricity are direct inputs to the machine. The machine can cut the bevel and mitre intersection curves automatically with its steered flame cutter. It is equipped with a motor drive and a template set-up system, which enables the machine to cut other intersection forms, that cannot be made using the lever system. All intersectional curves can be cut with the required weld chamfer. This can reduce the fabrication costs significantly.

Fig. 3.11 – A computer controlled automatic flame cutting machine

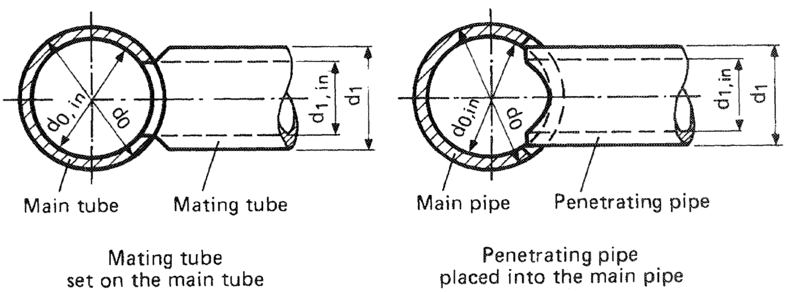


Fig. 3.12 – Concentric and eccentric penetration of bracing and main pipe

Nowadays various computer controlled flame cutting machines are available on the market. Their cutting precision and repeatability are very good compared to either the lever type machine or manual flame cutting. This also renders greater advantage for welding.

The following cutting facilities are possible for the modern machines:

- Single or double mitres
- Concentric or eccentric adjustment
- Concentric or eccentric penetration (see Fig. 3.12)
- Automatic weld penetration
- Other special adjustments

The running procedure is usually as follows:

1. The following inputs are read by the machine:
   - Outside diameter of the main CHS (d_0)
   - Outside diameter of the bracing CHS ($d_{1,2}$)
   - Inside diameter of the bracing CHS ($d'_{1,2}$)
   - Angle of inclination between the axes of the bracing to the main CHS (θ)
   - Opening angle of chamfer (β) for weld
2. After lighting the flame, the material is preheated for plunge cutting. During this process the flame cutter moves automatically a few centimeters beyond the intersection line into the waste metal.
3. Plunge cutting is followed automatically by switching in the selected cutting direction. The flame cutter comes out of the waste part into the given cutting curve and continues the cutting operation.
4. The cutting procedure is completed after a bit more than 360° rotation of the workpiece (CHS).

If plunge cutting does not take place beyond the given intersection curve, defects as shown in Fig. 3.13, may cause weld problems.

Fig. 3.13 – Defects due to wrong plunge cutting

3.1.3 Slotting

Hollow section connections are sometimes made by inserting welded fittings through slots cut out in the hollow sections.
The following types of connections using slots are mostly used:
- Connections with end gusset plates (Fig. 3.14)
- Connections with slots in the body of members (Fig. 3.15).

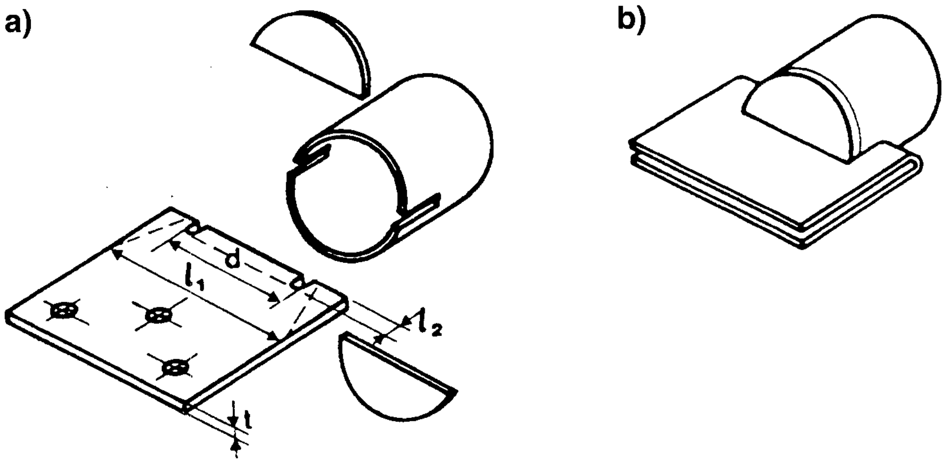


Fig. 3.14 – Slotted connection a) with a flat gusset plate b) with a bent gusset plate

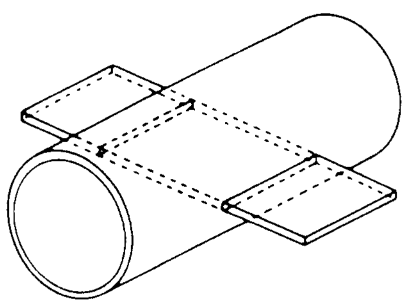


Fig. 3.15 – Slot in the body of a CHS with a flat plate passing completely through it

The slots, as a rule entirely closed by weld seams, should still be sealed to prevent internal corrosion. This is done by suitably shaped stoppers to close the semi-circular openings at the ends of hollow sections. For hot dip galvanised parts, however, openings are required to avoid bursting due to the high pressure in the closed volume of air.
Slots in hollow sections are cut by the following methods:
- Notching, using special blades
- Manual flame cutting
- Semi-automatic flame cutting
- Slotting with milling cutter
- Slotting with abrasive discs

For slotting with a manual flame cutter, it is convenient to drill a hole in the hollow section wall at the end of the intended slot. The diameter of the hole is slightly larger than the width of the slot. Finally the sides are cut with the flaming torch going from the ends towards the hole (Fig. 3.16).

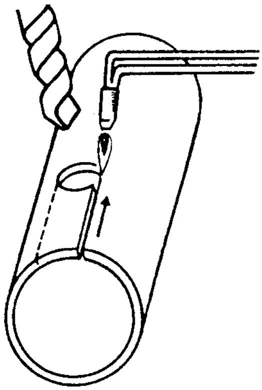


Fig. 3.16 – Cutting a slot with a manual flame cutter

A further method of slotting is to use a milling saw with its head set parallel to the workpiece (Fig. 3.17).

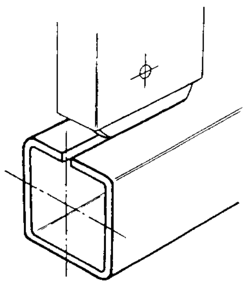


Fig. 3.17 – Cutting a slot with a milling cutter

Manual gas cutting can give rise to problems in downstream operations such as assembly accuracy or weld performance due to lack of accuracy in cut end or in slit width/length. Slit length, as a location for joint stress concentration, requires particularly accurate processing. Nevertheless, where manual processing is unavoidable, the flame cutter has to be guided. Further, stress relief due to heating as well as cutting can lead to section distortion. In order to avoid this, the head end can be left partially uncut until cooling is achieved.

3.1.4 Laser cutting

Laser cutting has been gaining more and more importance in the last decade due to its high quality, accurate performance, flexibility and low fabrication costs. The requirement for any post treatment is very small or none. The running procedure and the control are nearly identical to those of the computer controlled flame cutting machines, but laser beams are used as the cutting medium. Today, normal laser cutting machines can cut the following wall thicknesses for plane geometry without any problem:
- Non-alloy steel up to about 16 mm
- Stainless steel up to about 10 mm
- Auminium up to about 6 mm

The cutting rate can reach up to 10 m/min with a very low tolerance of ± 0.1 mm, which can play an advantageous role in further fabrication steps such as welding. A further quality is given by the heat affected zone (HAZ), which is very small.

However, the disadvantage lies in the high investment costs, which prevent many small and medium sized firms from applying this cutting method.

3.1.7 Plasma cutting

In plasma cutting, a gas (Ar, N_2 or Ar + N_2 or N_2 + H_2) heated by a concentrated electric arc is applied for cutting. In a thin stream the gas hits the workpiece with high speed. Due to the high concentration of energy, the cutting is quicker than with other existing methods and in most cases the cuts are made without any distortion. Quality cuts can be performed within the wall thickness range from 4 to 35 mm, while a thickness up to 45 mm can be cut with reduced quality.

Presently, small compact transportable machines, as well as high capacity installations are available on the market.

3.2 Flattening

Circular hollow sections in joints are often flattened at their ends to avoid the cutting of expensive and complex intersection curves and the preparation of any necessary welding bevels. End flattening of rectangular hollow sections is not a common procedure. In a structure, CHS with flattened ends are joined either by welding or by bolting (see Fig. 3.18 and 3.19) with plane cut end preparation.

Fig. 3.18 – Welded connection with flattened CHS bracing ends

Fig. 3.19 – Bolted connection with flattened CHS bracing ends (in order to increase the bearing capacity of the cross section weakened by the bolt hole, the flattened bracing ends are reinforced by plates welded to the ends)

The flattening can be carried out in hot or cold condition.

For hot flattening, only the area intended for flattening is to be heated up to a temperature range of 750 to 900°C. Heating can be done by electricity, oxyacetylene torch or butane and propane burners. Suitable continuous heaters can be installed for a low investment, when mass production is required (Fig. 3.20).

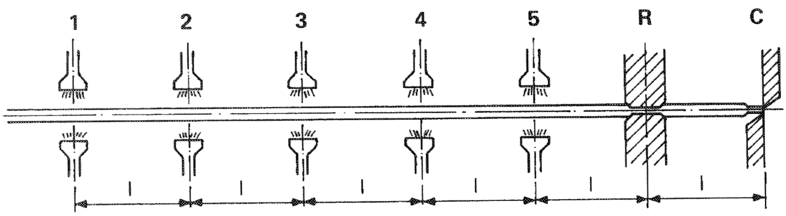


Fig. 3.20 – Design arrangement of heaters for a hot flattening installation

No rigid rule dictates whether hollow sections shall be flattened in hot or cold condition. So, cold flattening, which is relatively simple, quick and less expensive, is often used. The material is plastically deformed during the cold flattening process and deformations occur in both longitudinal and transversal directions, which may produce cracks. When cracks take place, they are at the flattened edges, where the largest strain occurs; a local strain may be over 200% (simple calculations will confirm it). As there is no well defined standard for the flattening procedure, it is recommended to carry out preliminary tests to prove the capability of a particular flattening procedure in case of large batches.

During cold flattening, cracks may occur along the weld seam of a hollow section. They can however be avoided by offsetting the weld seam from the line of extreme deformation.

Further, a proper choice of d/t is necessary related to the flattening process. In general, flattening is easier, when d/t is higher.

Based on the workshop practice as well as required joint strength, various types of flattening are used. The main possibilities are shown in Fig. 3.21.

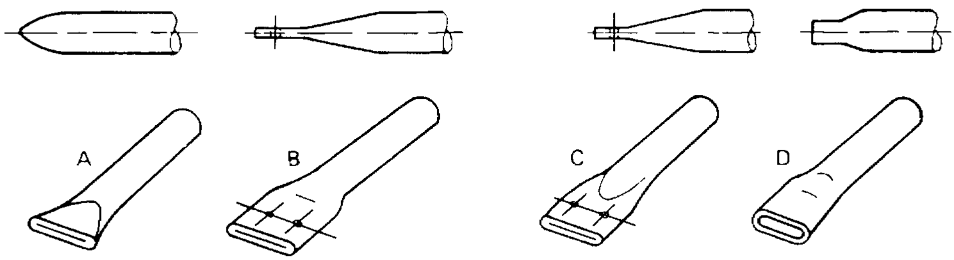


Fig. 3.21 – Flattening types
A. Cropping B. Full flattening C. Flattening with a recessed die D. Partial flattening

The sketches of the flattening tools for various procedures are shown in Fig. 3.22 through 3.26. The shape of the dies determines the inclination and the form as well as the length of the transition. They may initiate cracks, if they are not properly selected in conformity with one another.

3.2.1 Cropping

Cropping is a very economical method, where the cutting and full flattening of the hollow section ends take place in one operation by the same tool, which can be a shear, a guillotine or, for small sections, a notcher (see Fig. 3.22). In this process, the full flattening occurs at the very end.

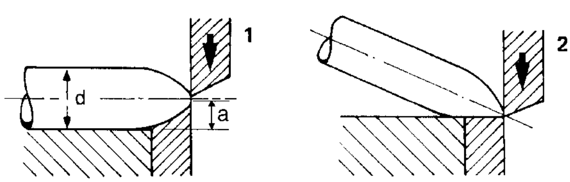


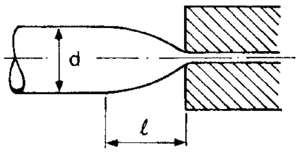
Fig. 3.22 – Sketch of a cropping machine showing working principles

As shown in Fig. 3.22 at 1, the workpiece is placed horizontally, while the fixed lower blade is adjusted so that it projects by a distance nearly equal to the radius of the CHS leading to a symmetrical flattening.

At 2, the fixed blade is at table level, while the CHS is slightly inclined to avoid non-symmetrical flattening. A cropped end is only suitable for welding.

3.2.2 Full flattening

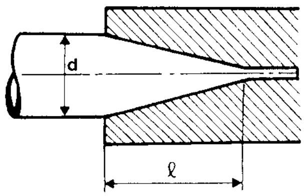
Fig. 3.23 shows the sketch of the die indicating that a relatively long portion is subjected to full flattening in this process. the length ℓ of the transition zone is recommended to be within the range 1.2 d $\leq \ell \leq$ 1.5 d. The edges of the dies are required to be rounded off to avoid any transverse crack.



1.2 d < ℓ < 1.5 d

Fig. 3.23 – Simple device for full flattening of CHS ends

Fig. 3.24 presents a design of the die to shape the transition zone over a greater length ℓ ranging between 1.7 d and 2.2 d.



1.7 d < ℓ < 2.2 d

Fig. 3.24 – Full flattening die for a longer transition zone

A further method of flattening especially applied in Japan is to insert an inner CHS section into the ends of the CHS to be flattened.The outer section is then flattened together with the inner section. This method is mostly adopted for flattening the bracing ends of a CHS joint in order to avoid cracks and strengthen the bracing ends more.

3.2.3 Flattening with a recessed die

In this case, the flattening is carried out in a press with two recessed dies embodying a gradual change of the tubular section (Fig. 3.25). The length of the transition area is often equal to 2d.
This shape is suitable for bolting and more efficient in tension and compression than the simple full flattening dies.
Although the investment cost is higher, for mass production this is compensated by its ease of flattening and lower rate of wear.

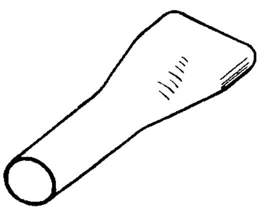


Fig. 3.25 – Flattened CHS with recessed dies

3.2.4 Partial flattening

The flattening operation is so regulated that parallel faces can be obtained by reducing the stroke of the press and introducing a distance piece into the flattened portion (Fig. 3.26).
This type of flattening is restricted to welded joints as demonstrated by the example in Fig. 3.27.

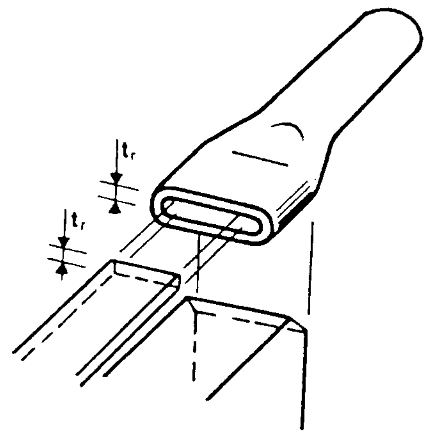


Fig. 3.26 – Sketch of a partial flattening device

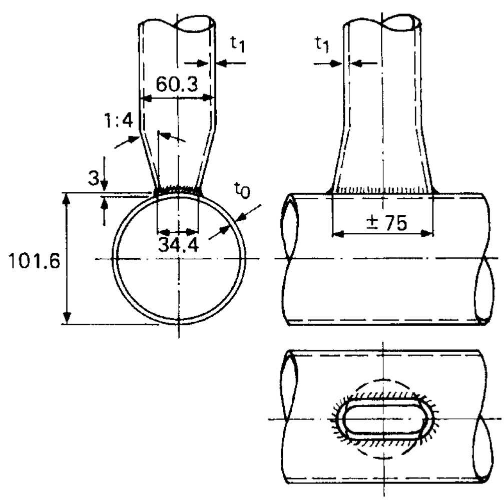


Fig. 3.27 – Configuration of a joint with partially flattened CHS ends

3.3 Bending (arching)

Hollow sections can be bent in either the hot or the cold condition. As the production cost of cold bent hollow sections is lower than that for hot bending, the former is applied normally, while hot bent hollow sections are used in special cases. While deforming a hollow section to give it a permanent curvature, buckling may occur in the compression zone on the inner side and the wall thickness may decrease as a result of tension in the outside zone. These

changes in thickness as well as the possible ovality of a tube should be kept as small as possible.
Bending of hollow sections depends on the following material properties:

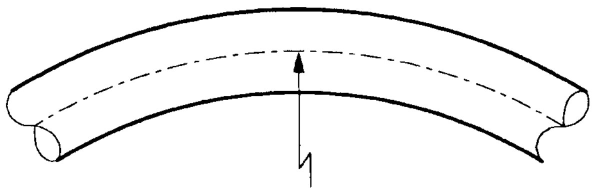
– Yield strength of the steel grade
  The lower the yield strength, the easier is the bending.
– Elongation percentage
  Adequate ultimate elongation percentage plays a vital role for the bending procedure.
– Fine microstructure of the material favours bending.

Further determining factors are the following geometrical parameters:

– ratio $\frac{\text{Diameter of CHS or Depth of RHS}}{\text{Wall thickness of hollow section}}$

– ratio $\frac{\text{Bending radius of curvature}}{\text{Diameter of CHS or Depth of RHS}}$

It is also important to know the absolute dimensions of the hollow section in order to decide whether hot or cold bending should be applied.



In general, bending is carried out in the workshop; however sometimes, especially for small dimensions it is performed on site. There are also large overlapping zones where both cold and hot bending procedures can be applied. It has sometimes to be checked whether heat treatment is necessary after cold bending in order to obtain the initial microstructure of the materials (heat treatment temperature according to the steel grade used).

3.3.1 Cold bending methods for CHS

In the following, several methods are described, which are applied for bending CHS in the cold condition.

3.3.1.1 Cold bending by pressing

The sketch shown in Fig. 3.28 demonstrates the working principle. Setting a hollow section between two fixed rollers, bending is made by the displacement of a central former, which is usually connected to a hydraulic actuator.
This operation may also be performed by keeping the central former still and pushing the side rollers.
This process is usually used for bending 180° arches with a wide range of dimensions. However localized pressing provides low accuracy and poor appearance compared to the mechanical processes given in Chapters 3.3.1.2 through 3.3.1.4.

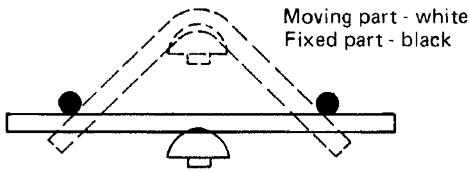


Fig. 3.28 – Cold bending by pressing

3.3.1.2 Cold bending by a "former" box

The working principle shown in Fig. 3.29 describes how the workpiece is forced into a pre-shaped "former" box "A". The "former" box "A" is fixed to a horizontal base. A straight guide box "B" is placed in front of the bent former. The workpiece "C" is forced into guide box "B" and then into the "former" box "A" by means of the actuator "D". The guide box "B" is then transferred to the position "B", so that the operation can be repeated from the other side.

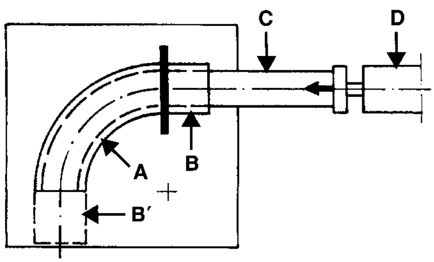


Fig. 3.29 – Cold bending using a "former" box

In order to avoid any damage of the tool, the ends of the workpiece must be provided with a guide plug. Further, lubrication is essential.
The method is only then economical if many bends using the same size of hollow section are required.

3.3.1.3 **Roller bender** (Fig. 3.30)

This tool, where bending is obtained by passing the workpiece through three rollers, is generally preferred by the fabricators of steel structures. All three rollers may be driven, but the central one, which determines the radius, can also be idle.

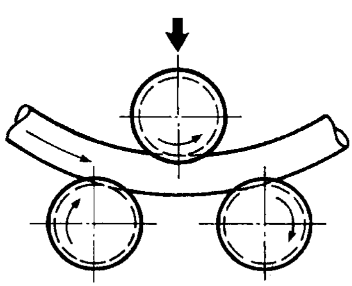


Fig. 3.30 – Cold roller bending with 3 rollers

The rollers must be adapted to the size of the hollow section to be bent. The roller dimensions are therefore in accordance with the sizes of CHS. Four roller benders are also available, in which one of them is idle (Fig. 3.31).

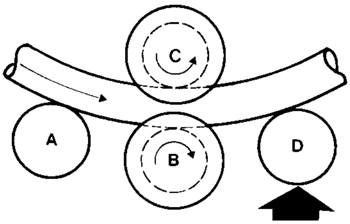


Fig. 3.31 – Roller bender with 4 rollers

For cold bending with roller bender, a bending radius of curvature of about 5 times the external tube diameter is used in practice.

3.3.1.4 Bending by means of mitre cuts (Fig. 3.32)

Usually for large radius bends, approximate curves can be obtained by joining straight sections end to end by welding the ends having been previously cut at an appropriate angle.

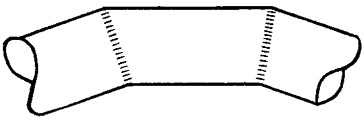


Fig. 3.32 – Mitre cutting and welding

3.3.2 Cold bending methods for RHS

The following methods are generally used for cold bending of RHS:
- Roller bending
- Mitre cuts or V-Notches
- Cold bending by pressing

3.3.2.1 Roller bending

In principle, the process is identical to that given in Section 3.3.1.3, although the results of the bending are different due to the shape and the flat faces of RHS. The inside face will compress, the outside wall will stretch and the side wall will exhibit both behaviours. Mostly the result is the convexity in the two side walls and concavity in the inner surface (Fig. 3.33). There will be some changes in height in the curved regions. Wall thinning and deformations should therefore be checked against the requirements of project application [19].
The bending radii for square and rectangular hollow sections by cold 3-roller bending are listed in Appendix D [19].

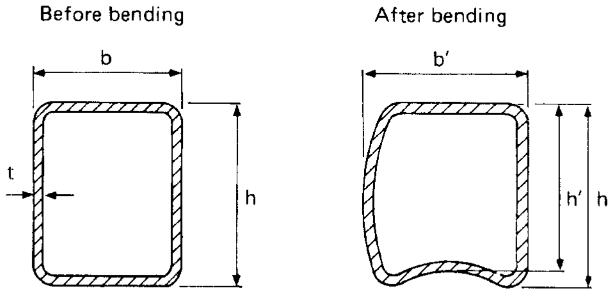


Fig. 3.33 – Typical RHS sections before and after cold roller bending

These have been obtained by extensive tests using a steel grade with the following nominal mechanical properties:

Min. yield strength = 350 MPa

Ult. tensile strength = 450 – 620 MPa

Young's modulus of elasticity = 200000 MPa

The values given in Appendix D can only give an approximate indication, because they are related to the details of the roller bender used to carry out the tests (see Table 3.2). It is therefore recommended to perform a few bending tests with a machine to determine the roller sizes and arrangements before starting the fabrication.

Table 3.2 – Details of the 3-roller bender used to determine the bending radii in Appendix D

Roller bender no.	Roller outer diameter mm		Distance between the vertical axis of the central roller and side rollers mm
	Moving central roller	Fixed roller	
1	430	385	710
2	515	460	1015

3.3.2.2 Mitre cuts or V-notches

Approximate bending with a large radius of curvature can be carried out by mitre cuts as for CHS (Fig. 3.32) described in Section 3.3.1.4.

For smaller RHS dimensions, the bent shape can also be obtained by cutting out V-notches on three of the faces and folding the remaining face. The notches are then closed and the edges are welded together (Fig. 3.34).

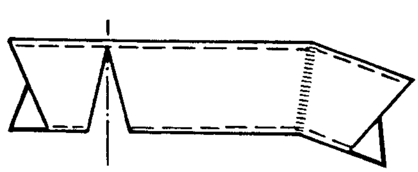


Fig. 3.34 – Obtaining RHS bent form by V-notch cut-out

3.3.2.3 Cold bending by pressing

This method as described in Chapter 3.3.1.1 can also be used for RHS.

3.3.3 Hot bending methods

The following hot bending procedures can be applied:
- Hot bending filling a hollow section with sand (CHS and RHS)
- "Hamburger Rohrbogen" (CHS only)
- Induction bending (CHS and RHS)
- Roller bender (CHS and RHS)
- Cambering (CHS and RHS)

Care must be taken when hot bending cold formed sections since the heat input required to hot bend the sections can result in changes of the mechanical properties.

3.3.3.1 Hollow sections filled up with sand (Fig. 3.35)

Although applicable to both CHS and RHS, this procedure is in general used for bending CHS with large diameters and wall thicknesses in the hot condition.
The hollow section is filled up with sand which is then compressed before heating in order to avoid deformations on the inner side of the workpiece and to keep the ovality as small as possible. The bending zone is then heated up to a temperature of 850°–1100°C and the workpiece is pulled about a template. The procedure takes place stepwise. After the first bent zone is cooled down, the bending of the neighbouring zone is started and so on, till the total required bending is completed.

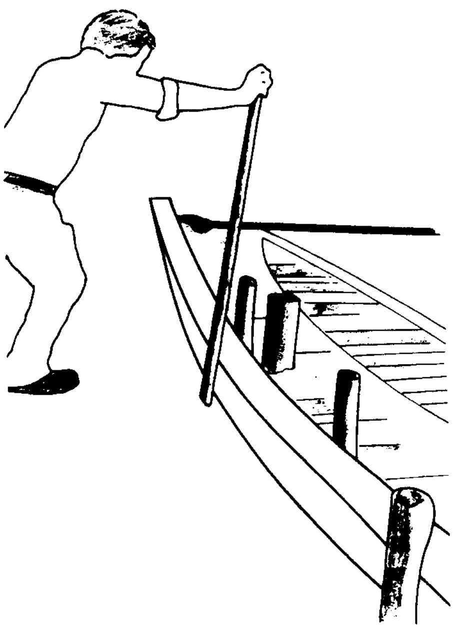


Fig. 3.35 – Hot bending with hollow section filled up with sand

3.3.3.2 "Hamburger Rohrbogen" (CHS only)

"Hamburger Rohrbogenwerk" was the inventor and holder of the patent for the classical hot forming procedure for manufacturing foldfree bends in CHS by heating it to a temperature between 850° and 1100°C and then pushing it onto an internal mandrel. An enlargement of the diameter together with bending to the required radius of curvature takes place.

3.3.3.3 Induction bending (Fig. 3.36)

This bending method can be used for both circular and rectangular hollow sections; for the latter however, further investigations are required [26]. The working principle of this machine is based on the induction heating of a short length under precise temperature control. The bending takes place only in the small heated zone. The hollow section is then pushed forward into the inductor and the following zone is heated and bent and so on.
The machine permits a large range of variables of diameter to wall thickness ratio, bending radius and angle to be accomodated. Very small radius bends, even in large diameters and wall thicknesses can be manufactured by induction bending [25].

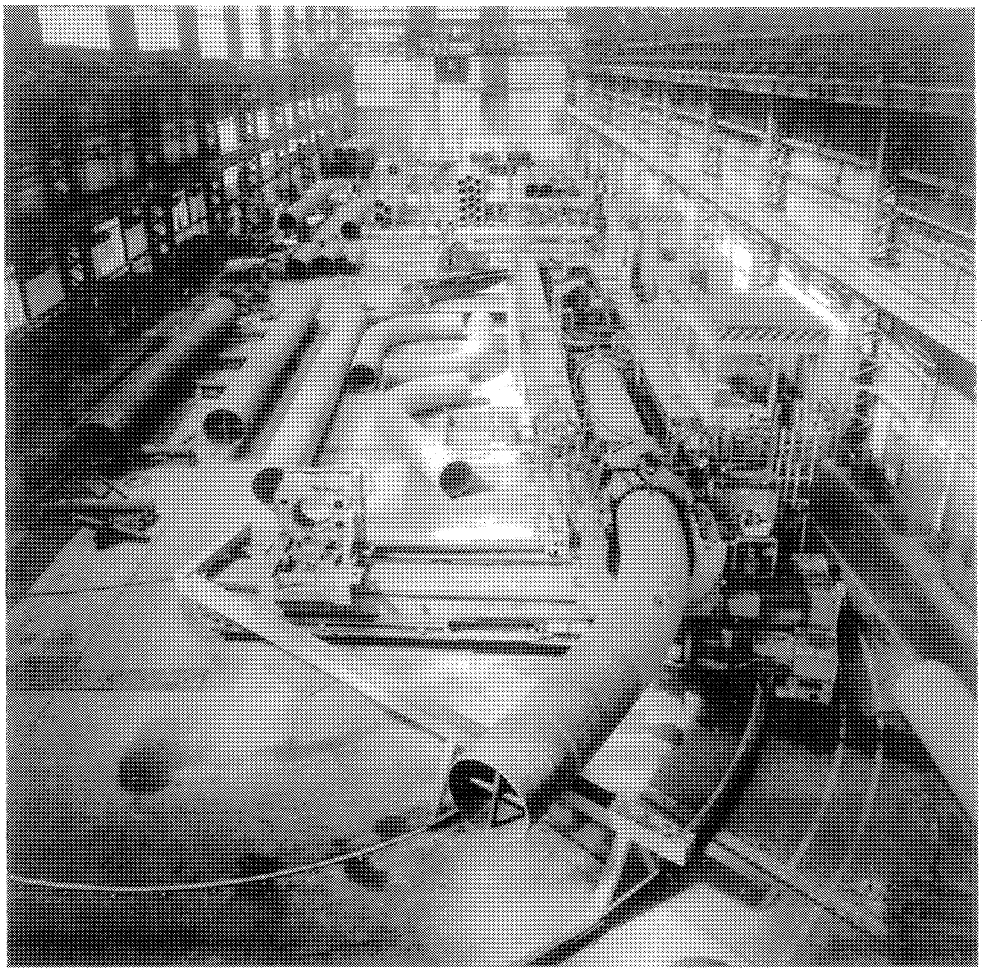


Fig. 3.36 – Induction bending

3.3.3.4 Roller bender

Both circular and rectangular hollow sections can be bent by this method.
With hot bending, the bending radius can be smaller than for cold bending (3 times the outer diameter of CHS).

3.3.3.5 Cambering

In many structures a relatively small bending curvature (very large radius) is required, for example when a girder needs to be precambered to ensure that it does not sag under load. These large radius bends can be produced cold by pressing or roller bending, as described in Section 3.3.1.1 and 3.3.1.3 respectively. However, another method, which uses no special equipment other than a heating torch, can also be used.
By heating just one side of a hollow section and allowing it to cool, it will bend, as it cools, towards the side that has been heated. With experience, this method can be quite accurate for the curvatures required, but it should be noted that the amount of curvature will vary with the heat input, the section size and thickness and the type of section (hot finished or cold formed).

3.4 Bolting

Basically, the bolting of hollow sections is not different from that used for the conventional steel construction. The calculation procedures for the application of ordinary bolts and the high tensile bolts without prestress as well as with controlled torque (prestressed) are recommended by various national and international standards [50, 54, 59, 66]. They will not be dealt with in this book, as the engineers and designers are already familiar with them. In general, the bolts and plates have to be checked for shear, bearing and failure of the net cross sectional area.
Further, the CIDECT design guides no. 1 [1] and 3 [3] have already dealt with the following bolted connections under predominantly static loading, the details of which have been discussed there:

- CHS joint flange connection [1]
- Tube – fork plate connection [1]
- Tube – plate connection [1]
- Tube – T-stub connection [1]
- Tube – gusset plate connection [1]
- RHS joint flange connection [3]
- RHS-gusset plate connection [3]

The CIDECT design guide no. 6 [6] describes also the more favourable fatigue behaviour of prestressed high strength bolt connections than those with ordinary bolts showing an example with flange joints.
The Section 4.4.1, 4.4.2 and 4.4.4 show various bolted connections of hollow sections as well as between hollow and open sections describing the background for design. As is described in Section 4.4, normal bolting for hollow section joints is only applicable to indirect connections via plates or open profiles as described above.

3.4.1 Blind bolting

Blind bolting (or single sided bolting) systems make use of either special types of bolts or inserts or special drilling systems. As the name implies, they allow bolting to take place from one side of the connection only, removing the need to get to both sides of the connection as is required for a standard bolt and nut connection. This allows for example bolted beam to structural hollow section connection details to be designed almost exactly as they would be for a beam to open section column (see Fig. 3.37).

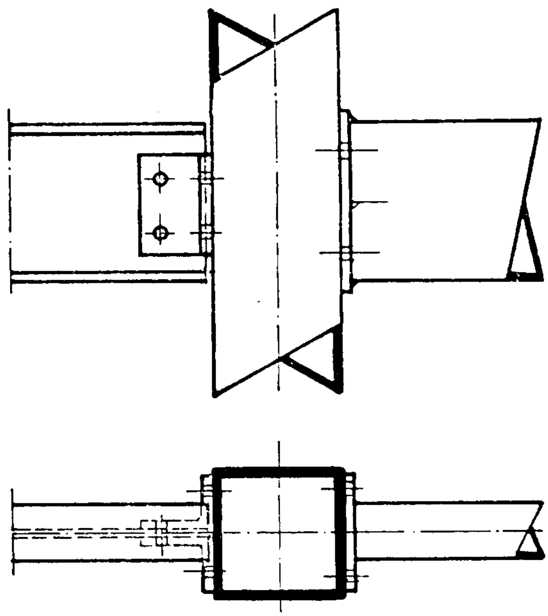


Fig. 3.37 – Blind bolted beam to column connections of hollow sections

Although a number of blind bolting systems have been in existence for a number of years, they have not been used in structural applications, mainly because their diameters were too small for structural applications. There had also, as a result, been very few investigations into their structural strength and behaviour.
In recent years, however, several blind bolting systems, for example Flowdrill, Lindapter HolloFast and HoloBolt and Huck Ultra-Twist have become available in structural bolt sizes up to M20 or even M24.
CIDECT has undertaken extensive tests to prove the load bearing capacity in the last years for the Flowdrill and Lindapter HolloFast and HolloBolt systems [20 to 23].

3.4.1.1 Flowdrill with hollow sections

The flowdrill system is a special patented method for extruded holes, which uses a four lobed tungsten-carbide friction drill to hot extrude a hole. This forms a truncated cone on the far side of the work material and a small upset on the near side. The upset can be automatically removed by a milling cutter incorporated into the drill bit. The hole is then threaded, preferably with a roll (forging) tap, rather than a cutting tap, to produce a threaded hole, which has an effective depth or effective thread length between 1.5 and 2.0 times the material thickness. Fig. 3.38 shows the Flowdrill system schematically.
The most important advantage of this system is that the equipment is fabrication shop based, in which standard bolts are used. The designer is able to use standard beam and column layouts and requires no specialised equipment on site.

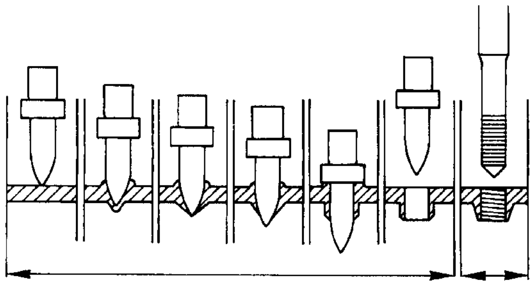


Fig. 3.38 Flowdrill system

As the test results demonstrate [20, 21], its perfomance is as follows:

- Threaded holes can be produced in both hot and cold formed RHS in thicknesses from 5 mm to 12.5 mm with M16, M20 and M24 ISO course thread profiles.
- The full tension capacity of grade 8.8 bolts can be carried by the flowdrilled holes and roll tapped ISO course threads, provided that the RHS thickness is greater than the minimum thickness shown in Table 3.3, for both hot finished and cold formed RHS in structural steel grades with nominal yield strengths of 275–355 N/mm^2 [62, 64].

Table 3.3

Bolt size	Minimum RHS thickness mm
M16 grade 8.8	6.4
M20 grade 8.8	8.0
M24 grade 8.8	9.6

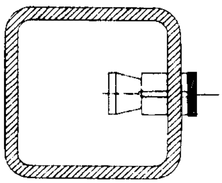
- The shear capacity of flowdrilled holes can be calculated using the normal shear design formulae for a standard bolt/nut connection.

3.4.1.2 Lindapter HolloFast and HolloBolt

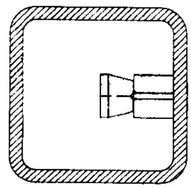
HolloFast expansion bolts are composed of a standard bolt and a special steel insert to be introduced into a hole produced by usual drilling techniques. The inserts are capable of accepting M8, M10, M12 and M16 standard grade 8.8 bolts. Fig. 3.39 shows the installation procedure for the HolloFast insert.

The working principle is that once the insert has been introduced in a hole of proper diameter using light hammer blows, the action of the tightening of the bolt in the truncated cone thread makes it separate the cone and pull it inside the HolloFast body. The consequent deformation and expansion of the cylindrical part creates four fins. These elements provide the mechanical interlock necessary to prevent the pull-out of the bolt.

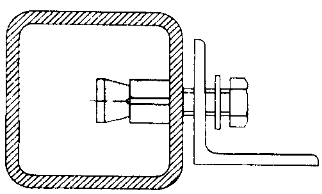
HolloBolt is a further development (Fig. 3.40), where the installation procedure is slightly different, but the basic mechanics of the fixing method are the same. HolloBolt consists of 3 pieces: a standard 8.8 bolt, a mild steel sleeve and a truncated cone. These pieces are pre-assembled by the producer. The fixing is made by the insertion of a single element through the assembly of the two pieces to be fixed together.



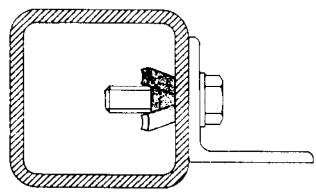
1 Drill hole as required and insert Hollo-Fast body into hole, threaded cone end first.



2 Press HolloFast into hole until the knurled face is flush with steelwork.



3 Pass bolt through hole in the fixture and screw into the threaded cone.



4 Proceed to tighten the bolt. The action of tightening the bolt separates the cone and pulls it inside the HolloFast body, which expands to form a secure threaded fixing.

Fig. 3.39 – HolloFast insert detail and installation procedure

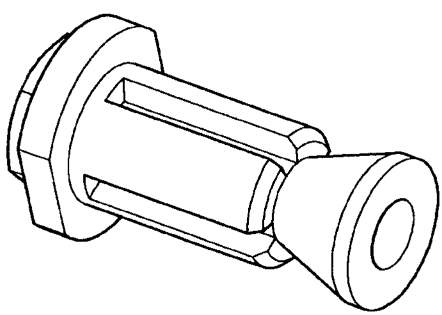


Fig. 3.40 – HolloBolt system

The results of tension and shear tests are shown in [22]. As further tests for example on moment connections are still to be performed, final conclusions cannot be made at this stage.

3.4.1.3 Huck Ultra-Twist

Huck International Inc., headquartered in Ogden, Utah, U.S.A., has developed and is now marketing blind bolts [33] that have tensile strengths and installed tensions meeting those specified for ASTM A325 bolts (equivalent to grade 8.8 bolts). Known as the Ultra-Twist fastener, they are available in sizes equivalent to 3/4 inch (19 mm), 7/8 inch (22 mm) and 1 inch (25.4 mm) diameter ASTM A 325 bolts. Fig. 3.41 shows an exploded view of an Ultra-Twist fastener and Fig. 3.42 illustrates the installation sequence.
Installation is by the use of an electric bolting wrench (as for twist-off type bolts), rather than by the use of the hydraulic wrench that was required for an earlier Huck high strength blind bolt (the HSBB). Also, the Ultra-Twist fasteners are used in holes 1/16 inch (2 mm) larger than the outer diameter of the units, which provides conventional clearances for fit-up.

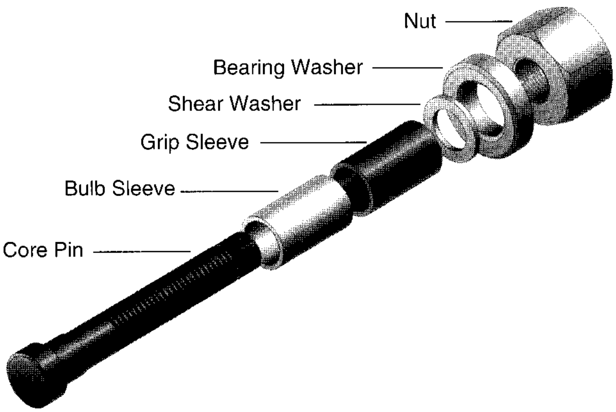
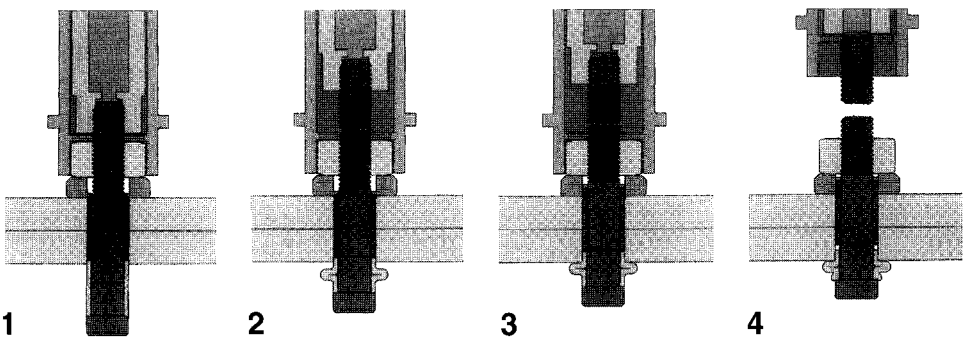


Fig. 3.41 – Exploded view of Huck Ultra-Twist fastener



1 The ULTRA-TWIST blind bolt is installed from one side of the structure by a single operator. The installation tool is the standard electric shear wrench tooling used for installation of Twist-Off Control (T-C) type fasteners. The fastener is inserted and the tool engaged.

2 The backside bulb is fully formed in the air to a uniform diameter regardless of grip.

3 As the installation load increases, a special internal washer shears allowing the backside bulb to come into contact with the work surface and for all clamp load to go into the work structure.

4 Continued torquing of the unit develops the required clamp and the torque pintail shears off, completing the installation. Using a standard S60EZ shear wrench, installation time for a 3/4″ fastener is approximately 30 seconds.

Fig. 3.42 – Installation sequence of a Huck Ultra-Twist fastener

3.4.1.3.1 Connection failure modes with Huck blind bolts

Comparative tests [34] on bolted, extended end-plate moment connections between I-shaped beams and RHS columns have been performed using both regular ASTM A325 (grade 8.8) bolts and the Huck HSBB. The connection performance in terms of stiffness, moment capacity and ductility was found to be similar for the two types of bolts and it is believed that the Huck Ultra-Twist blind bolt would produce similar results. If Huck HSBB (or presumably Huck Ultra-Twist fasteners) are used in a RHS column face and are loaded in tension, a potential failure mode is punching shear of the fastener through the column face, in which case the column thickness becomes a critical parameter [34]. To avoid this failure mode, the limit states resistance of the blind bolt in tension should be less than the limit states resistance of the column face in punching shear. It can be shown [9] that this will be achieved if:

$t > (0.8\ T_r)/(d_f'\ f_u)$

where T_r = tension resistance of the fastener
d_f' = diameter of HSBB primary sleeve after installation, or diameter of the Ultra-Twist fastener + 6 mm (estimated effective bulb diameter)
t = wall thickness of the hollow section
f_u = specified minimum tensile strength of the RHS material

Another critical failure mode for an unstiffened RHS column face loaded by point tension loads at the fastener positions is yielding of the RHS connecting face. This failure mechanism can occur due to the flexibility of the column face at medium to large wall slenderness ratios. The limit states resistance for this failure mode can be calculated by assuming that the column wall is loaded like a 90° T-joint with the branch member in tension. (See "Chord face yielding" in Table 2 of CIDECT Design Guide no. 3 [3], which is based on a yield line mechanism.) In this case, the "branch member" can be assumed, for two bolts in tension, to be of width ($w + d_f'$) and depth d_f', where w = distance across the RHS column face between the bolt hole centres.

If either of these failure modes (punching shear and column face yielding) produces an inadequate resistance for the RHS column face in the connection tension region, the column face will need to be reinforced. This is best achieved by welding on a doubler plate to the column face. Unfortunately, column reinforcement will nearly always be necessary for practical moment connections, but may not be needed for simple shear connections.

The problem of requiring a very thick column wall to achieve an unreinforced moment connection is recognized in Japan where recent research has focused on developing a method for increasing the column thickness just in the connection region, by using an induction heating device and jack [35].

3.4.2 Stud welding

Studs can be welded onto the face of a hollow section after cleaning the surface of the material carefully. As Fig. 3.43 shows, some methods of stud welding leave a collar at the root (where the stud meets the section). Where this is the case, the bolt holes in the connecting flange are to be recessed to clear the collar (Fig. 3.43a) or clearance washers are to be fitted (Fig. 3.43b).

For light weight fixings such as claddings fixed directly to hollow section purlins, stud and self-tapping screws are employed.

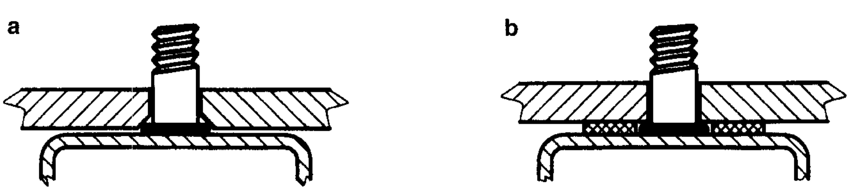


Fig. 3.43 – Stud welding with
a) collar clearence by flange recess b) collar clearance by fitted washer

3.5 Nailing

As an alternative to bolting, circular hollow sections can be nailed together to form a spice connection by inserting one CHS inside another, for which the inside diameter of the larger equals the outside diameter of the smaller. The nails are arranged symmetrically through two wall thicknesses (Fig. 3.44). Another alternative is to join two CHS of the same outside diameter by means of a tubular collar over both CHS ends.

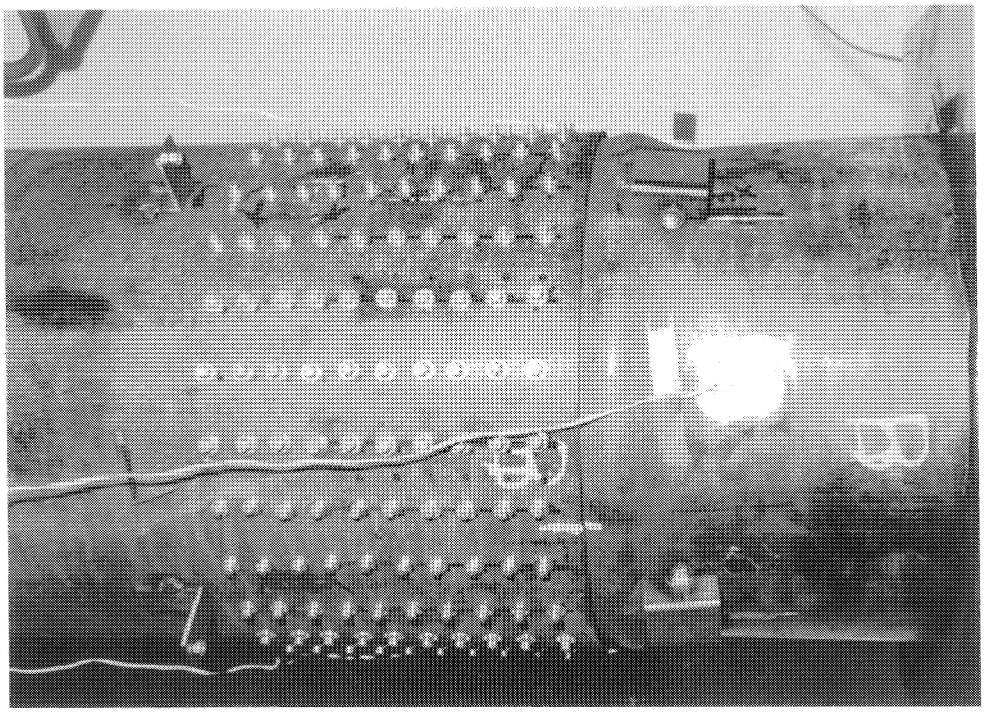


Fig. 3.44 – Nailed CHS connection after testing

Nailing involves an independent powder-actuated tool (or gun) which drives a high strength ballistic point pin (or nail) into the steel through two wall thicknesses (Fig. 3.45).
The closed nature of this connection does not allow one to confirm adequate penetration through both CHS walls directly. The "stand-off" height of the nail head from the outer CHS wall and the measurement of sufficient insertion of one CHS into the other can however demonstrate the connectivity. Failure modes observed [30] are shear failure of the nails and bearing/shear-out failure of CHS wall. The formulae to determine the ultimate connection strength are given in [6].
One other application of the nail as a structural connector is as a mechanical shear connector in concrete filled hollow sections [31], wherein the nail is driven through the steel wall and into the concrete.

Fig. 3.45 – Fabrication of a nailed CHS connection

3.6 Welding methods

As has been previously described in Chapter 3, welding represents the major method, by which hollow sections are joined. Subchapter 2.1.1 explains the impact of welding operations on the steel grades used for manufacturing hollow sections. Applications of welding as well as a combination of welding and bolting to hollow section structures and subassemblies are illustrated in Chapter 4.
This chapter is dedicated to the methods of welding of hollow section connections and the related solutions to the problems of post treatment of welded hollow section structures.

3.6.1 Methods for welding hollow section joints

Welding of hollow section joints belongs mainly to the group of fusion welding methods [69], although, if the number of units are large, friction welding belonging to the group of pressure welding methods also comes applicable. Among the five subgroups of fusion welding, namely autogenous gas, metallic arc, electric resistance, electro beam and plasma, which produce the fusion heat in their own specific manners, metal arc welding [51, 58, 70] is predominantly used in the following three versions for jointing hollow sections:

1. Shielded metal arc welding (SMAW)
2. Flux cored arc welding (FCAW)
3. Gas metal arc welding (GMAW)

However, for special applications i.e. offshore constructions, submerged arc welding (SAW) is also used.
As regards welding equipment and machines, three methods, i.e. manual, semi-automatic and machine or fully automatic, are to be differentiated. The first two are normally used in the case of hollow sections. Fully automatic welding is not usual, but can be applied when practicable.

Manual shielded metal arc welding with stick electrodes coated with a layer of shielding flux chemicals is used in workshop as well as for site welding. In particular, they can be applied when disadvanteous welding positions including overhead exist and/or restricted welding access prevails.

The coating of the electrode has the function of shielding glowing liquid steel at the weld from the detrimental effects of air by means of slag or gas, it is important to make an appropriate selection of electrodes. This has to be decided from case to case; all decisive points, such as joint types, welding positions and weld investigation methods, determine the type of electrodes used. In general, the mechanical properties of the weld material are superior to those of the basic materials.

For the welding of hollow section structures, the electrodes of the rutile acid and basic types are applied.

Depending on the steel grades, wall thickness and weld forms, the following electrode types are recommended:

S235 and S275 [62, 64]:

For wall thickness	$\leq$ 16 mm (butt weld) $\leq$ 30 mm (fillet weld)	Rutile or basic, hydrogen controlled electrodes
	> 16 mm (butt weld)	Basic, hydrogen controlled electrodes

S 355 [62, 64]:

For all wall thicknesses	Basic, hydrogen controlled electrodes

In a workshop, where different steel grades are fabricated, it is recommended to use hydrogen controlled electrodes only.

In general, the recommendations of the electrode manufacturers should be followed to protect and store the electrodes.

They must be kept dry and undamaged. Drying ovens should be used for the hydrogen controlled basic electrodes. An alternative is to use low hydrogen vacuum packed electrodes.

Manual welding demands adequate skill and experience from the welder, although the welder has a higher degree of freedom.

Flux cored arc welding is a semi-automatic process using electrodes as continuous hollow wire fed in from a spool on the welding machine. The wire contains flux chemicals, which provide protection of the arc and the molten metal from the harmful effects of oxygen and nitrogen. Over and above, shielding gases are also delivered to the operator's "gun". Mainly applied in the workshop, this system requires expensive equipment, which necessitates large investment. But this is compensated by a higher working rate i.e. deposition two or three times faster than SMAW and saving of time, because the welder does not require to move much.

Gas metal arc welding is also a semi-automatic process similar to the previous one. However, in this case, the continuous wire to be fed is a solid wire and the weld is shielded by an inert gas Argon or Helium (Metal inert gas MIG) or by a less expensive gas CO_2 or gas mixture (80% Ar + 15% CO_2 + 5% O_2) (Metal active gas MAG).

The advantages of gas metal arc welding consist of :

1. Fast welding process, can reduce fabrication costs
2. Narrow heat affected zone of the weld
3. Absence of slag, which can prevent welding in difficult positions. As the slag does not require to be removed from the subsequent welds, the welding time is shorter and the fabrication cost is lower.
4. Main fields of applications are:
    1. Non-alloy and low alloy steels (MAG)
    2. High alloy steel (MIG)

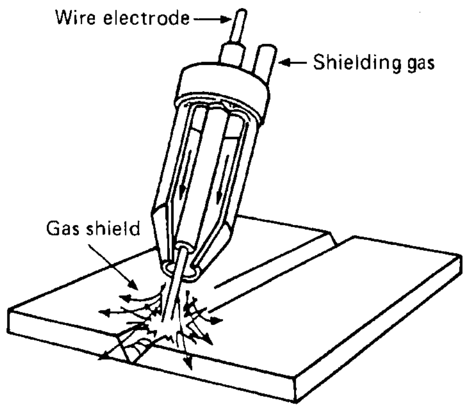


Fig. 3.46 – Semi-automatic (MIG) welding method

The disadvantages are:
1. Not suitable for site welding due to large amount of equipment, such as electric control, spooled solid wire electrode, wire feeder, shielding gas supplier
2. Welding access substantially restricted by the space needed for the gas shield nozzle, even though various shapes of nozzles are available.

3.6.2 Welding positions and sequences

Three principal points should be mentioned at the beginning:
1. Welds must not start or stop on a corner of RHS.
2. For the smaller thicknesses, multipass welds should be avoided, wherever possible.
3. Appropriate welding sequences have to be followed, because they affect the shrinkage, residual stress and deformation of a welded structure significantly.

Depending on the position and movability of the structural elements, the following four welding positions are shown for structural hollow section joints together with the welding sequences.

1) 360° rolling weld (Fig. 3.47)
   A downhead (flat) weld is made, while the section is rotated through 360°.

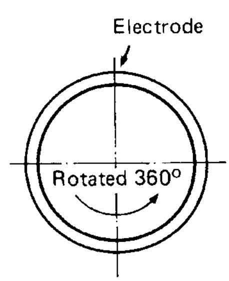


Fig. 3.47 – 360° rolling weld

2) 180° vertical-up weld (Fig. 3.48)
   For a lattice girder construction, all the welds are typically made on the top side and the panel then turned over (through 180°) to complete the operation.

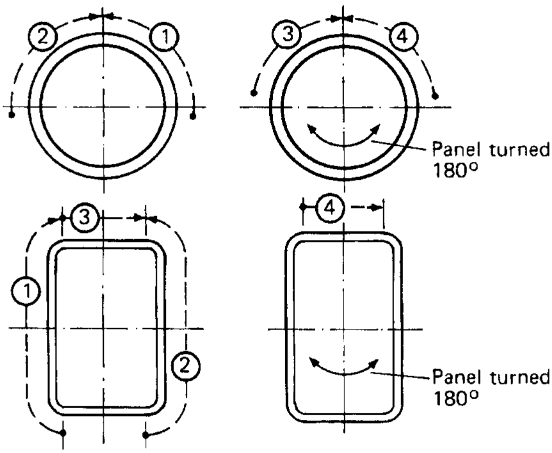


Fig. 3.48 – 180° vertical-up weld

3) Vertical-up weld (Fig. 3.49)
This position, although rare, is only used when the hollow sections cannot be moved.

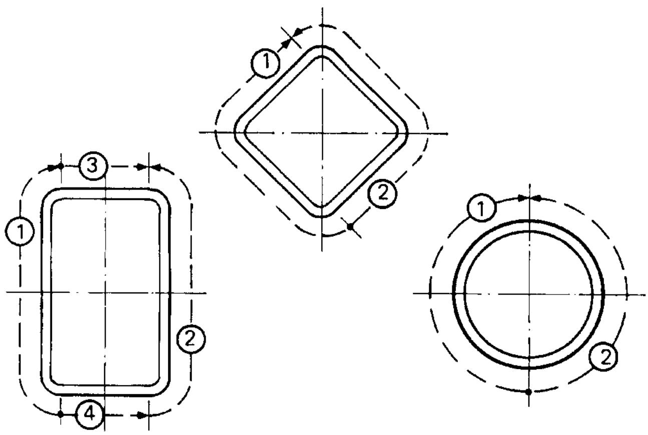


Fig. 3.49 – Vertical-up weld

4) Horizontal weld (Fig. 3.50)
This position is necessary when the members are in an upright position and cannot be moved.
If the members are in a horizontal position, the welds are made in a vertical position.

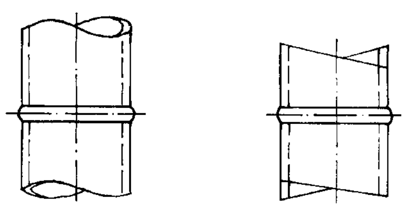


Fig. 3.50 – Horizontal weld

3.6.3 Tack welding

Tack weld is a short weld done for the preliminary joining of hollow section members in a structure for temporary fixing prior to the final welding of the assembly. The throat thickness of tack welds has to be in accordance with the root position. This should guarantee a clean connection at the weld root. The ends of tack welds should be properly done to permit good fusion into the root run.
Tack welds have to be located in positions suitable for the stop/start positions as shown in Fig. 3.47 through 3.50. As tack welds become part of the final weld itself, they have to be carried out with great care. That is why the welders need special qualification for tack welding to do this job [58].
CHS tacking is done by circumference welding, when the CHS diameter is small. This can however cause problems for large tack welding, which can be easier avoided by reducing the tacking length to a minimum of 1/10 of the circumference. Nevertheless, the tack weld must be free from defects and any repair must be performed by a qualified welder.
With regard to tack welding of a CHS joint as shown in Fig. 3.51, welding has to be avoided at the symmetrical position A of the branch tubes due to the local stress concentration there.
In general, minimum tacking length of a branch tube can be decreased to 1/10 of the tube outer diameter.

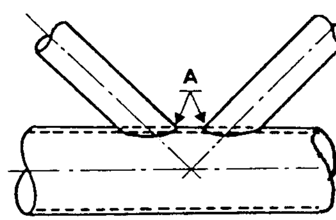


Fig. 3.51 – Tack weld has to be avoided at A

For RHS, the tack weld should be carried out in the straight line part (not at the corners).

3.6.4 Pre- and post-heat treatment of welded constructions of hollow sections

[67] recommends pre-heating a workpiece before welding, if the wall thickness of the structural elements differ by 10 mm from one another. Further, pre-heating may also be necessary if a hollow and a solid profile have to be welded together.
In general, a low temperature environment and also humid atmosphere as well as thick walled hollow sections may lead to a critical condition and further to cold cracks. This can be effectively prevented by pre-heating the workpiece in the range between 50°C and 200°C depending on the steel grade. This expensive operation may however be avoided by taking care that parts to be welded are free of condensation and also using hydrogen controlled electrodes.
The pre-heating temperature is determined based on the carbon content and carbon equivalence CEV (see Chapter 2.1.1), welding current, voltage and speed, thickness of workpiece, weld type and hydrogen content of the electrodes.
For the non-alloy structural steels S235, S275 and S355 [62, 64], pre-heating is not required in general. However it is recommended for an ambient temperature lower than + 5°C and for wall thickness ≥ 30 mm.
Specifically for S355, it is not necessary to make any pre-heating treatment up to 13 mm wall thickness for fillet welds and up to 20 mm wall thickness for butt welds.
For larger wall thicknesses, a minimum of 125°C pre-heating temperature is recommended.

Pre-heating for S460 is to be provided for wall thickness > 8 mm for fillet weld and > 12 mm for a butt weld. A minimum of 175°C pre-heating temperature is recommended in this case.
For flame cutting pre-heating of the parts intended to be cut is not usually necessary. However, pre-heating improves the cutting edge. Pre-heating to a temperature of 120°C is recommended for this purpose.
Stress relief heat treatment is performed, after welding is carried out, only when the welding stresses have to be reduced. Normally, the stress relief temperature lies between 530° and 580°C. For high strength steels e.g. S460, this is about 30° to 50°C lower then the anealing temperature of the material.

3.6.5 Residual stress and deformation due to welding

A workpiece is heated locally by welding. Thermal elongation of the material is not uniform, as this is obstructed by the surrounding cold material.
Shrinkage stress occurs due to the contraction of the workpiece as it cools down. Either this is transformed into deformation or twisting or remains in the welded part as residual stress.
The deformation and residual stress in a welded hollow section structure depend on the following parameters:

- Thickness of weld
- Number of weld passes
- Distance of the weld to the neutral axis of the structural element
- Restraint of a welded structural element by the connection members
- Rigidity of the structural member in a welded structure
- Angle of inclination of the member axes to one another
- Welding sequence
- Welding method

The deformation due to shrinkage is strongly prevented in a rigid construction during welding. On the other hand, the welding residual stresses increase significantly in this process. The possibility of an engineer to design a structure is restricted to reduce the deformation leading to higher residual stress or reduce the residual stress and increasing the shrinkage. The decision has to be made considering both effects.
In order to reduce the straightening and aligning work after welding, the distortions can be compensated by corresponding pre-deformations. The procedure is shown by an example with a lattice girder in Fig. 3.52.

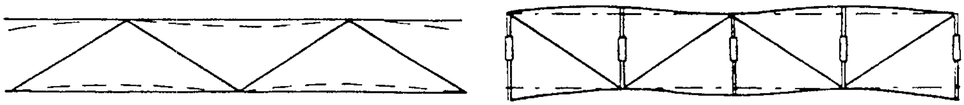


Fig. 3.52 – Pre-deformation of a welded lattice girder
a) Estimation of deformation b) Presetting the chords with jacks

After estimating the deformations of a lattice girder due to welding, the chord members can be preset accordingly by means of jacks.
Residual stress and shrinkage are determined by the weld arrangements and sequences proportionately. Tack welds before the final welding must be sufficiently numerous and strong enough to absorb the transverse shrinkage stresses, when welding takes place.
Fig. 4.75, which shows the recommended welding sequence for joints in a hollow section lattice girder, demonstrates that the welding proceeds always from the inside to the outside direction. This results in the free movement of parts towards one another due to shrinkage and consequently in low deformation and small residual stress.
Further measures to lessen deformations and/or residual stresses are local heating in appropriate places (Fig. 3.53), hammering of the welds (seldom used) etc.

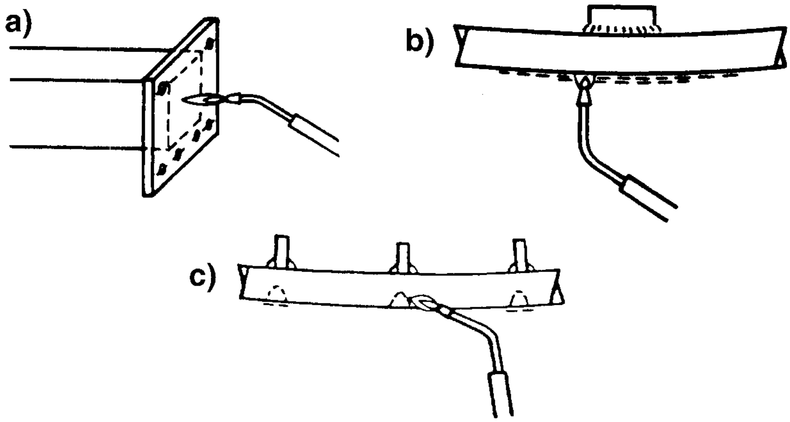


Fig. 3.53 – Reduction of welding deformation by local heating
a) Circular heat application
b) Linear heating to avoid shrinkage on connections of long length
c) Heating in a triangular mode to suppress angular distortion caused by fillet weld

3.6.6 Weld defects and repair

Fig. 3.54 gives a survey of possible weld defects in fillet and butt welds.

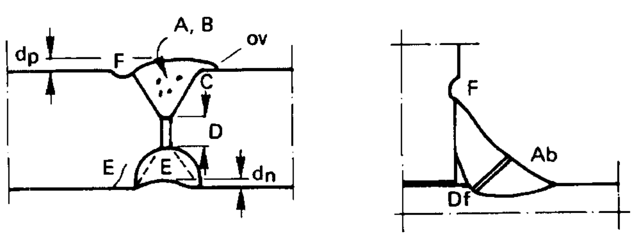


Fig. 3.54 – Types of weld defects

E	= Crack	Df	= Incomplete root penetration in fillet weld
C	= Lack of fusion	D	= Incomplete root penetration in groove weld
A	= Gas cavities	F	= Undercut
Ab	= Worm holes	d_n	= Insufficient throat
B	= Slag inclusions	ov	= Overlap

The acceptable and unacceptable weld profiles are illustrated in Fig. 3.55 [58].
The faces of fillet welds may be slightly convex, flat or slightly concave. The maximum convexity is, however, restricted depending on the leg size or width of the individual surface bead L (see Fig. 3.55 (1)), as recommended by [58].
Butt welds shall preferably be made with slight face reinforcement. Face reinforcement shall have a gradual transition to the plane of the base metal surface. They shall be free from discontinuities, excessive convexity, insufficient throat, excessive undercut and overlap.
Weld defects can be repaired by removing weld metal or portions of the base metal by machining, grinding, chipping or gouging.
Weld defects such as overlap, excessive convexity or reinforcement have to be removed without substantial removal of the base metal. Thorough cleaning of the surface is compulsory before welding.

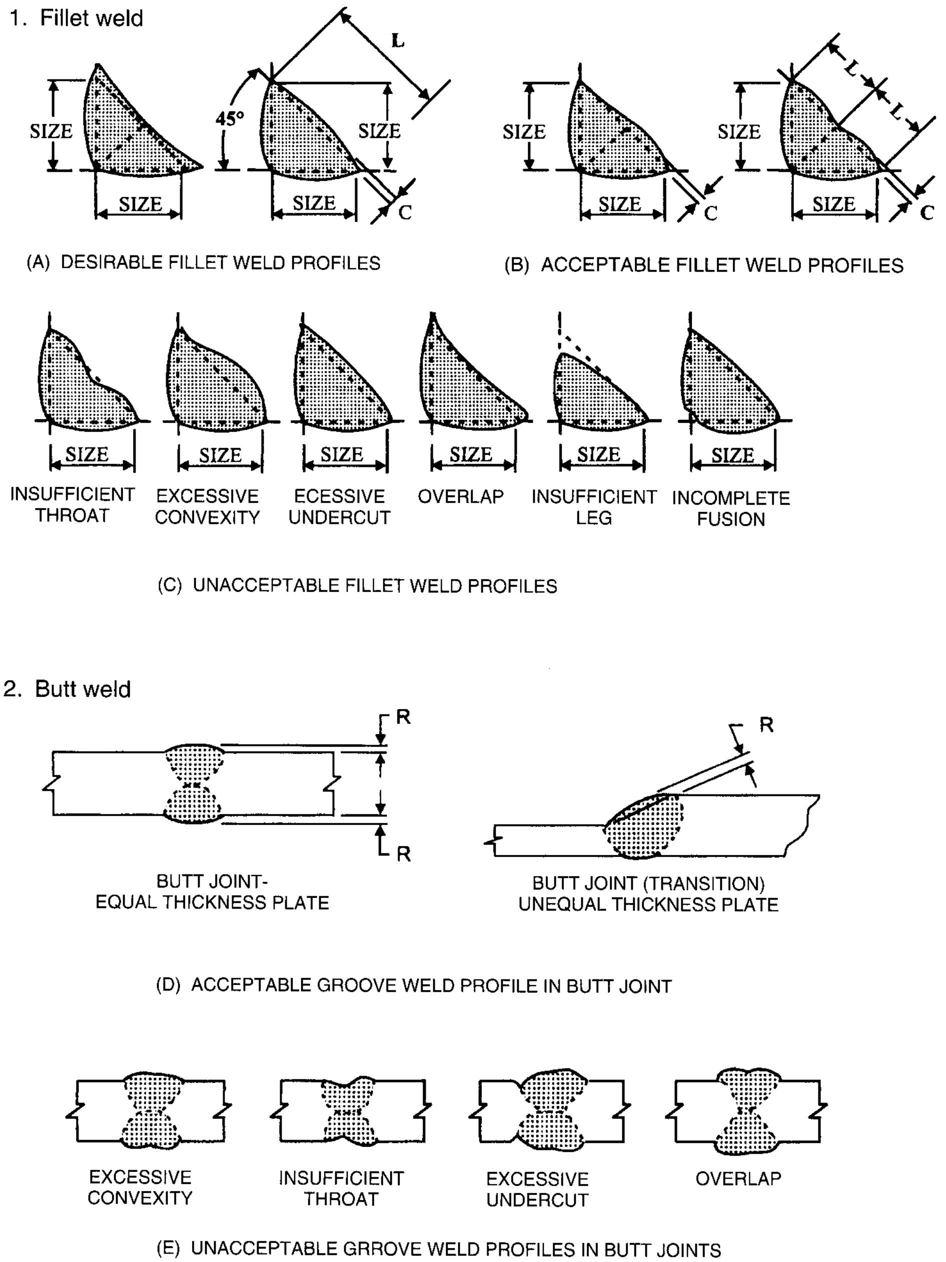


Fig. 3.55 – Acceptable and unacceptable fillet and butt weld profiles

Any deficiency in weld size by excessive concavity, undersize weld and undercut has to be compensated by depositing weld metal. Incomplete fusion, excessive weld porosity or slag inclusions are to be removed and rewelded. Cracks in the weld or base metal are repaired by removing the cracks and sound metal (depending on thickness) beyond each end of the crack and rewelding.

3.6.7 Inspection of welds

Welds in steel structures can be checked either by destructive (in laboratory only) or non-destructive tests. A number of test methods belongs to each of the two groups. They have their advantages and disadvantages and accordingly their specific fields of application. From an economical point of view the extent of inspections should be kept to a minimum. Fillet welds are preferred to butt welds.

Destructive tests consisting of tensile, folding, impact resistance, hardness and fatigue tests are usually made before the final welding of a structure starts. They are also to investigate new materials, construction types and welding methods indicating the welding parameters. These tests are also carried out to check the professional ability of welders.

Contrary to the destructive tests, the following five non-destructive test methods can give conclusive information as to the effective quality of a weld:

- Visual inspection
- Magnetic particle test
- Dye penetration test
- Ultrasonic inspection
- Radiographic inspection by X- or γ (gamma)-rays

A very close visual inspection of the weld seam as well as the weld vicinity, both prior to and after welding, is of vital importance.

It is therefore recommended to check the root gap between the parts to be welded, angle of inclination between the structural members, uniformity of weld edge preparation, angle of bevel, width of face alignment and the complete removal of oil, grease etc. from the weld locality before welding. It is also necessary to engage skilled and qualified welders for the particular type of weld in question.

After welding, surface defects such as undercut, overlap, and cracks as well as weld appearance (roughness of bead surface, bead width etc.) are to be scrutinized visually. The measurement of weld throat thickness and the transition of weld seam to base metal (this is of specific importance for fatigue loaded construction) is to follow with gauges developed for this purpose.

The magnetic particle test is a quick and convenient method to discover surface defects like fine cracks, which are not obviously visible. This method is mainly applied to find weld defects in nodal joints, which are very difficult to determine by using other methods e.g. ultrasonic or radiographic inspections. Fine magnetic particles are sprayed on to the surface to be checked and a magnetic flux field is produced there by means of a magnetic coil or yoke. When a crack distorts or discontinues the magnetic field, the magnetic particles line up along the cracks indicating even the finest ones (up to 1/10000 mm) distinctly. The measurement record is made by photographs.

The dye penetration test explores the weld defects rising to the surface of weldments. The procedure consists of cleaning the surface to be checked thoroughly first and then applying a penetrating red dye solution to it by means of a brush or a spray. The solution is allowed to act for about 5 to 10 minutes, during which the dye is drawn into even very minute cracks. In order to attain this condition, the dye solution must have low surface stress and high capillarity. The excess dye is then wiped off by cloth followed by cleaning the surface with water or a solvent specially developed for this purpose. When the surface is dry, either a white powder is applied thinly to it or a quick drying white developer solution is sprayed on it. This sucks the dye from any defect into which it has been drawn, thus marking, red on white, a clear outline of the fault. The results can be documented by photographs (Fig. 3.56).

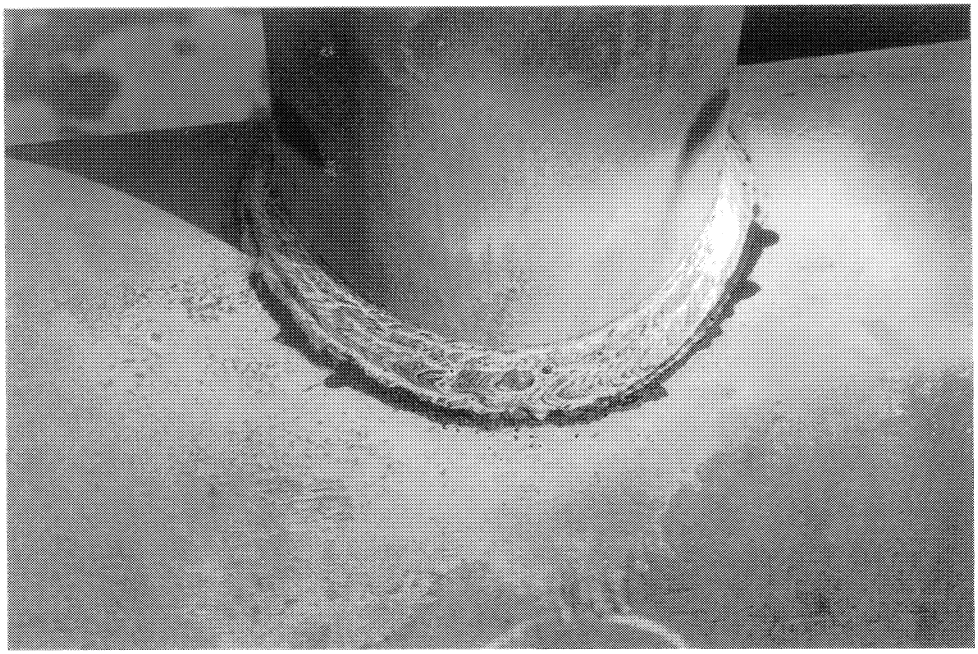


Fig. 3.56 – Cracks determined by dye penetration test on a CHS X type joint

Ultrasonic inspection is a very quick operating method, which however requires qualified and experienced examiners to carry it out. High frequency sound waves are sent into a weldment by a sender, which are then reflected from the locality of a defect. The echo is displayed electronically on an oscilloscope screen, which acts as a receiver. The exact location and the approximate size of the defect are determined by measuring the time for the wave to cover the distance. It is however very difficult to determine the type of the defect exactly, and demands adequate judgement and experience of the operator. It is to be mentioned that this procedure can only find out defects perpendicular to the direction of the sound wave. The lack of a permanent record of the findings has been considered a disadvantage of the process in some situations, but electronic recording equipment is now available (Fig. 3.57).

Fillet welds and partial joint penetration groove welds cannot be checked reliably by the ultrasonic method, as the signals become difficult to interpret.

Radiographic inspection consists of directing either X-rays or γ (gamma)-rays from Cobalt or Iridium through a weldment and producing a photographic film (Fig. 3.58). This method is specially capable to find out incomplete fusion, porosity and slag inclusions. Irregular shapes such as those in joints and variations of thicknesses are not suitable for radiography.

As longtime exposure to X- or γ-rays is detrimental to health, the examination takes place in an enclosed space.

Radiographic tests offer reliable values for about 16 to 20 mm wall thickness. Ultrasonic tests gain in importance beyond that. It is difficult to inspect the corner region of a rectangular hollow section. For fillet welds, it is not possible to use radiographic or ultrasonic tests, as they do not give reliable results. Surface defects on fillet welds can only be determined by dye penetration or magnetic particle tests.

In practice, the weld examination is generally restricted to visual inspection, which demands an experienced inspector to judge the weld quality.

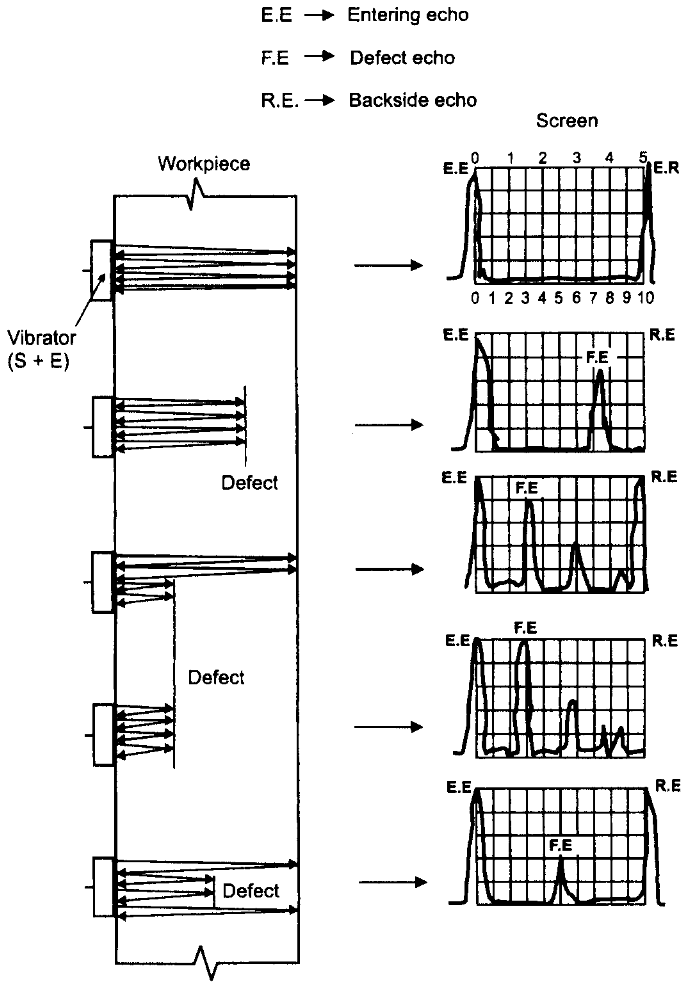


Fig. 3.57 – Indication of weld defects on a screen

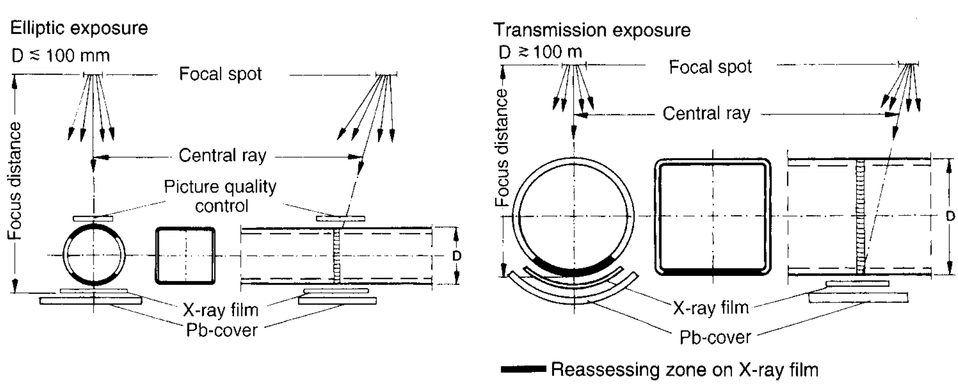


Fig. 3.58 – Production of X-ray films

3.6.8 Welder qualification

Welders, welding operators and tack welders must be adequately qualified to perform welding, which can meet the required load bearing capacity of a structure. In order to prevent non-qualified workers carrying out welding, examinations to qualify as a welder have been prescribed in various codes and standards [52, 58, 68, 74].
The type and extent of the welder's qualifications are determined by the welding jobs done in a workshop. The standards contain the following different criteria, which have to be accounted for:
- Welding method
- Form of structural element
- Weld type
- Steel grade
- Electrode
- Thickness of specimen (plate and tube)
- Diameter of tube specimen
- Welding position
- Butt weld detail
- Weld root

Approved welding research centres and technical supervision authorities are responsible for these examinations [52]. It is important to note that the welders, who weld hollow section lattice structures, shall be approved by means of an appropriate weld test. The test should be configured to include the positional geometry and single-sided combined fillet-butt welds common to welded hollow section joints [67].

3.6.9 Welding of metal coated or galvanised hollow section structures

As adequately large hot dip galvanising vats are not often available for dipping complete structures, the fabricators are sometimes forced to connect single galvanised structural members by welding (if bolted construction is not preferred!). The question of welding the galvanised elements arises in this case.
In general, the zinc oxide layer is burnt off in the welded zone (about 50 mm from the weld seam) and then removed by blasting or grinding, so that the coating material does not affect welding. After welding, the area undergoes protecting treatment by painting and spraying reconditioning material e.g. zinc rich paint, metallised zinc. Due to the ease of application and low expense as well as good adhesion and wear resistance, zinc rich paint is commonly used.
Poisonous fumes are emitted during burning-off the zinc oxide layer. It is therefore necessary to take proper measures for their extraction while welding indoors.
Arc welding under controlled atmosphere as performed by the MAG method, does not need any step for burning the coated zinc layer off. This is done during welding without any modification of the working method and without any loss in the mechanical properties of the weld. Reconditioning of the weld region as described above, is made after welding.

3.6.10 General recommendations regarding welding operation

1. It is particularly important that the accessibility for welding is assured. Welding torch, gas shielding nozzle and electrode clamp must have enough space for performing welds conveniently.

2. Often the fabricators are inclined to specify and carry out larger welds with weld throat thicknesses larger than technically required. This is not only more expensive but also harmful due to the danger of excessive shrinkage and distortion as well as change of the microstructure of the base metal in the heat affected zone (HAZ) by additional heat input.
3. Fillet welds are preferred to butt welds. They are to applied as long as their sizes do not become excessive.

   If a fillet weld is not feasible, partial penetration butt welds can be applied, as they are less expensive than complete penetration butt welds without backing bars. The latter, however, can be practicable with backing bars.
4. The crowding of a joint by welding too many plates or sections is not only detrimental to the welding accessibility, but can also increase external corrosion by producing water or snow traps.
5. As visual inspection is the cheapest, most convenient and most applied weld inspection method, it is necessary that the welding engineer should possess the required qualification and also experience to carry out this job. Other methods are applied to critical joints, only when they are feasible.

3.7 Holing

Holes are normally made in structural hollow sections only by drilling. It is not possible to do this by punching due to its hollow shape unless an internal support is used.

3.8 Application of robots

Robotics have the potential to improve speed, productivity, quality and working conditions as well as to decrease the time and cost for the fabrication significantly. Meanwhile the use of robots has been wide spread to welding, transfer and handling, assembly, painting and inspection. Especially in the field of welding, arc welding robots have acquired a dominant position in most industrialized countries.

However robotics technology integrated with a material handling system and a design process (i.e. CAD/CAM) raises crucial process production and cost issues, which require careful consideration before it can be usefully applied. The issues include (in no particular order):

- The degree of fabrication
- The level of batch production attainable
- The interface of planning, design, manufacture and production
- The level of design standardisation
- Material handling
- Production engineering
- Logistical, operational and organisational management
- Financial effects and consequences

As an example of the application of robots in a hollow section structure, Fig. 3.59 shows a robotized system for finish-welding a complete truss, which is a part of a grillage (Delta system [32]) consisting of square hollow sections. In this unit, two chords, verticals and diagonals can be accomodated in a rotary frame with chucks, which provide free access for the robot to all welding points. Figs. 3.60 and 3.61 contain the exploded view of the "Delta" joint and the structure respectively. In this case, there is a further unit with two tables and a robot travelling on a linear path, where the joint quarter parts are welded to two faces of the chords.

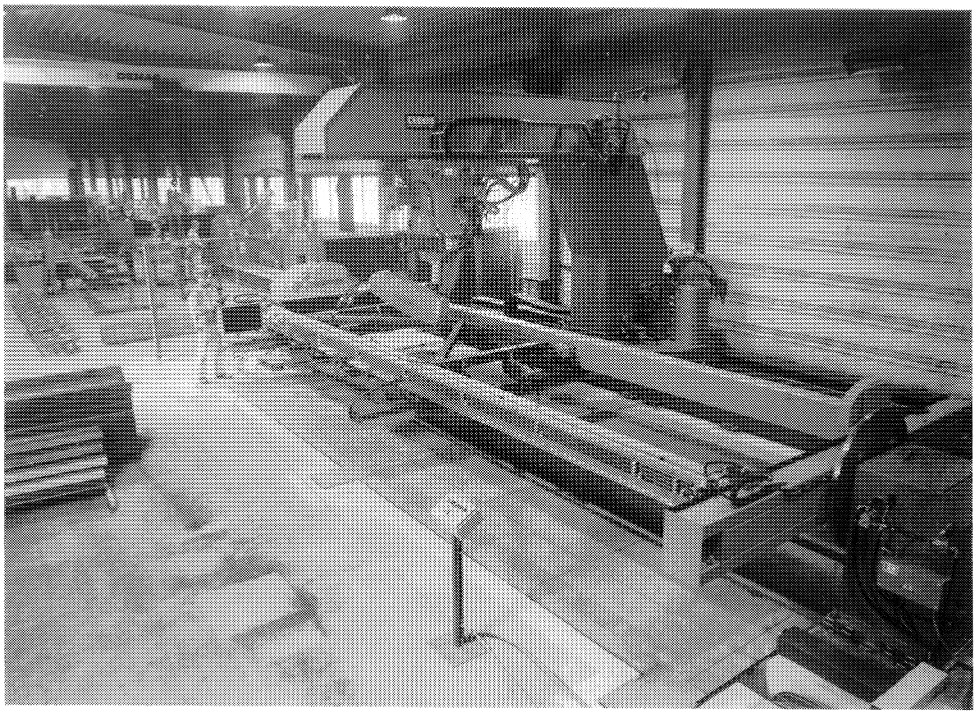


Fig. 3.59 – Robotized welding system for lattice girder in "Delta" system [32]

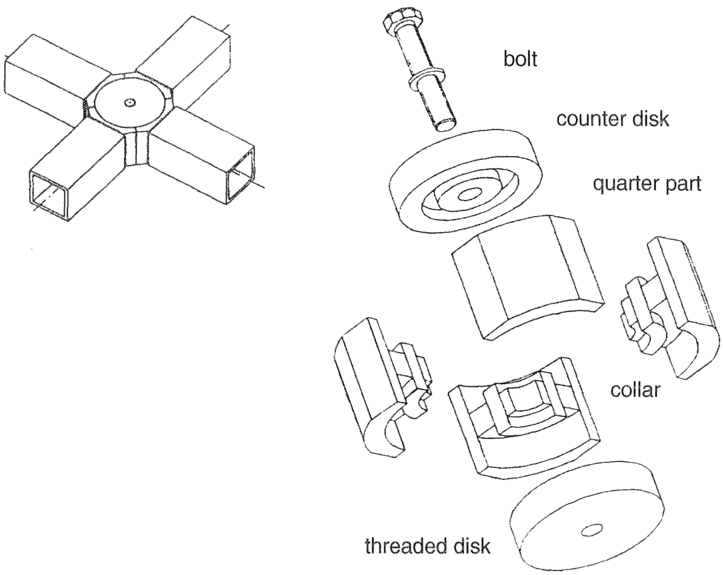


Fig. 3.60 – Exploded view of "Delta" joint

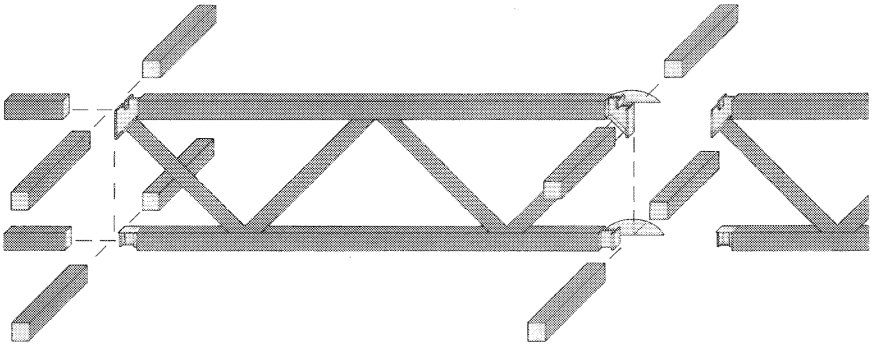


Fig. 3.61 – Parts of a "Delta" strucure

According to [32], automation achieved for "Delta" structures by robots has reduced the production time by a factor of 4 as against manual welding. Besides flexibility attained by the application of robots, constant quality standards and very high precision are the other attributes.

Fig. 3.62 – Tree-shaped columns in air terminal Stuttgart

3.9 Castings for structural hollow section connections

When a connection is very difficult to fabricate by welding or when many identical and fairly complex connections are required, it can often be economic to use castings instead of the normal welded fabrication methods. The decision to use castings will depend on many items such as the complexity of the connection, the number of similar connection, the size of the connections and relative costs as well as the feasibility, which depends on the mechanical properties including fracture toughness and fatigue and weldability. The use of castings for tubular joints offers obvious advantages. It offers a homogeneous integral component with low residual stresses and low geometrical stress concentrations compared to welded joints. Further, the thickness of the casting can be varied to accomodate any areas of high stress. Specially for tubular joints under fatigue load, cast joints are significantly stronger than welded joints.

Fig. 3.62 demonstrates the application of cast steel joints for the tree-shaped columns in the air terminal at Stuttgart, Germany. Fig. 3.63 illustrates the continuous smooth transition in the joint avoiding any abrupt change of cross section and inclination. The complexity of joints, which can be made using castings, is illustrated in Figs. 3.64 a and b.

Fig. 3.63 – Cast steel joints for tree-shaped columns

Castings can be made from a variety of materials, however, for applications to structures they will generally be a) cast iron b) cast carbon steel or c) cast stainless steel.

The type of cast iron used in structural applications is generally spheroidal graphite iron (nodular cast iron, ductile iron), because of it's improved ductility compared to other cast irons. It is, however, not easily weldable and is, therefore, much more suitable for bolted connections than welded ones. These irons, depending on their grade, can have 0.2% proof strengths between 200 and 700 N/mm^2 in tension.

Cast steels, both carbon steel and stainless steel, can be produced with very similar mechanical properties to those of structural hollow sections. They also have similar welding characteristics to structural hollow sections with the same carbon equivalent value CEV. Suitable welding processes for cast steels are manual metal arc (MMA), metal inert gas (MIG) and tungsten inert gas (TIG).

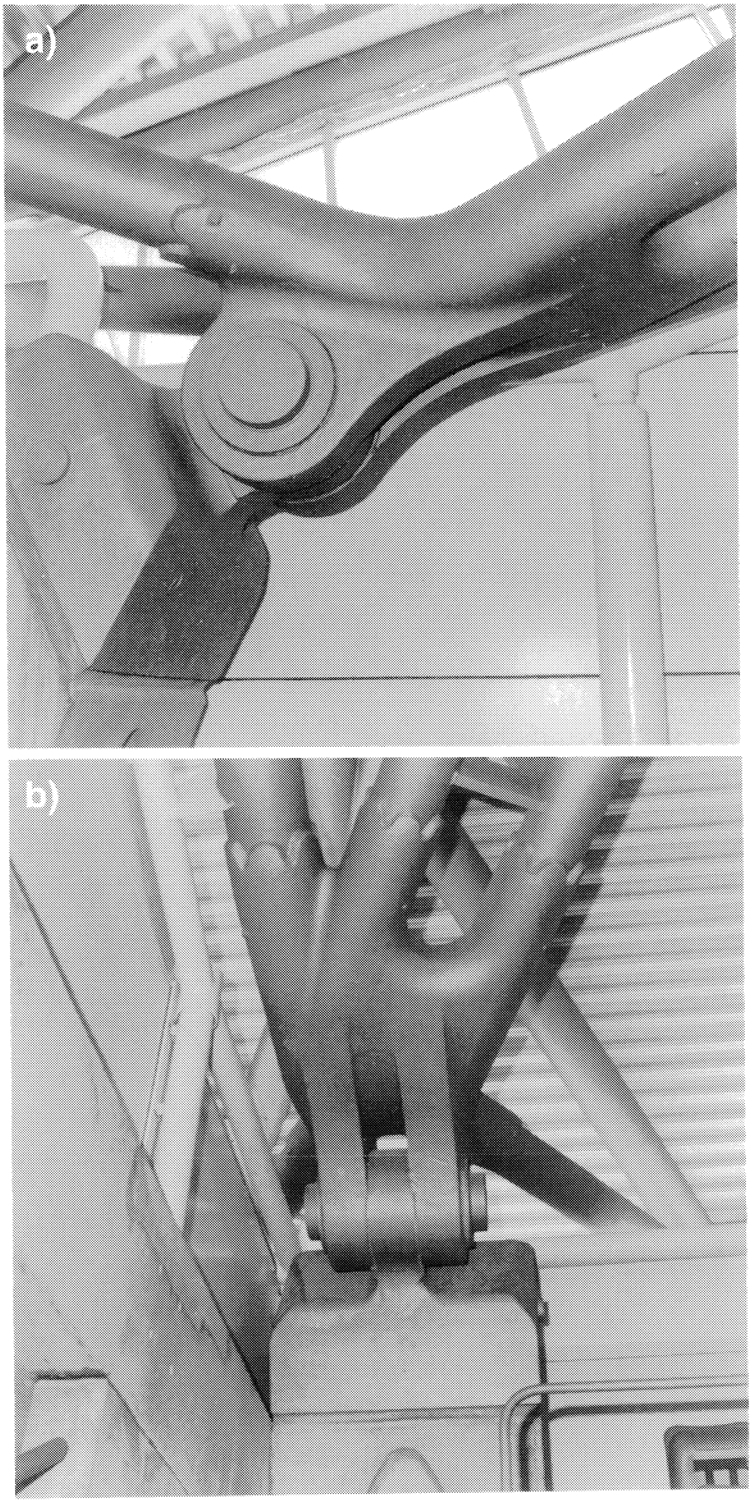


Fig. 3.64 a and b – Steel castings for swimming pool roof (Ponds Forge Sheffield England)

As well as different materials there are also many different casting methods. The chosen method will depend on the type of material, the numbers required, their weight and size, dimensional tolerances and surface finish. If it is decided to use castings, the casting company should be contacted early on in the project so that all of these items can be discussed and an idea of the cost determined.
An over view of the use and application of castings in construction is given in [72].

4 Structures and subassemblies of hollow sections

In this chapter, a number of structures and arrangements for subassemblies will be shown pointing out the basic arguments for the use of hollow sections in them. However, it has to be underlined at this point that an attempt only to minimize mass while designing a structure is a wrong concept. Other items as listed below may outweigh the cost of the material considerably:

- The selection of proper structural arrangements conducive to a rational and economical fabrication and erection
- The exploitation of low drag coefficients of hollow sections in special cases like wind loading or water current environments
- The choice of the right type of corrosion protection and procedure to attain adequate durability against fire and corrosion if required

4.1 Beams and girders

4.1.1 Single section beams

Rectangular hollow sections with their long sides set in the plane of bending are used normally for uniaxial bending load, as they have maximum bending stiffness I_{max} in this position. It is evident that I or H sections offer more economical solutions for uni-axial bending. For multiaxial bending however, square or circular hollow sections may be more suitable.

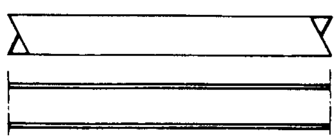


Fig. 4.1 – Single section beams

In general single section beams are the cheapest solution for small spans. Hollow sections offer good shear properties when used for short spans without the need for any stiffening. For longer spans they possess excellent lateral stability characteristics [2] and also have a high resistance against lateral transvers loads.

4.1.2 Lattice girders (trusses)

Considering the excellent statical properties of hollow sections under axial compression and tension as well as their efficiency for lateral stability due to significantly large torsional stiffness, they are highly suitable for application in lattice girders (see Fig. 4.2). Lattice girders are fairly simple to design comprising an upper and a lower chord member with a lattice consisting of bracing members. The chords may or may not be parallel.
The characteristics of a lattice girder are mainly given by the span l_0, the depth h, the lattice geometry and the distance between the joints. The depth h is determined by consideration of the span, the loads and maximum allowed deflection. Truss forces may be lowered by increasing the depth h, which also increasing the lengths of the web members at the same time. The ideal span to depth ratio is usually found to be between 10 and 15.

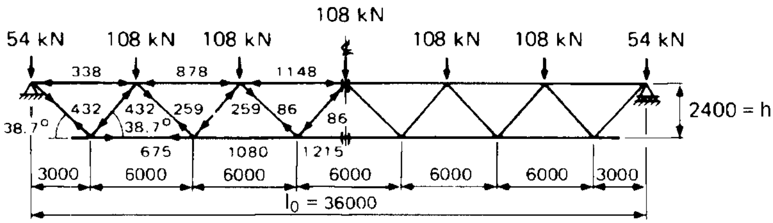


Fig. 4.2 – Warren truss showing applied loads and member forces

The joints can be constructed either by welding the truss chords and bracings directly or by welding gusset plates to the members and joining them by bolting (Fig. 4.3). The design and calculation procedures for uni-planar trusses are illustrated in [1, 3].

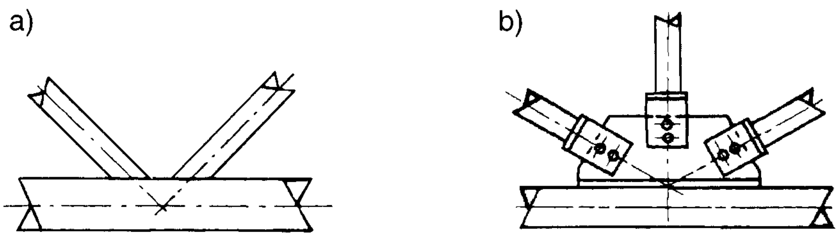


Fig. 4.3 – a) Welded and b) bolted truss joint

Warren type girder (Fig. 4.4a)

These girders provide not only an architecturally aesthetic but also an economical solution. Optimisation studies regarding the lattice webs have indicated that the favourable angle of inclination θ between the chord and bracing lies in the range of 40° to 50°. However, a lower value of θ may decrease the number of joints minimising also the associated fabrication costs. It is further noted that a minimum θ of 30° is recommended in order to assure sufficient weld penetration at the heel of the web member.

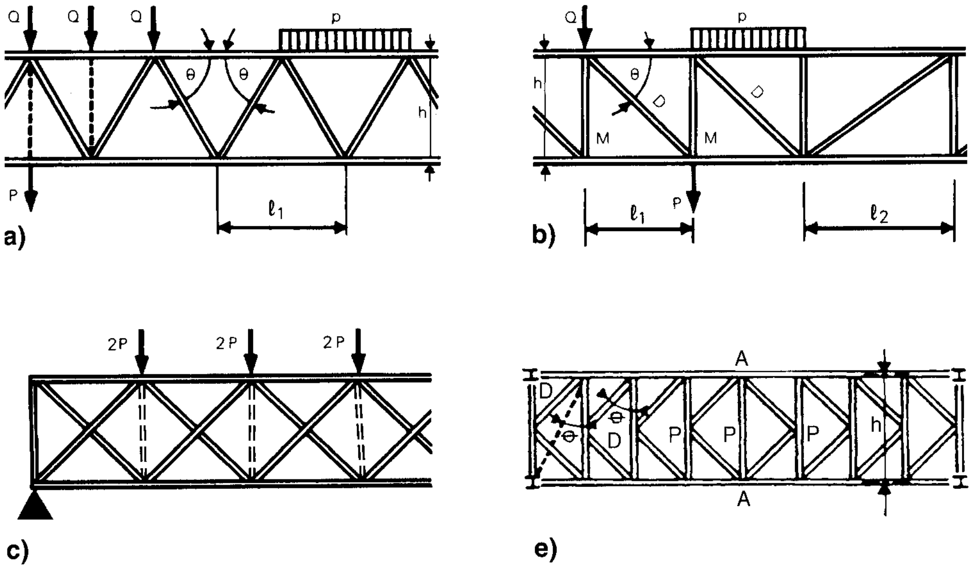
In case it is required to support all load points in a chord e.g. to eliminate chord bending moments, additional vertical members can be added (see Fig. 4.4a).

A further advantage of welded Warren trusses consists of the use of gap joints, which are also more economical from the viewpoint of fabrication. This type of truss also offers more open space for the arrangement of service pipes, cables and other facilities.

Pratt type girder (Fig. 4.4b)

The joints of a Pratt type girder consist of a vertical and an inclined bracing connected to the chord. This arrangement however results in increasing the number of the bracing members and hence also the number of joints compared to that of the Warren type girder. This makes the Pratt type a less economical solution due to the increased fabrication and corrosion protection costs.

Within the feasibility range it is important to note that sometimes an economical design is possible by this lattice arrangement, where the main compression loads occur in the shortest members i.e. the verticals.



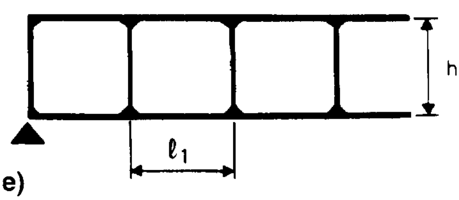


Fig. 4.4 – Various types of lattice girders
a) Warren type girder and modified Warren with virticals
b) Pratt type girder
c) Howe type girder
d) "K"-lattice girder
e) Vierendeel girder

Howe type girder (Fig. 4.4c)

The girder of this type consists of "X" lattices with or without vertical members. One further form of this girder type may offer special advantage regarding the transport and assembly on site, when two half depth girders are welded complete in the workshop and then bolted on site at the intersections of the bracing members (Fig. 4.5).

Girder with "K" lattices (Fig. 4.4d)

Due to the relatively large number of structural members and joints, the fabrication time and the labour costs are high, when this girder type is used. However, it may be selected for particularly deep girders, since a reduction of the effective lengths of the bracing members can be attained in this case. Further, as for the How type girder, the fabrication procedure with two halves of the girder can also be applied here.

Vierendeel girder (Fig. 4.4e)

Vierendeel girders proposed by Arthur Vierendeel in 1896 are manufactured by connecting chord to bracing members nearly always at an inclination of 90°. The bracing members of the T joints are subjected to substantial bending moments as well as axial and shear forces. The calculation procedure for the design of this type of girder in welded form is shown in [1, 3]. This girder layout in general tends to be less structurally efficient than the ones described previously. It does, however, have significant advantages in certain applications, e.g. in overhead motorway sign gantries, where the electronic signs are usually square or rectangular modules which can be fitted between the vertical bracings without any visual obstruction.

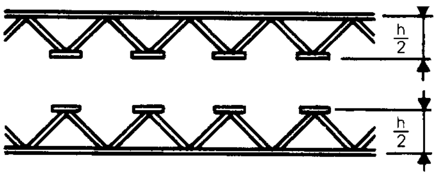


Fig. 4.5 – Howe girder with two half depth girders

Triangular and quadrangular girders (Fig. 4.6a and b)

Triangular and quadrangular girders consist of three and four chord members respectively, with joints of TT, XX and KK types (Fig. 4.7) are designed for multiplanar loadings, i.e. the joints have a "spatial" type of strength to withstand loads and bending moments from all directions. Due to their inherent stability, these girders do not require any external bracing of any kind and hence constitute autonomous bearing elements. Further, the extremely high rigidity of these girders offers easy handling in the workshop as well as during transport and erection. All these qualities lead to their frequent use as structural components that are inclined or vertical (masts, stanchions, towers, etc.) as well as horizontal ones (buildings, footbrides, etc.).

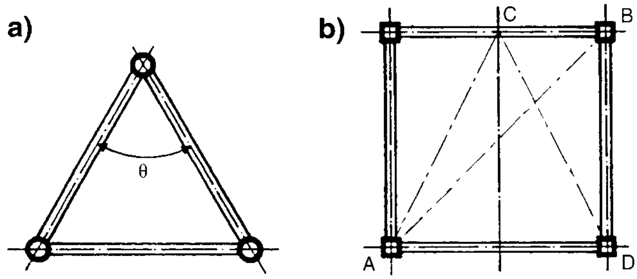


Fig. 4.6 – a) Triangular and b) quadrangular girder

The ratio of the span "l" to the depth "h" of these girders lies usually between 15 and 18. The width is often made equal to the depth; however, the transport facilities represent the limiting factor as far as welded subassemblies are concerned. Triangular girders can be designed with apex pointing upwards or downwards, both used often in roof structures (Fig. 4.8). The former leads to the saving of the depth of the girder on the total height of the building, while the lateral faces are most suitable for providing natural lighting.

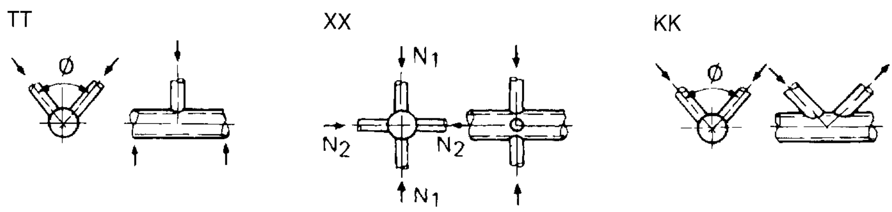


Fig. 4.7 – Multiplanar joints

The latter is ideal for applying cladding directly to the chords; if purlins are needed, they can be of the continuous type. This configuration is also very suitable for gangways or footbridges (Fig. 4.9).

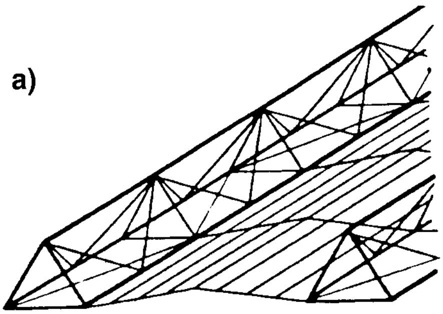


Fig. 4.8 – Triangular girders with apex pointing a) upwards b) downwards

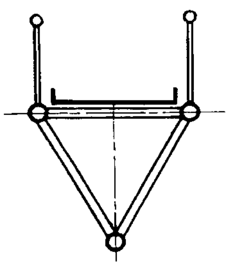


Fig. 4.9 –Triangular girder for gangway or footbridge

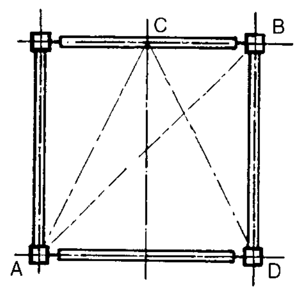


Fig. 4.10 – Quandrangular box girder as a combination of two lateral single plane girders

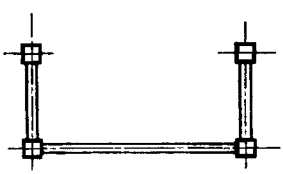


Fig. 4.11 – Three-faced quadrangular girder for a footbridge (U-frame)

Figures 4.6b and 4.10 show two design possibilities for quadrangular girders – one entirely welded and the other made of two lateral single plane girders, which are subsequently joined by two bolted lattices. In both cases cross section braces can be provided along AB or AC and CD (see Fig. 4.10). An alternative with only three faces can create an ideal shape for footbridges (Fig. 4.11).

References [1, 3] contain the design and calculation procedures for multiplanar girders.

4.1.3 Arched girders

Arched girders can be classified into the following two main types:

- Girders springing directly from their abutment (Fig. 4.12a)
- Girders raised on legs and forming the cross member of a portal frame (Fig. 4.12b)

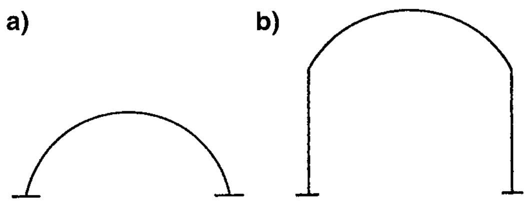


Fig. 4.12 – Arched girder types a) springing directly from abutments, b) standing on legs and forming the cross member of a portal frame

Taking account of the simple fabrication, single section arches are usually circular, whereas lattice girder arches can be circular, elliptic or parabolic. The required bending procedure of single hollow sections for the fabrication of arches has been described in Chapter 3.3.

Considering the statical system, they are designed with the following basic configurations

- 3-pinned arch (Fig. 4.13a)
- 2-pinned arch (Fig. 4.13b)
- Fixed arch (Fig. 4.13c)

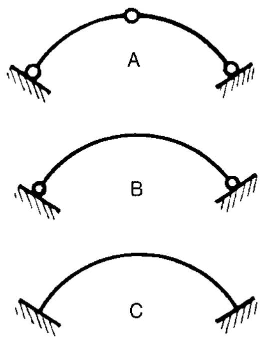


Fig. 4.13 – Statical systems for arches

The choice of the configurations depends principally on the soil properties for the girder springing from the abutments, whereas the raised arches may require a tie or a tie-strut to balance the thrusts at the springing points (Fig. 4.14).

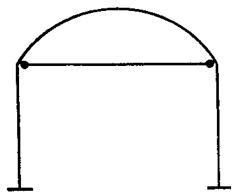


Fig. 4.14 – Raised arch with a tie-strut to bear the springing thrust

As shown in Fig. 4.15, the bracing members ensure the stability of the vaults. Due to their excellent lateral stability behaviour very wide spans are possible without any interconnecting bracing between the arched girders.

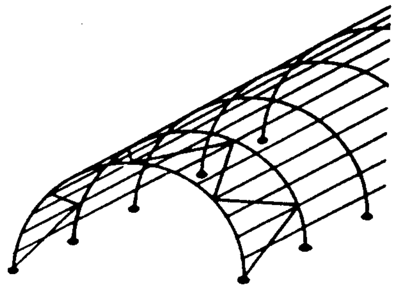


Fig. 4.15 – Wide span vaults with interconnecting bracings between the arched girders

A typical structural arrangment of a single hollow section arch is shown in Fig. 4.16. Arch member "B" is welded to a baseplate "A", which can be attached to a tie member "C". The baseplate "A" is drilled for bolting to the concrete base or to the head plate of the raised hollow section leg of a frame. Fig. 4.17 illustrates the principle of the connection of a lattice arch to the concrete base either fixed (A) or pinned (B).

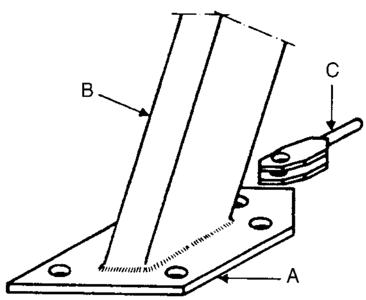


Fig. 4.16 Connection at the foot of a single hollow section arch

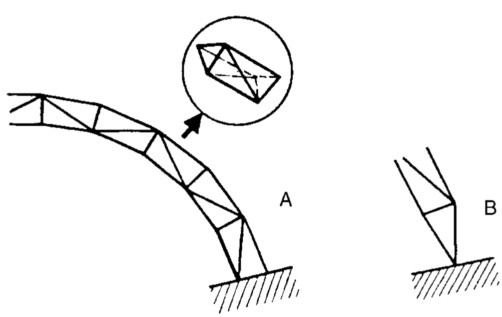


Fig. 4.17 – Principles of connection at the foot of lattice arch

4.2 Columns

Due to the superior buckling behaviour of hollow sections to that of open sections, which is demonstrated by the larger moment of inertia about the weak axis I_{min} in Fig. 2.4, the choice of hollow sections for columns in buildings as well as for other structural elements under compression in many other engineering sectors [6] has become very frequent. The design and calculation procedure for hollow section columns has been adequately discussed and described in [2].

Fig. 4.18 shows as an example of how much material mass can be saved when CHS/RHS is used for a column with a buckling length of 3 m instead of open sections.

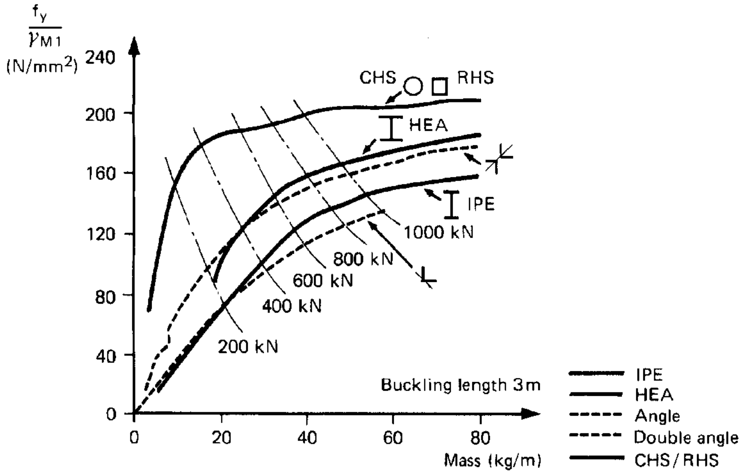
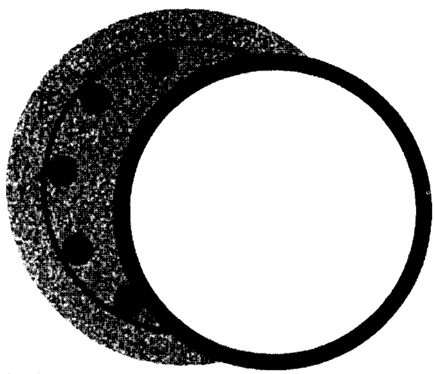


Fig. 4.18 – Comparison of the masses of hollow and open sections under compression in relation to loading

Further, the economical competitiveness of a hollow section column against a reinforced concrete or a UC column regarding the increase of revenue achievable by maximising the net lettable floor area in a multi-storey building is illustrated by Fig. 4.19.

a)
Overall dimensions of Circular columns



CHS
323.9 mm

b)
Overall dimensions of Square Columns

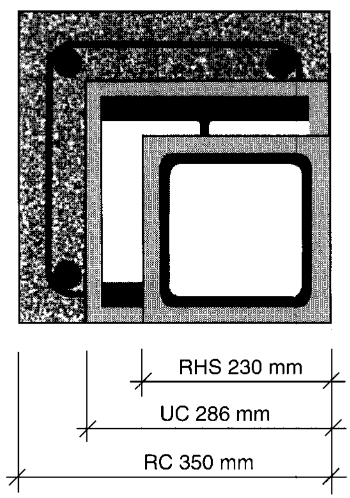


Fig. 4.19 – Relative space occupied by internal columns for a) CHS and b) RHS option against a reinforced concrete (RC) or a UC column

The structural arrangement for a column has to be made based on the end moment at the foot. This can comprise a single leg (Fig. 4.20) or a lattice construction (Fig. 4.21).

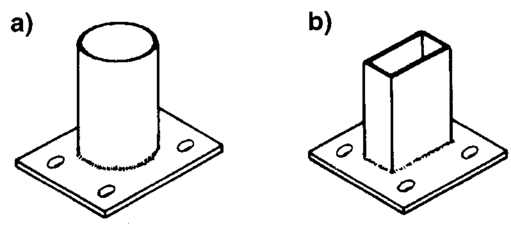


Fig. 4.20 – Single leg column welded to base plate a) CHS b) RHS

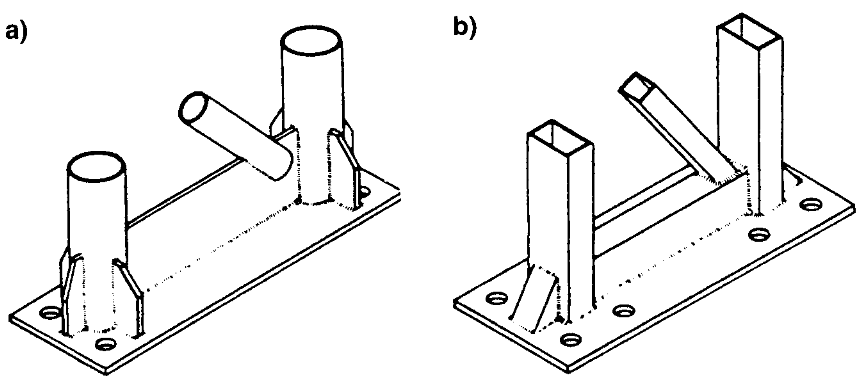


Fig. 4.21 – Lattice column welded to base plate a) CHS b) RHS

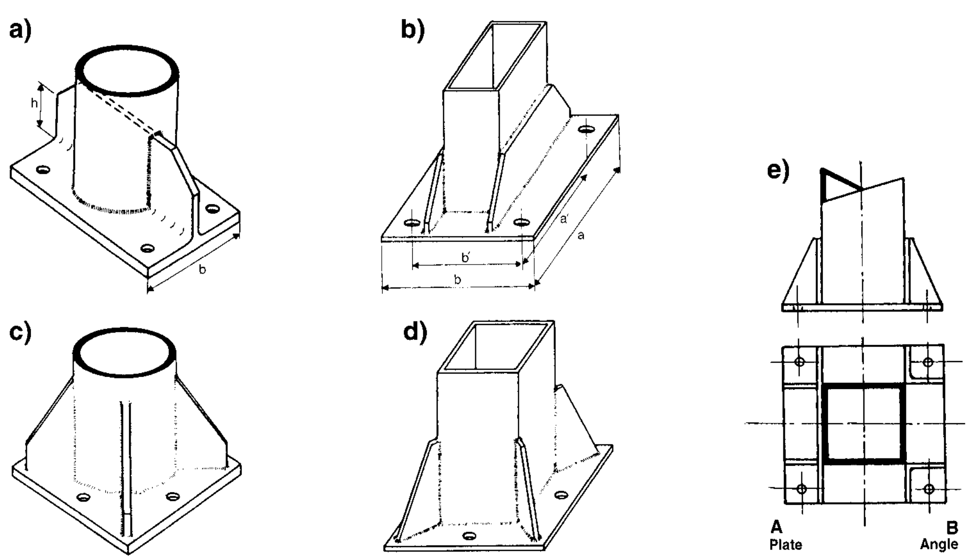


Fig. 4.22 – Single leg columns with various stiffener arrangements a) and b) for uniaxial bending c) through e) for biaxial bending

Column bases consisting of a single end plate welded to the foot of the column, as shown in Fig. 4.20, are the simplest solution, even if a fairly thick plate is required. The column ends are saw-cut or milled to a plane surface to bear evenly on the base plate. These are suitable for small moments, whereas various arrangements of stiffeners can be applied to the column-base plate connection, when larger moments have to be withstood. The stiffeners can consist of flat plates or angles, as illustrated in Fig. 4.22 a to d and also Fig. 4.21. The locations of the stiffeners depend on the directions of the bending moments to be resisted. However in principle, stiffeners should be avoided, if possible, by using a thicker base plate.

Column bases can also be made adjustable (see Fig. 4.23) allowing for height and out of plumb adjustments. When specified, an effective hinge in a particular plane can also be incorporated into the column bases by means of a pin or other devices (Fig. 4.24). A special arrangement for connecting an internal rain water down pipe at the foot of a hollow section column is shown in Fig. 4.25. An elbow, in plastic or cement, is bedded into the concrete foundation and connected to the rain water down pipe. In this case, measures are to be taken for the protection of the inside of the hollow section against corrosion. This can be done either by galvanising the hollow section or by means of a seal at the head and the foot of the column.

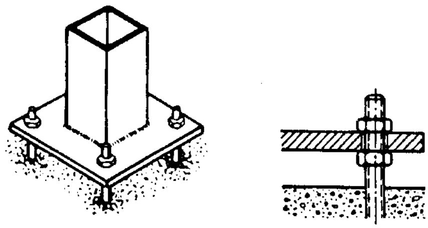


Fig. 4.23 – Column base with adjustable support

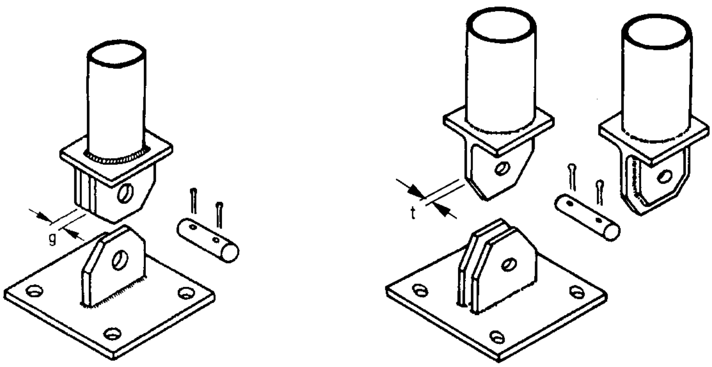


Fig. 4.24 – Column base with hinged support

4.2.1 Concrete filled hollow section columns

Composite columns made by filling hollow section columns with concrete, with or without reinforcement bars (Fig. 4.26), are a further development, which combines the merits of both concrete and steel. They have a higher ductility than concrete columns and at the same time column connections may be constructed following design knowledge about structural details in steel. Concrete filling does not only add significantly to the load bearing capacity of hollow section columns, but also increase their fire resistance considerably. Design and calculation procedures are given in [4, 5].

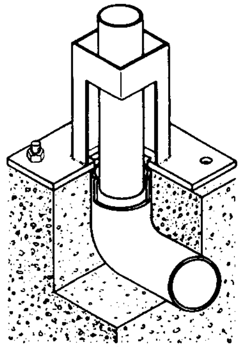


Fig. 4.25 – Column base with rain water down pipe

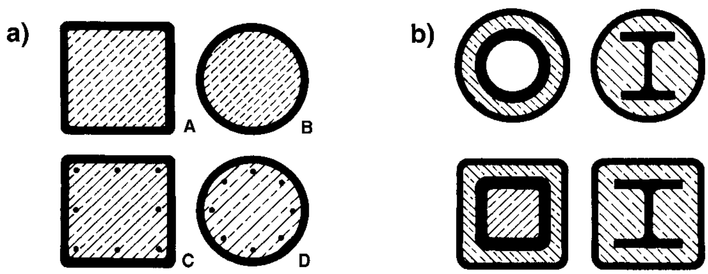


Fig. 4.26 – Cross sections of concrete filled hollow sections
a) with reinforcement bars b) with reinforcing profiles

The qualitative aspects of the composite hollow section columns, which give them special preference to architects and structural engineers, are listed below:

- The pleasant appearance of the slender columns is retained, while their load bearing capacity rises without enlarging the external dimensions.
- The preference of many architects to present visible steel in their design is feasible. The possible colouring of the surface area for aesthetic reasons and also for corrosion protection by spray or paints causes low costs due to the smaller surface area of the slender columns.
- As the hollow section acts as permanent shuttering, any additional shuttering as for solely concrete columns is not necessary; this adds to the economy for fabrication. Due to the concrete, confined and held by the hollow section, there is no splitting away even if the ultimate concrete strength is reached.
- Composite hollow section columns, especially with a corresponding percentage of reinforcing steel bars, can in some circumstances reach more than 90 minutes of fire resistance time without any additional external fire protection measure. This can save the overall expenditure for structures considerably, particularly for multi-storey buildings.

4.2.1.1 Concrete filling of hollow sections

Concrete filling of hollow section columns, circular, square or rectangular, can be conducted in workshop or on site depending on the facilities available.
The following recommendations are made regarding the preparation of steel hollow sections, reinforcing bars and concrete [7, 8] for the concrete filling operation.

Hollow section columns

1. Concrete filled hollow section columns must have small vent holes drilled in the walls in order to prevent the column from bursting under the steam pressure generated by vaporisation of the dehydration water locked in the concrete fill during a fire. They are preferably in pairs for each single length and at each floor level in the case of columns for multi-storey buildings. The distance of the holes has to be between 10 to 20 cm from the top and the bottom of the columns, where they are closed by steel plates. Intermediate holes have to be drilled if the column length exceeds 5 m (Fig. 4.27). The diameter of the vent holes should not be less than 20 mm.
2. Load transfer through concrete filled columns of a multi-storey building can take place simply through head plates (Fig. 4.28).
3. The inside surface area of hollow sections has to be free from water and other impurities e.g. oil, grease etc. prior to filling them with concrete. However no special treatment of the inner surface is necessary.

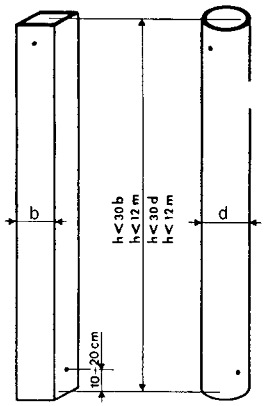


Fig. 4.27 – Length of concrete filled columns and arrangement of the vent holes

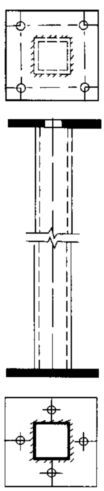


Fig. 4.28 – Concrete filled hollow section columns with head plates

Reinforcement bars

1. The reinforcement bars, with 4% of the concrete sectional area as a maximum, should be taken into account for the calculation of the load bearing strength of a concrete filled hollow section column at the service temperature. When exposed to fire, this value may be exceeded; however, the calculation for the load bearing strength has to be made with 4% reinforcement for the service temperature design.
2. For practical reasons (installation and concrete casting), it is not recommended to use reinforcement for sections smaller than 200 mm (diameter or width) for site filling and 160 mm for filling in a workshop.
3. The reinforcement bars should be covered by the concrete depending on the maximum size of aggregate "D" measured by the sieve. The gap between the bar and the internal surface of the hollow section wall should be between 1.5 D and 2.0 D (see Fig. 4.29), with the maximum concrete cover lying between 2.5 and 5 cm. Special cover of the reinforcement may be necessary for the fire design.

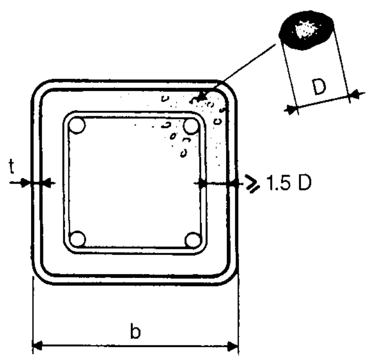


Fig. 4.29 – Concrete cover for the reinforcement bars in concrete filled hollow section columns

4. Various reinforcement bar and stirrup arrangements for concrete filled hollow section columns are shown in Fig. 4.30.

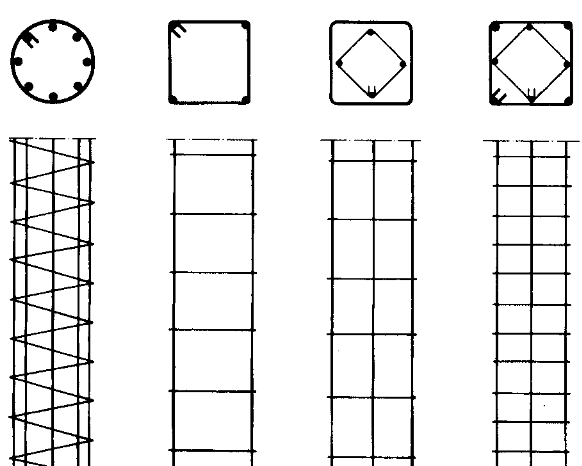


Fig. 4.30 – Reinforcements in concrete filled hollow section columns

Concrete

1. Sufficient plasticity is required for the concrete mix. It is recommended to use a higher sand and cement content (i.e. water to cement ratio is lower) and reduce the maximum aggregate size. Here, special care has to be taken in the case of pumping the concrete into the column from the bottom.
2. Maximum aggregate diameter "D" shall be as follows:
   - smaller than 1/8 of the internal dimension of a hollow section for a column without reinforcement
   - smaller than the fictitious radius r for reinforcement as defined for the smallest mesh, with

     $$r = \frac{a'b'}{2(a' + b')} \qquad \text{(see Fig. 4.31)}$$

   - smaller than half the distance b' between two longitudinal bars
   - smaller than two thirds of the distance between the longitudinal bar and the inside surface of the hollow section.

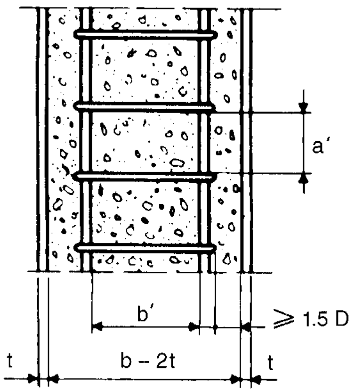


Fig. 4.31 – Arrangement of bars and stirrups in concrete filled hollow section columns

3. The cement usually used is artifical Portland cement. The concrete strength classifications C35, C45 and C55 (28 days strengths ≥ 35 or 45 or 55 MPa) are common.
   The so-called slump test determines the consistency of the concrete; the sinking must be between 7 and 10 cm
   A volumetric ratio of 0.82 between sand and gravel is normally taken.
4. Additives, which may cause corrosion of steel e.g. calcium chloride, have to be excluded. The use of plastifiers and liquidisers are however recommended.

Concrete filling operation methods

After the preliminary preparation of the components e.g. hollow section, reinforcement and concrete, described above, the filling of hollow section columns with concrete is performed in accordance with the following methods. Their applications are however dependent on the conditions described below.

Gravity filling (see Fig. 4.32 through Fig. 4.34)

A funnel is used to fill a hollow section column of less than 4 m height (Fig. 4.32a), whereas a bottom emptying hopper is used for hollow sections of larger dimensions, i.e. d or h > 500 mm (Fig. 4.32b). The filling of fresh concrete should be made in steps of 30 to 50 cm, where it is vibrated by means of pokers immediately after being laid (Fig. 4.33).

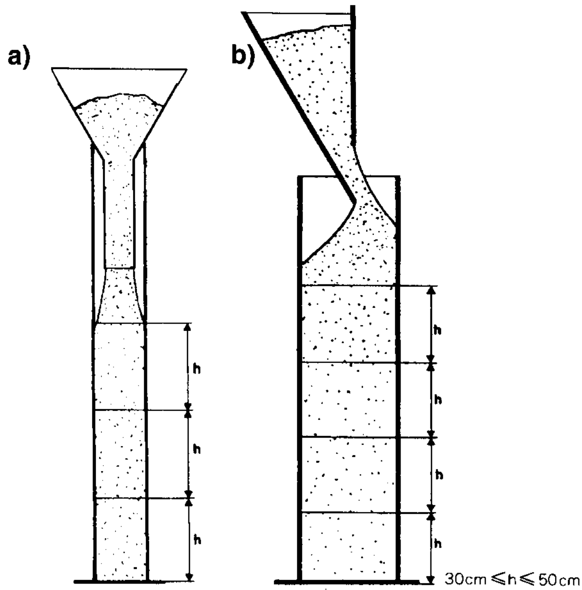


Fig. 4.32 – Gravity filling with concrete mix by means of a) funnel b) bottom emptying hopper

Further, it is recommended to use a funnel with a variable neck length in order to avoid the segregation of the concrete mix (Fig. 4.34).

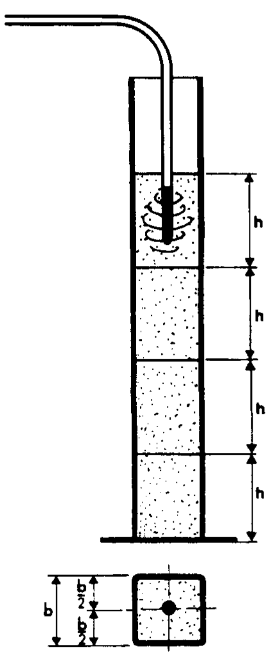


Fig. 4.33 – Vibration of concrete mix by a poker

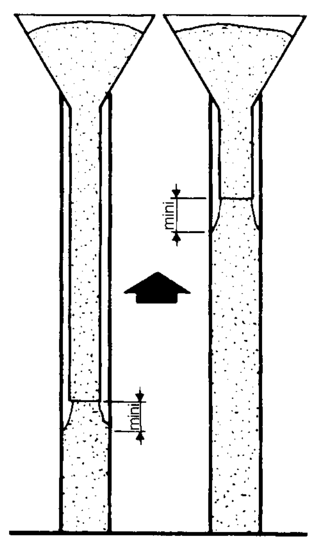


Fig. 4.34 – Gravity filling with concrete mix by a funnel with variable neck length

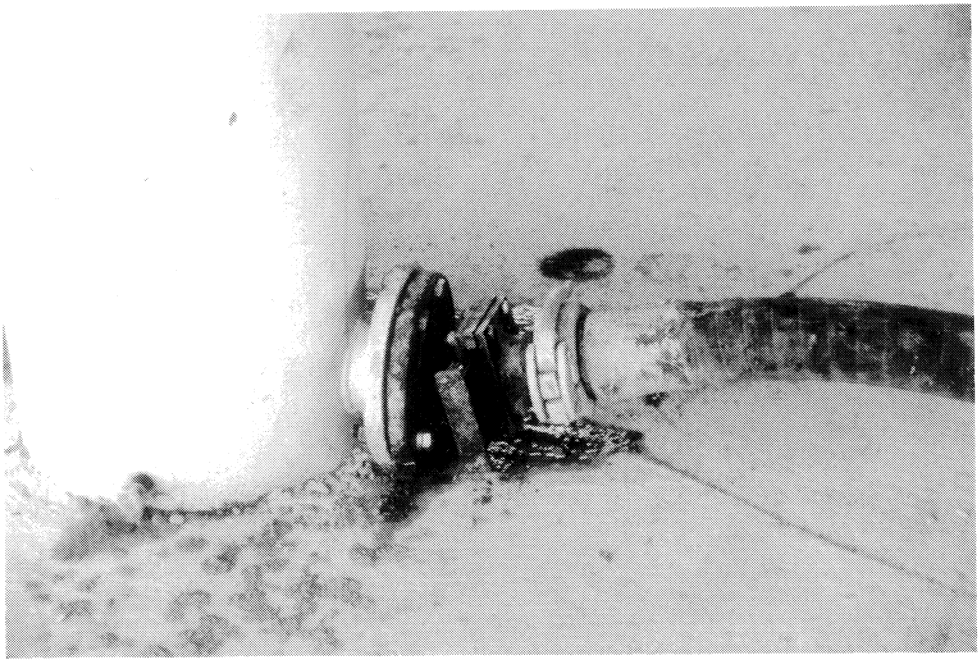


Fig. 4.35 – Pumping concrete into a CHS column from the bottom

Fig. 4.36 – Controlling the concrete filling during pumping by hitting the column with a hammer

Concrete pumping

This method consists of pumping concrete in the hollow section through a flexible pipe with its end either closely above or lower than the concrete surface. A thorough mixing of the concrete components is favoured in the latter case.

Concrete mix is sometimes pumped into the column from the base, which necessitates a hole at the bottom (Fig. 4.35). Concrete can be pumped up to a number of floors in a building.

Vibrators
Vibration during the concrete filling is possible using vibrators from outside held against the hollow section walls or by pokers inside as shown in Fig. 4.33.

Check to ensure concrete filling
This is done by hitting a concrete filled column at close intervals by a hammer from the outside. Fig. 4.36 shows a man checking the level of the rising concrete by hitting the column with a hammer and controlling the concrete filling by the sound. The ensuing sounds reveal possible defects in the filling.

Remedies to remove defects in concrete filling
The defects by inadequate concrete filling can be eliminated by piercing holes, injecting cement and then closing up the openings. In case of any lack of filling at the ends, it is recommended to level them off by means of compensated setting mortars (Fig. 4.37).

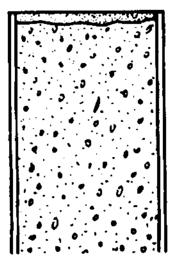


Fig. 4.37 – Levelling off the inadequately concrete filled ends of a hollow section column by mortars

Connections of concrete filled hollow section columns from floor to floor in a building with regard to concrete filling
The procedures vary for the site and the workshop installation of columns.
For the installation on site, the filling of each section of column is carried out step by step from floor to floor as the structural work progresses. Concrete filling must be done right up to the planes of connections by overfilling the column and then levelling off with a trowel before the hardening of the concrete.

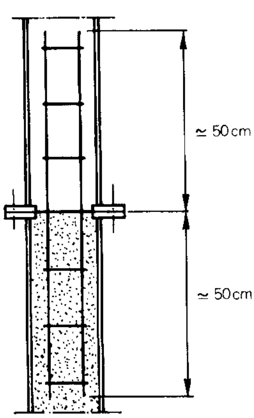


Fig. 4.38 – Inserting light reinforcement through the connection between two column sections

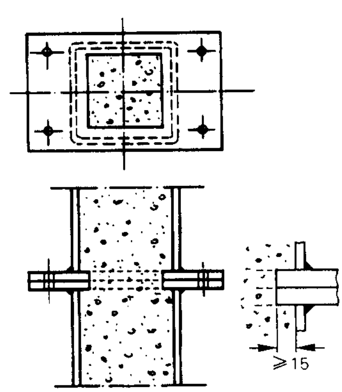


Fig. 4.39 – Steel plates welded to the open ends of the column and then bolted together

In order to ensure the continuity of concrete through different column sections, the following alternatives can be applied:
- Fig. 4.38 shows the method of inserting a light reinforcement through two sections.
- The open ends of the column sections are closed by steel plates, which are bolted or welded together (Fig. 4.39).

The joining of concrete filled columns prefabricated in the workshop can be carried out directly by welding or preferably by bolting on site.

4.3 Space structures

Space structures consist of structural members lying in a number of intersecting planes. This means, each of them belongs to two or more planes, and are predominantly loaded by axial loads. Space structures are manufactured using identical elements designated as modules, which can be linear (Fig. 4.40), planar (Fig. 4.41) and three-dimensional (Fig. 4.42).

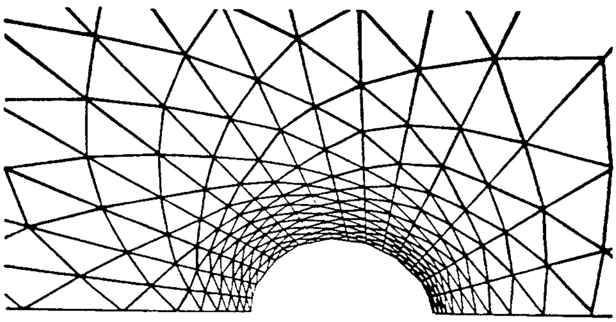


Fig. 4.40 – Space structure with linear modules

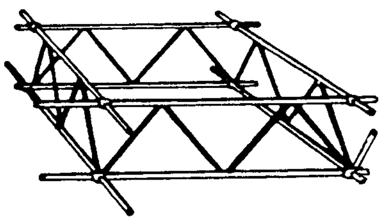


Fig. 4.41 – Space structure with planar modules

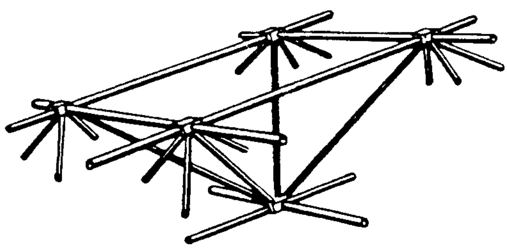


Fig. 4.42 – Space structure with three-dimensional modules

The application of identical elements favours the mass production and the standardisation of the parts leading to their economical industrial prefabrication in a workshop, simple storage and transportation, easy assembly and erection as well as dismantling on site.

A characteristic element of all wellknown systems of space structures is the joint, where the structural members are connected to one another by bolting, less often by welding and clamping. The connection in total i. e. the joint and the details of the member ends, influences the economy of a space structure significantly. A number of prefabricated connectors, which are available in the market, is shown in Fig. 4.43. Prefabricated connectors and members are transported to the site and assembled there on the ground. The assembled structure is then raised by a crane and placed on the supporting structure.

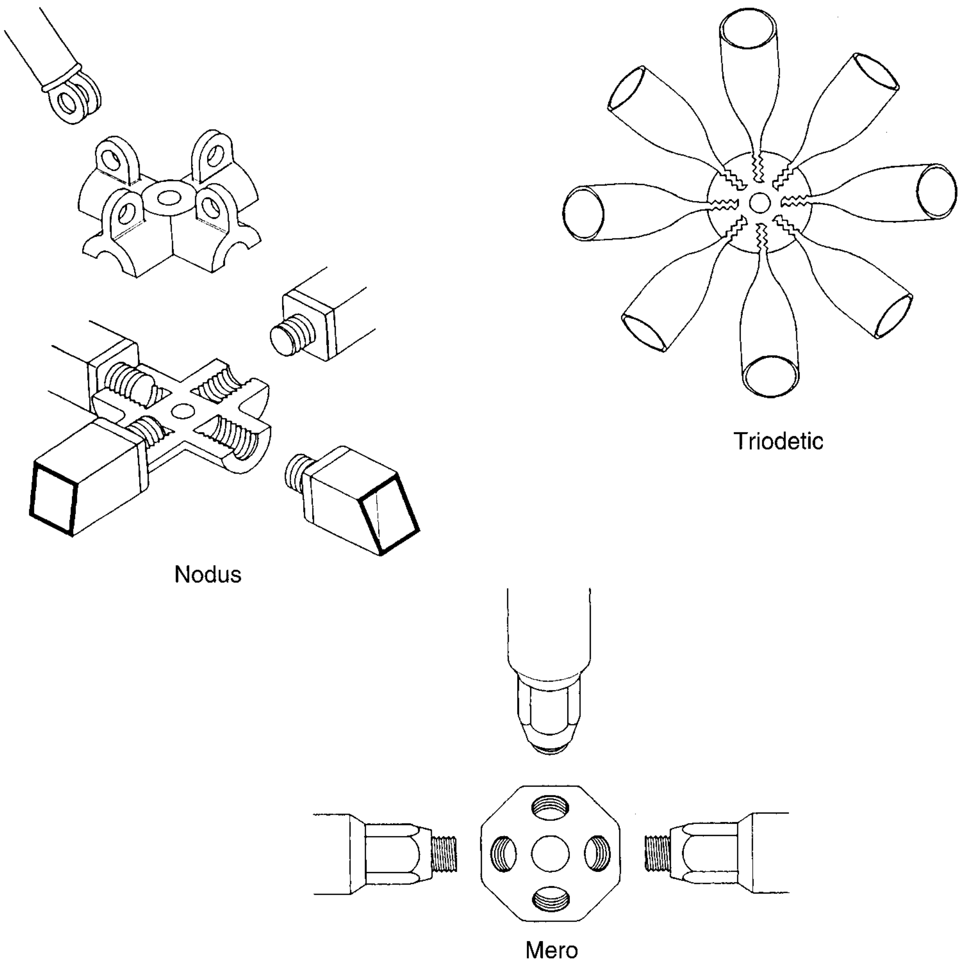


Fig. 4.43 – Some prefabricated connectors

The larger the span, the more justified is the adoption of a space structure. A certain disadvantage for the application of space structures may be the relatively long time and high labour costs for the design calculations, which should not be underestimated even when computers are used.

Due to the following advantages, the structural hollow section is the prime cross section used for lattice space structures:

– High buckling and torsional strength and excellent structural behaviour due to an even distribution of material around the axis (especially perfect for CHS) without the weak planes pertaining to open sections
– Due to a higher permissible load than for open sections, substantial saving of material for a given space structure
– Due to reduced surface area, less maintenance work e.g. painting or cladding for fire and corrosion protection
– Easy jointing of the structural elements by welding or bolting
– Aesthetic appearance preferred by many architects

4.4 Connections

The connections in a hollow section structure play a vital role with regard to the economy of design and fabrication. As has already been mentioned in Chapter 3, the connection types, besides welded and bolted, can be classified into two main classes (Fig. 4.44):

- Direct connections, where the members are directly joined to one another
- Indirect connections with the members are connected indirectly through gusset or end plates

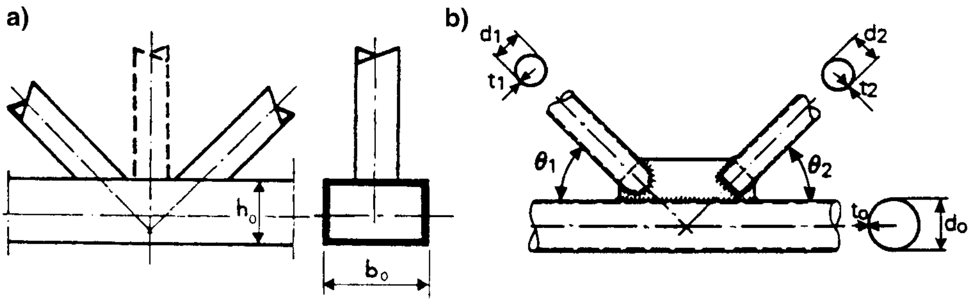


Fig. 4.44 – Welded hollow section connection a) members directly welded to one another b) members joined through gusset plates indirectly

In general, structural integrity and fabrication economy speak in favour of the direct connections; in this case, the structural integrity is statistically higher because of the single load transfer from one hollow section to the other, while for the indirect connections the transmission of load takes place twice – first from one hollow section to the plate and then from the plate to the other member. The fabrication work, consisting of welding or bolting or the combination of both, is also less voluminous in the case of direct connections.

4.4.1 Beam-to-column connections

The realisation of the simple design of a beam-to-column connection necessitates the provision of a certain degree of flexibility or rotation capacity in order to accomodate beam end rotations with the deflection of the beam. The fabrication of the bolted connections consists almost always of a combination of welding and bolting.

A wide variety of beam-to-column connection details have been used in practice and some of them are illustrated in Figures 4.45a through 4.45f. In all cases the columns are built with hollow sections, while the beams consist of either rolled I or hollow sections.

Figures 4.45a) and b) illustrate two very common connections, where a flat plate or a tee section is welded to the column face with the beam web bolted to them. An angle can also be welded to the column in order to support the beam while erecting.

Transmission of very heavy shear loads from beam to column is facilitated by the connections shown in Figures 4.45c) and d). The moment M = Q · e is reacted by the column.

A variety of alternatives of semi-rigid connections with various levels of rigidity are shown in Figures 4.45e) through 4.45f). In Fig. 4.45e), a plate is welded to the face of the column. Further the web of the beam is bolted through two angles to the plate. Fig. 4.45f) contains a detail, where an end plate is welded to the beam. The end plate is then joined to the face plate welded to the column by bolting. It is worth mentioning here that the extended width plates are not required if blind bolting systems (see Chapter 3.4.1) are used.

a)

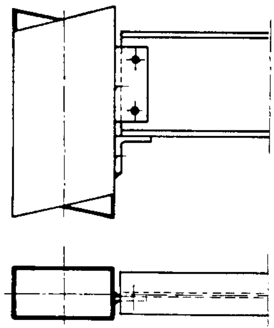
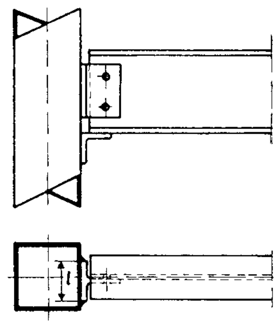


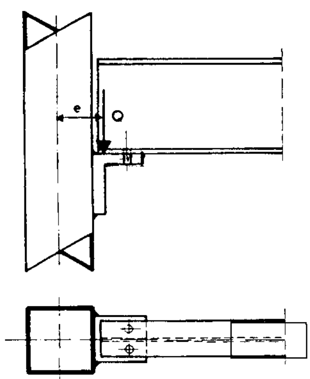
Plate welded to column face and bolted to beam web

b)



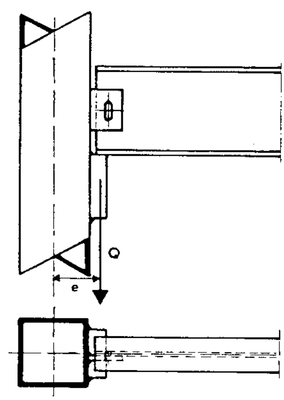
Tee section welded to column face and web of tee section and beam joined by bolting

c)



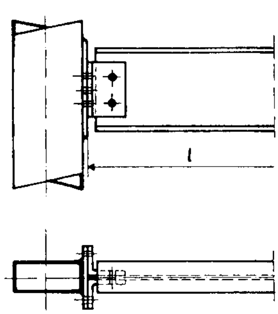
Angle welded to column face and bolted to bottom flange of beam

d)



Plates welded to column face horizontally and vertically, where the vertical plate is joined by bolting to the beam web (long hole) while bottom flange of beam is set on the horizontal plate

e)



A pair of angles bolted to plate, which is welded to column face; angles are further joined to the web of beam by bolting

f)

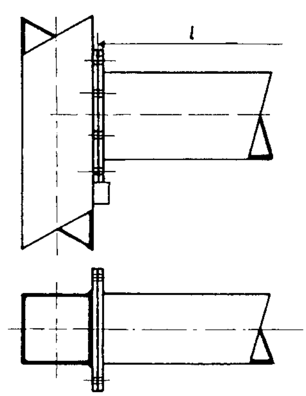


Plate on column face bolted to beam end plate

Fig. 4.45 – Beam-to-column connections fabricated by a combination of welding and bolting

If an I-section beam is directly welded to a RHS column face (Fig. 4.46), the in-plane rigidity of the column flange is very small for β ≪ 1.0 against the concentrated load from the beam flange, so that the column flange may collapse and the column web may buckle under the stress produced in the beam flange during moment loading.

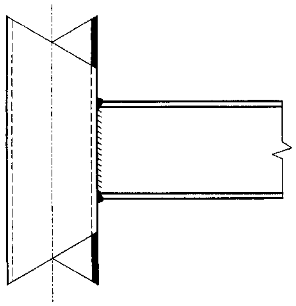
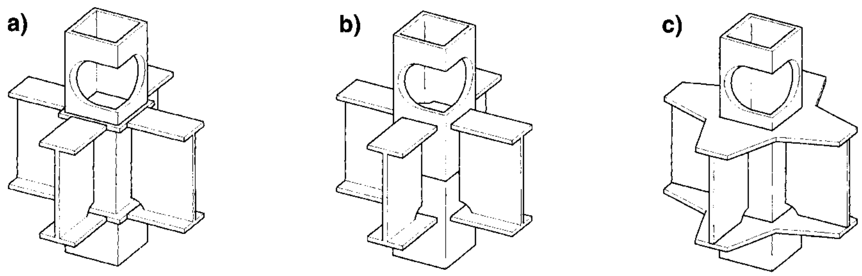


Fig. 4.46 – Beam-to-column connection with members directly welded to each other

In order to avoid this deformation behaviour, it is often necessary to provide stiffeners for moment connections at the level of the beam flanges, through which the flange stresses are transmitted to the opposite side of the column whilst preventing web crippling. The diaphragms developed to act as stiffeners as well as to serve as a mounting platform for beams [14] fall into three types; viz., through diaphragm, interior diaphragm and exterior diaphragm as shown in Fig. 4.47.



(a) Through draphragm (b) Interior draphragm (c) Exterior draghragm

Fig. 4.47 – Types of diaphragms to increase the rigidity of beam-to-column connections
a) through diaphragm b) interior diaphragm c) exterior diaphragm

Figures 4.48a) and b) illustrate connections, where the beams are continuous and extend on both sides of the column. They can be designed to transmit very heavy loads and bending moments.
Fig. 4.49 shows an interesting Swedish development for a very economic connection with a continuous RHS column and a "welded top hat" section as a simply supported beam. The beam support comprises an end plate, welded on to the beam, which is carried on a cleat on the column by bolting. The bolts are located in the extended end plate outside the hollow section column. The floor slab can be laid on the bottom flanges of the beams (Fig. 4.50). In this position, there are no beams protruding below the floor. The built-in beam is protected against fire by the floor slab, which means that special fire protection can be wholly or partly omitted. A similar system, "Slimfloor" is also produced by British Steel – Sections and Plates. Fig. 4.51 shows simpler supports for top hat beam to hollow section columns.

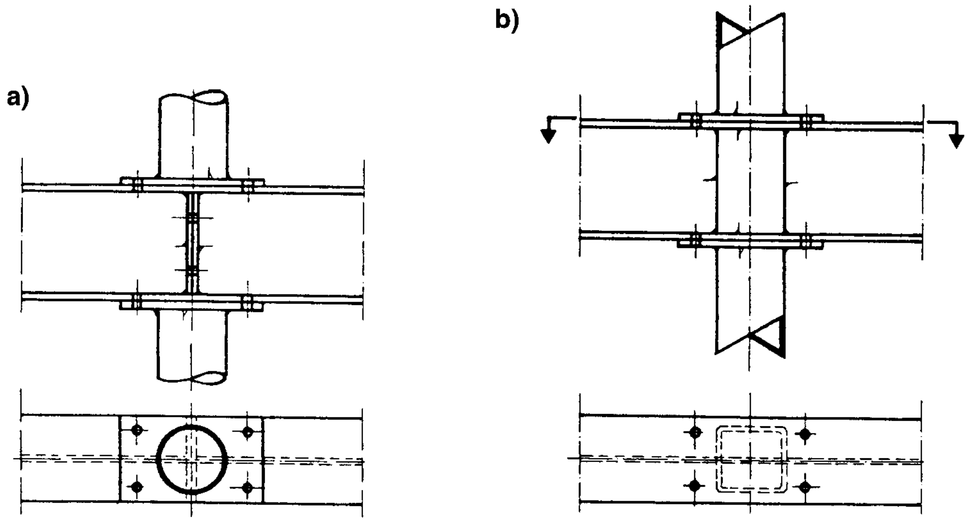


Fig. 4.48 – Beam-to-column connection with continuous beam extending on both sides of column in a) CHS b) RHS

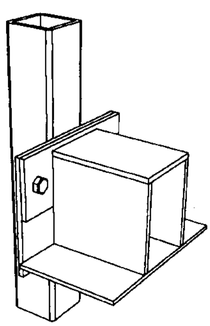


Fig. 4.49 – "Welded top hat"-beam support to RHS column

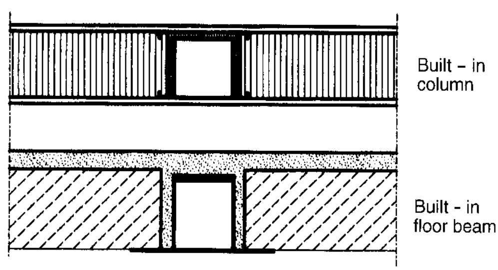


Fig. 4.50 – When beams and columns are built into floors and walls, they do not obstruct space and require little or no additional fire insulation

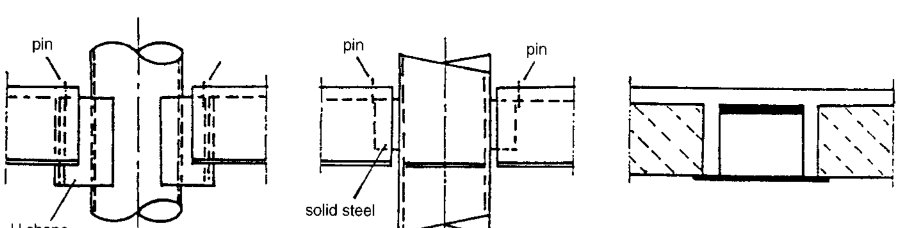


Fig. 4.51 – Simple support for top hat beams to CHS and RHS columns

4.4.1.1 Beam-to-column connections with concrete filled hollow section columns

Connections for concrete filled hollow section columns to beams are usually similar to those for ordinary hollow section columns.

In the connections of Figs. 4.52a) and b), the compressive components of the moments are transmitted directly to the steel with the concrete behind it. The vertical force in the connection goes only to the steel. Its further transmission into the concrete can be only achieved through friction at the steel / concrete interface.

Connections which carry the load by the steel of concrete filled hollow section columns can only be used for smaller loads. For higher loads, a compound arrangement as shown in Fig. 4.53, is applied. A dowel is welded to the face plate on the column. This is then inserted into the hollow section through a drilled hole. Finally the hollow section column is filled with concrete.

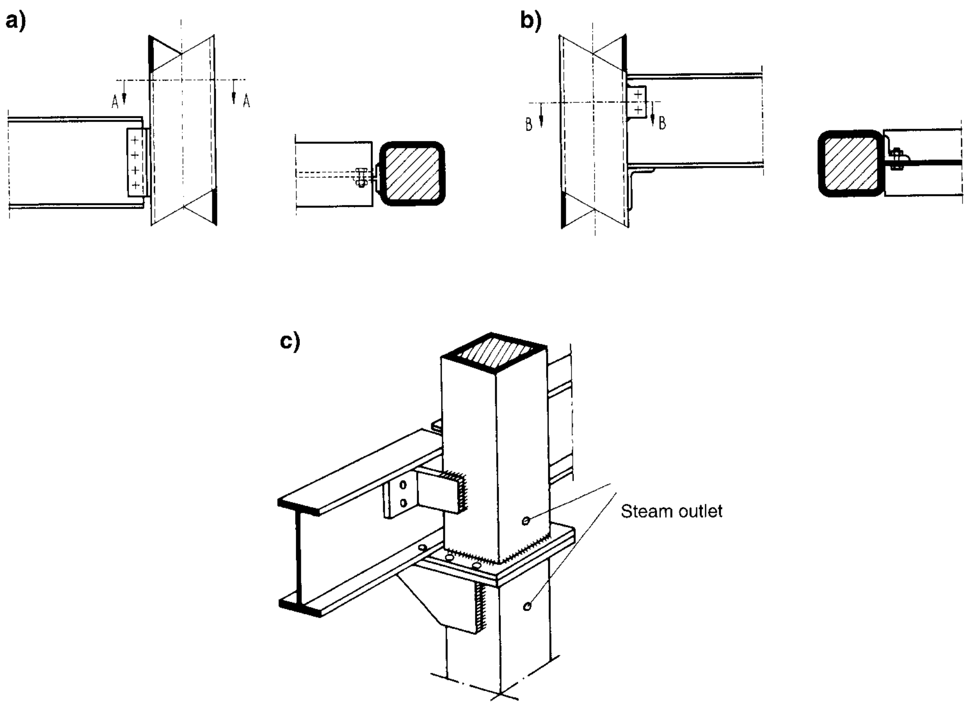


Fig. 4.52 – Beam to composite column shear connections a) tee section welded to column face and then joined to beam web by bolting b) angle welded to column face and then joined to beam web by bolting c) an eccentric connection

Figures 4.54 and 4.55 show two design solutions with two further types of construction for load transmission; a) steel collar b) connection plate inserted into the concrete filled hollow section column.

The first one is a patented construction of a Swiss firm, while the second one has been developed through investigations in Germany [15] for very high beam shear loads.

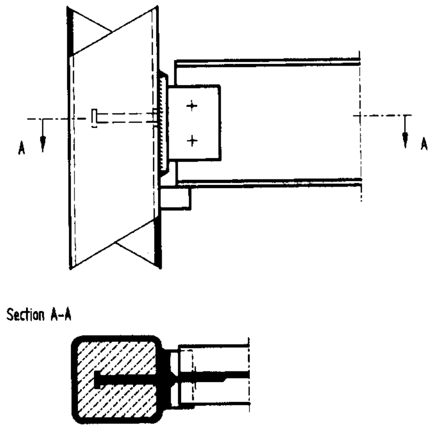


Fig. 4.53 – Load transmission into concrete core by inserting a dowel or stud

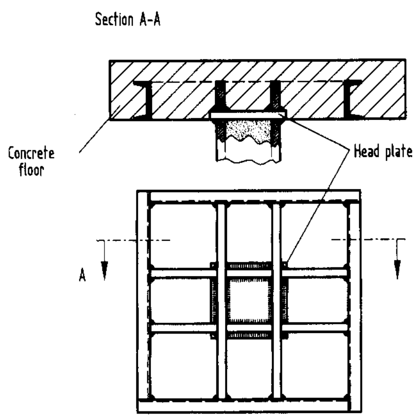


Fig. 4.54 – Load transmission with steel collar

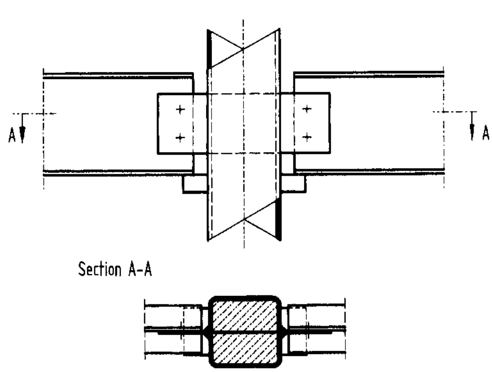
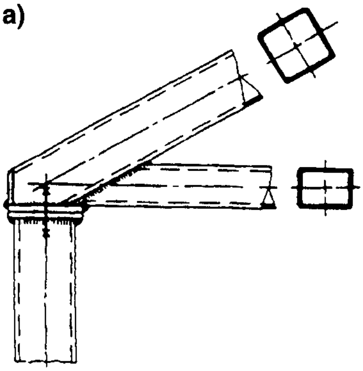


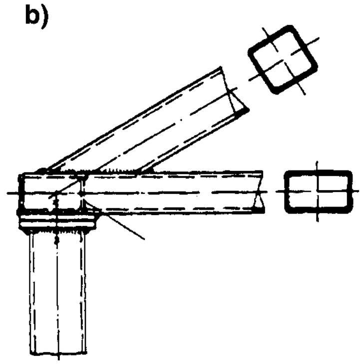
Fig. 4.55 – Load transmission through an inserted connection plate

a)



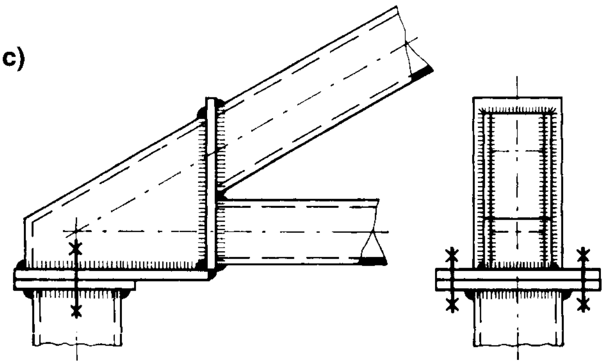
Continuous upper chord

b)



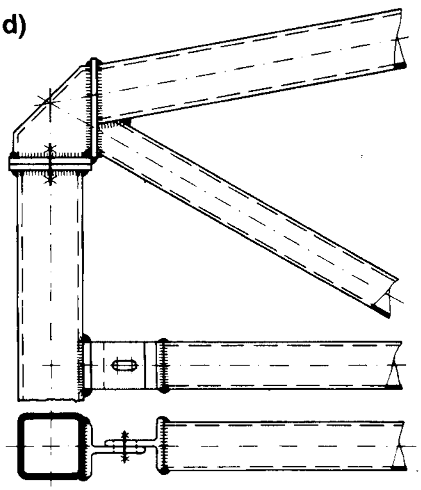
Continuous lower chord

c)



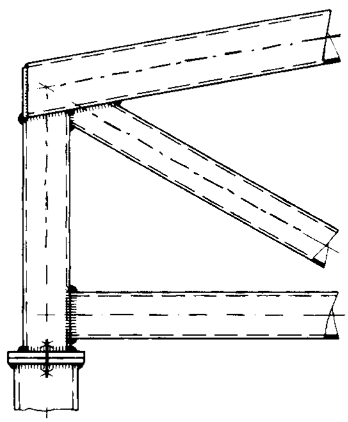
Preset shoe

d)



Console welded to upper chord and bolted tee connection with lower chord

e)



Welded truss bolted to column

Fig. 4.56 – Various types of column-to-truss connections

4.4.2 Column-to-truss connections

These constructions are mostly to be found at the supporting locations of trusses on hollow section columns. Fig. 4.56 illustrates various construction details, most of them configured for bolting. This is often to attain easy assembly and erection by avoiding welding on site. Usually, units (trusses) which have been welded in workshops, are bolted on site. The whole operation is obviously governed by transport limitations. Figures 4.56a) and b) show details, where the joint is made over a continuous upper or lower chord of a truss. In the latter case, it has to be considered whether an additional stiffening plate has to be welded in order to withstand high loads transmitted through the lower chord (see Fig. 4.56b)). It is also possible to weld a shoe to the converging upper and lower chords and use it as a bearing foot (see Fig. 4.56c)). This detail is however quite expensive. Upper and lower chords can also be welded or bolted at separate locations on columns using tee sections or flat plates (see Fig. 4.56 d) and e)).

4.4.3 Directly joined connections

4.4.3.1 End-to-end connections

For hollow sections, the end-to-end connections are made principally by butt welding. The method is simple, given by three cases as shown in Fig. 4.57:

Case 1 No weld preparation at the member ends (thin walled hollow sections)
Case 2 Member ends are bevelled for welding (thick walled hollow sections)
Case 3 End bevels are backed by an internal backing ring, which supports the liquid weld as well as helps the members to line up.

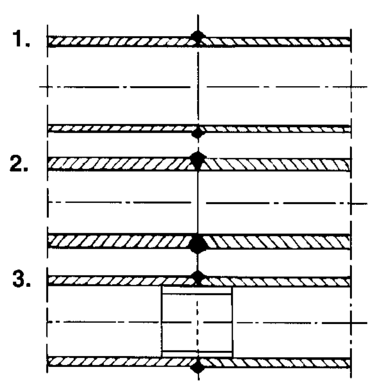
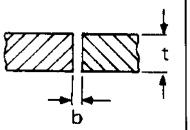
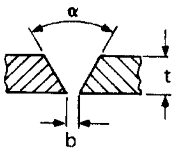
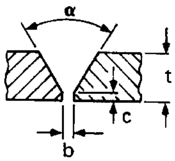
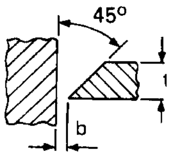


Fig. 4.57 – Welded end-to-end connections

The aim is to develop either the applied factored load or the full strength of the weaker member by obtaining adequate weld penetration. Appropriate electrodes have to be selected with relation to the steel used. Tables 4.1 and 4.2 illustrate the weld preparations for these kinds of joints both with or without a backing ring. Fig. 4.58 shows the end preparation of the members, when the wall thicknesses of the two members differ from each other.

Special preparations are required when the members cannot be welded in the horizontal position (see Fig. 4.59). Fig. 4.60 shows a detail of a backing ring for RHS joints other than shown in Table 4.2.

Table 4.1 – End-to-end connections without backing ring

Wall thickness mm	Application	Weld type	Weld preparation	α degr.	b	c
up to 3	one sided	I-type		–	t	–
3 to 20	one sided	V-type		≈ 60	0 to 3 mm	–
up to 20	one sided	Y-type		≈ 60	0 to 4 mm	1.5 to 4 mm
3 to 20	one sided	groove weld		45 to 60	0 to 3 mm	–
6 to 20	both sided					

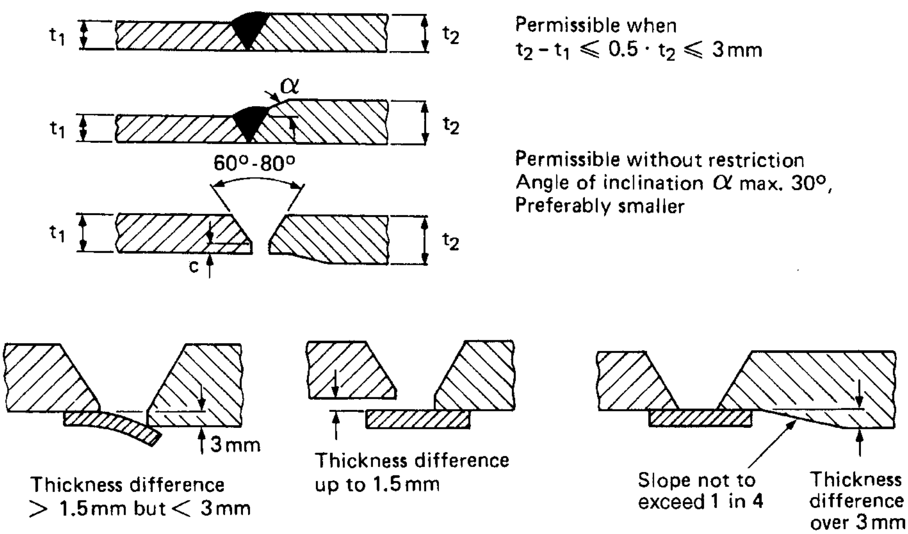


Fig. 4.58 – Weld preparations for end-to-end connections with members of different wall thicknesses

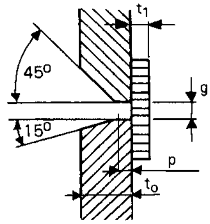
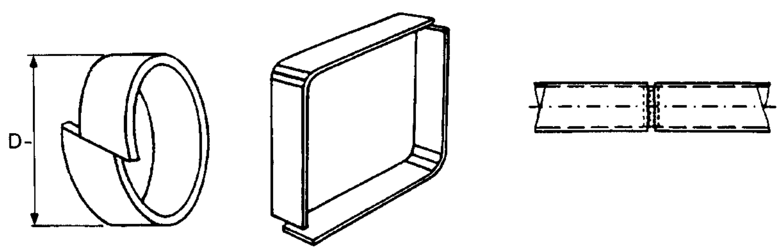
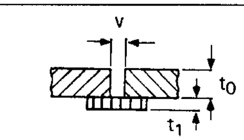
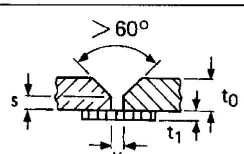
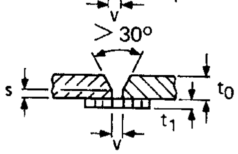


Fig. 4.59 – End-to-end connection welded in the vertical position

Table 4.2 – End-to-end connections with backing ring



Weld preparation for butt weld	Wall thickness t_0, mm	v		s		Backing plate thickness	
		min mm	max mm	min mm	max mm	min mm	max mm
	3	3	5	–	–	3	3
	5	5	6	–	–	3	5
	6	6	8	–	–	3	6
	< 20	5	8	1	2.5	3	6
	$20 \leq t_0 < 30$	8	10	2	3	3	10

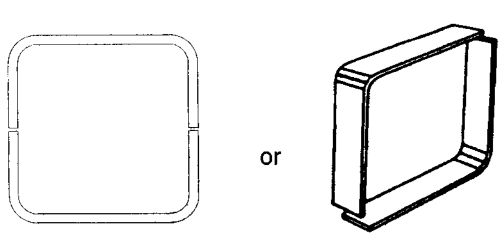


Fig. 4.60 – Detail of a backing ring for RHS end-to-end connection

For the installation of a backing plate into an RHS, special care should be taken regarding the fitting in the corner part. Bending of the corner part of the backing plate has to be executed very accurately and during installation, any mismatch has to be rectified by local heating and hammering. The gap between two backing plates has to be filled up by welding in order to prevent defect initiation during welding itself.

4.4.3.2 Welded knee joints

Fig. 4.61 shows the design of welded knee joints (90° inclination) with two fundamental types of welding details:

1. Simple knee joint (Fig. 4.61a)
2. Knee joint with a transverse stiffening plate (Fig. 4.61b)

Although Fig. 4.61 shows construction with square or rectangular hollow sections, welded knee joints can also be made of circular hollow sections. The design calculation for knee joints of square and rectangular hollow sections can be found in [3]. The same for CHS knee joints is to be found in [17].

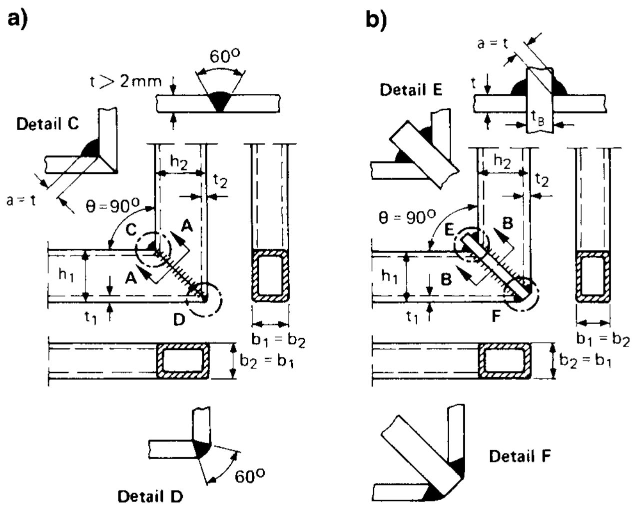


Fig. 4.61 – Knee joints   a) without stiffeners   b) with stiffening plate

A plain knee joint, consisting of identical leg and lintel cross sections, is simple and inexpensive, although it is meant for low loads. These joints tend to fail by excessive deformation of the transverse face by compression.

A welded knee joint with an intermediary stiffening plate is applied to adapt different member sizes or when additional joint strength is required. Excessive deformation occurs only for very thin hollow sections in this case. The thickness of the inserted plate must not be less than 1.5 times that of the thicker hollow section and, in any case, at least 10 mm.

An alternative design to increase the joint strength is to reinforce the joint by welding haunches (offcut of RHS of the same width as the two main members) (Fig. 4.62) or lateral plates (Fig. 4.63) to avoid local buckling.

While welding the haunch, the weld gap must not exceed 3 mm (Fig. 4.64a). A simple alternative is to make the width ratio $b_i/b_0 \approx 0.85$ (Fig. 4.64b).

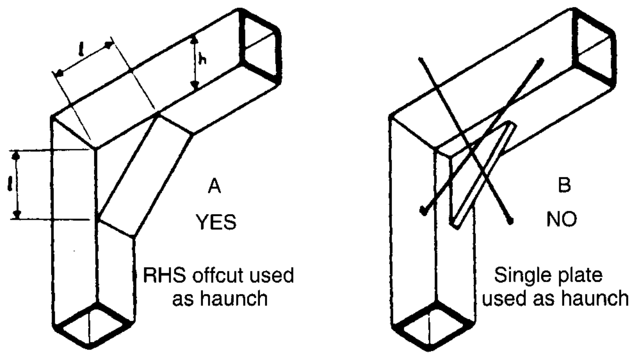


Fig. 4.62 – Reinforcement of knee joint with RHS offcut as haunch

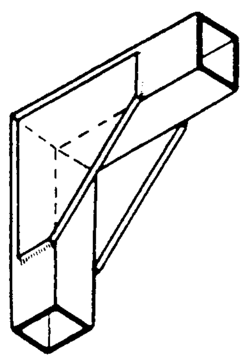


Fig. 4.63 – Reinforcement of knee joint with gusset plates

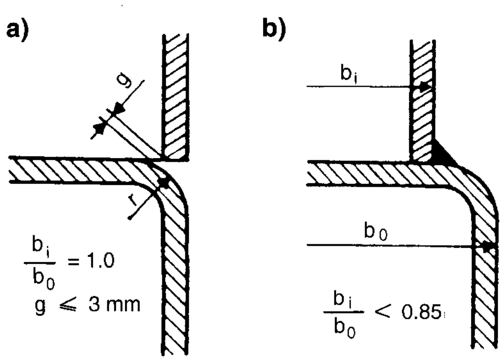


Fig. 4.64 – Welding a haunch

The angle of inclination between the axes of the hollow section members can exceed 90^o (obtuse angle $90^o < \theta < 180^o$). These joints can in any case be designed similar to 90^o-joints, since their behaviour is better than that of 90^o-joints (Fig. 4.65).

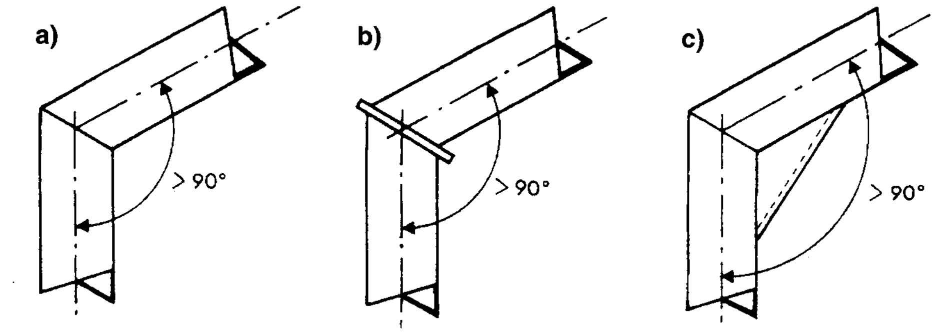


Fig. 4.65 – Knee joints with obtuse angles

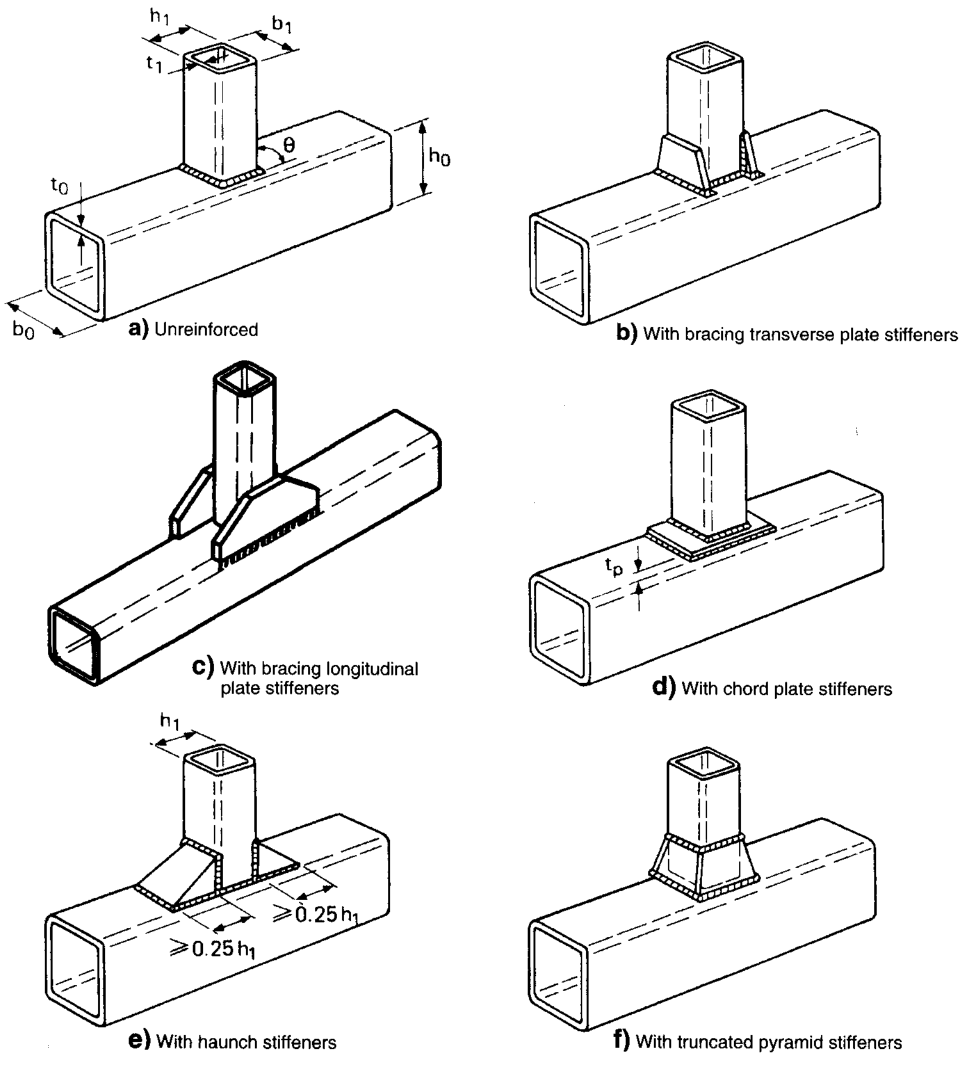


Fig. 4.66 – Vierendeel connection types

4.4.3.3 Vierendeel girder joints

Figures 4.66 a) to f) show some Vierendeel connections of RHS; they can be considered as T joints, where the vertical member is loaded mainly by bending moment as well as axial and shear forces. The calculation of strength of Vierendeel joints, unreinforced and reinforced, can be done according to [1, 3, 66, 71].
The unreinforced joint (Fig. 4.66a) is the simplest and the least expensive type; however, for $b_1/b_0 < 1.0$, this connection may be too flexible for a moment resisting one. The stiffness increases considerably, if $b_1/b_0 = 1{,}0$ is used. This leads however to difficulty in welding at the sides due to the corner radius of the chord, especially if the corner radius is large (see Fig. 4.64a).
In principle, the joint strength and flexural rigidity of an unstiffened joint decrease as the chord slenderness ratio b_0/t_0 increases and as the bracing to chord width ratio b_1/b_0 decreases. A $b_1/b_0 \approx 1.0$ and a low b_0/t_0 may lead up to almost full rigidity; other values result in semi-rigid connections.
Fig. 4.66a) is also applicable to CHS joints with similar restrictions as for RHS joints.
An evaluation of the reinforcements is done in the following:

- The reinforcement shown in Fig. 4.66b) results in only marginal increase of joint strength, since the connection of the plate at the chord corners is the stiffest location and most of the load is transmitted there. Other parts become effective after considerable yielding or even cracking. This type is therefore not recommended for application. However, the joint with longitudinal plate stiffeners as shown in Fig. 4.66c) demonstrates a better loading behaviour.
- The connection shown in Fig. 4.66d) is adequately efficient provided the chord flange stiffening plate thickness t_p is larger than the chord wall thickness and the stiffening plate length is sufficient. Considering the relatively moderate increase of fabrication cost, this type of reinforcement can be recommended. CHS joints can also be reinforced applying this method, although due to curvature this is not as good as for RHS joints.
- The connection with stiffening haunches shown in Fig. 4.66e) is stronger than that in Fig. 4.66d) and can provide the full moment capacity of the bracing. The haunches should have a dimension of $0.25\ h_1$ and can be made from an offcut of the bracing. It is recommended as an efficient and economical way of reinforcement.
- Fig. 4.66f) illustrates by far the most effective form of reinforcement, although it is the most costly from a fabrication point of view. This however may be applied in the rare cases, where the vertical has a much smaller width than the chord.

The reinforcements discussed above are often used when Vierendeel joints have to be repaired.

4.4.3.4 Truss joints with directly welded members

The possible combinations of circular, square and rectangular hollow sections as chords, diagonals and verticals in a truss are listed in Table 4.3.

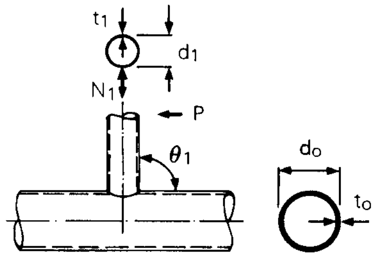
Table 4.3 – Combinations from cross section forms for bracing and chords

Bracing	Chord
□ ▭ □	□ ▭ □
○	□ ▭ □
○	○
○	I I
□ ▭ □	I I

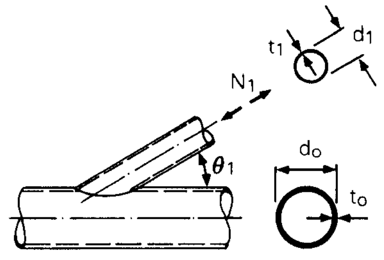
This also contains a further possibility consisting of I-sections as chords and hollow sections as bracings

Table 4.4 – Truss joints in CHS

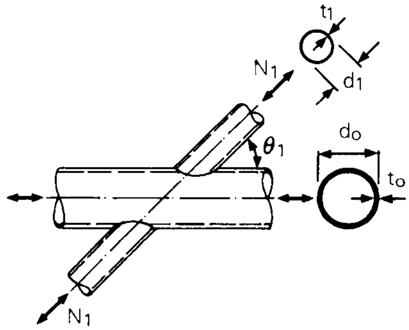
T



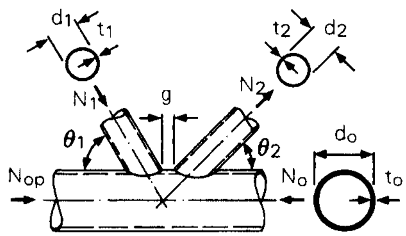
Y



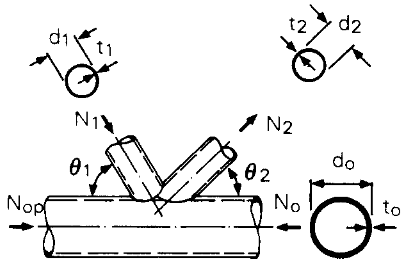
X



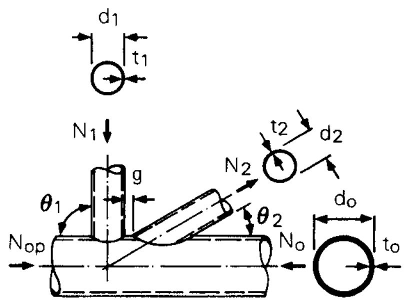
K with gap



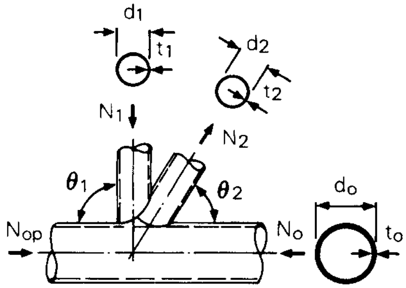
K with overlap



N with gap



N with overlap



KT with gap

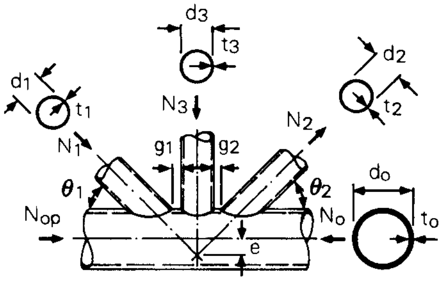
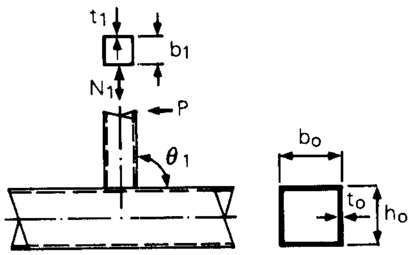
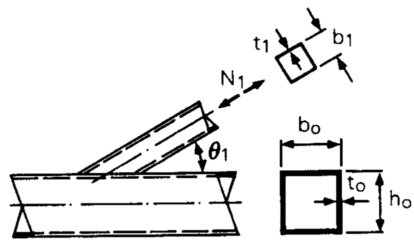


Table 4.5 – Truss joints in RHS

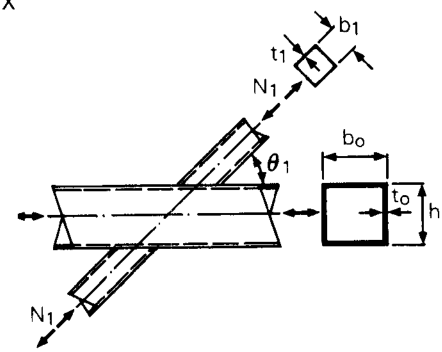
T



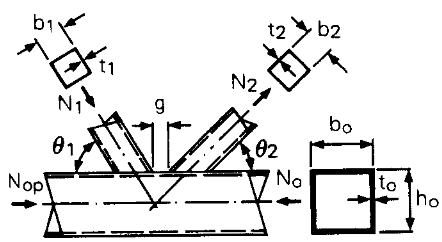
Y



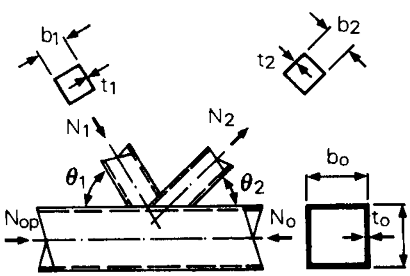
X



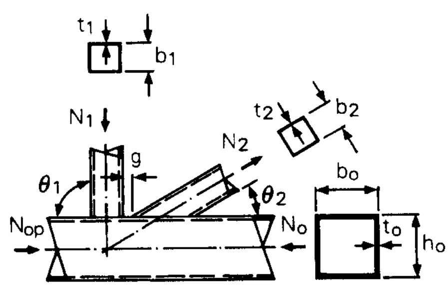
K with gap



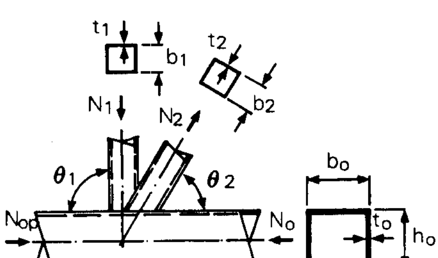
K with overlap



N with gap



N with overlap



KT with gap

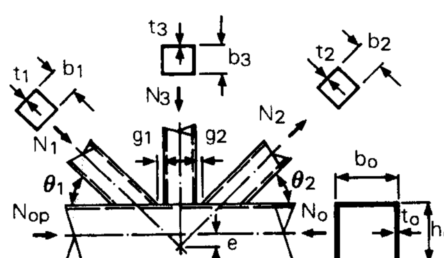
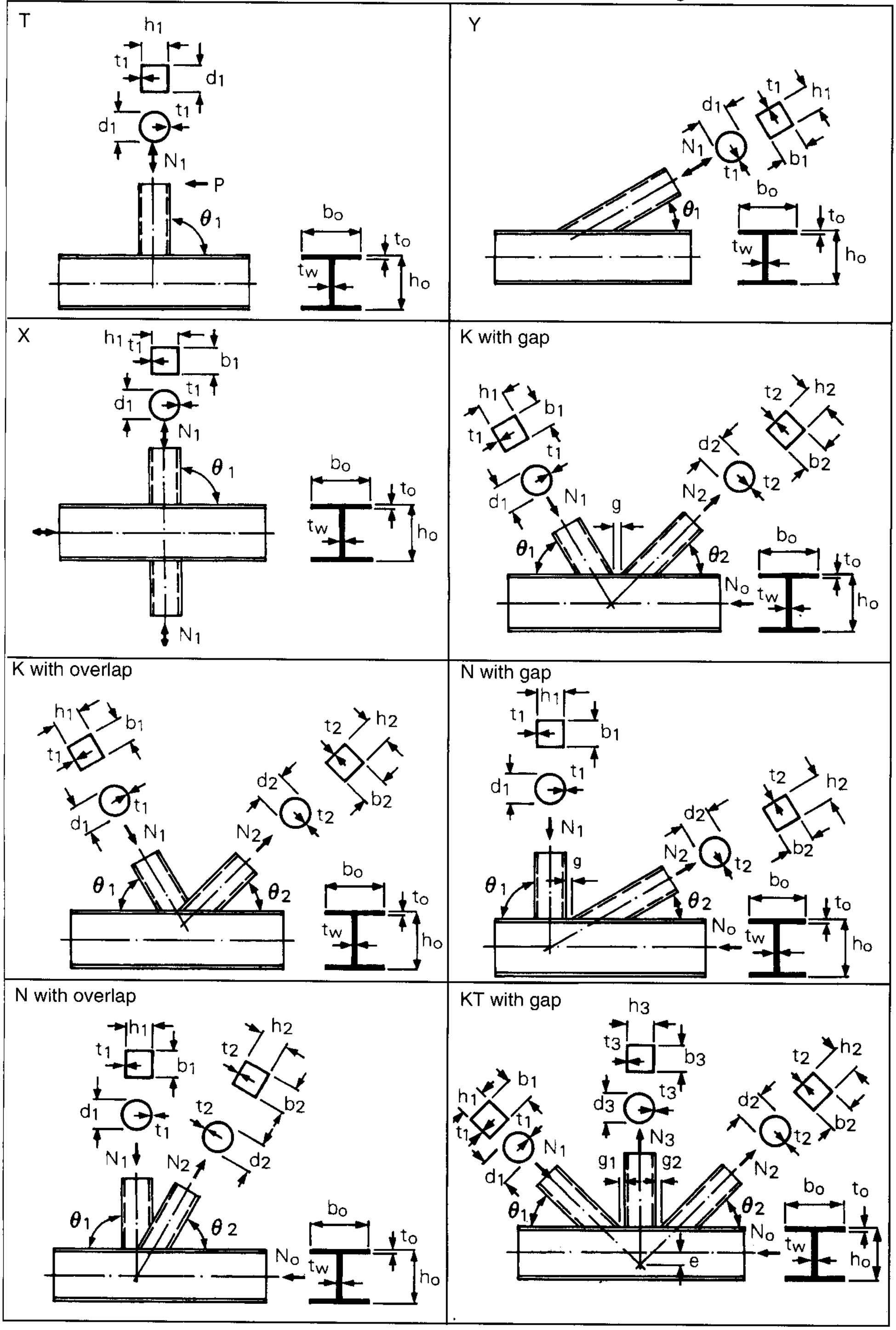


Table 4.6 – Truss joints with I-section as chord and hollow section bracings



Tables 4.4, 4.5 and 4.6 illustrate the main geometrical joint configurations as follows:

- T or Y joints
- X joints
- N or K joints
- KT joints

A further definition for N, K and KT joints is:

- Gap "g" between the toes of the bracing members (ignoring welds)
- Partial or full overlap of the bracing members

The strength calculation of these joints can be made using [1, 3, 9, 66, 71], which all give identical formulae.

Members of truss joints of hollow sections are directly welded to one another normally as joints using fillet and partial or full penetration butt welds; illustrated in various standards [51, 54, 58, 60, 67]. Selection of the weld type depends mainly on the angle of inclination θ of the bracing to the chord as well as the wall thickness of the bracing, which in general is smaller than the chord wall thickness.

Truss joints between directly welded hollow sections are made using fillet or a combination of fillet and butt (groove) welding.

Figures 4.67 and 4.68 show the basic conditions of applying fillet and butt (full or partial penetration groove) welds [67]. The details demonstrate the change of welding bevel opening angle from point to point around the intersection periphery (see also Fig. 3.9).

As the detail Z_2 in Figures 4.67 and 4.68 is expensive, welding according to Fig. 4.69 can be suggested.

For the crown toe (X point), saddle (Y point) and crown heel (Z point) in the above mentioned figures, the following conditions prevail:

Crown toe

$\theta \leq 60^o$, all thicknesses:	butt weld (Detail X_1)
$\theta > 60^o$, $t_1 < 8$ mm:	fillet weld (Detail X_2)
$t_1 \geq 8$ mm:	butt weld (Detail X_3)

Saddle

$t_1 < 8$ mm, d_1/d_0 or $b_1/b_0 \leq 0.85$:	fillet weld (Detail Y_1)
$t_1 < 8$ mm, d_1/d_0 or $b_1/b_0 = 1.0$:	butt weld (Detail Y_2)
$t_1 \geq 8$ mm:	butt weld (Detail Y_3)

Crown heel

$t_1 < 8$ mm:	fillet weld (Detail Z_1)
$t \geq 8$ mm:	butt weld (Detail Z_2)
	or fillet weld (see Fig. 4.69)

$\theta \geq 30^o$ is recommended to assure appropriate penetration of the weld in the heel region. For a combination of fillet and butt welds, the transition must be smooth and continuous.

The weld used most frequently is the fillet weld, which can be configured convex, plane or concave; the throat thickness "a" is described by an isosceles triangle (see Fig. 4.70).

IIW suggests [61] $a \geq 1.1\ t_1$ for S355 and $a \geq t_1$ for S235/275. Eurocode 3 [66] recommends a range of throat thicknesses for fillet welds in accordance with the steel grades (Table 4.7).

It has to be mentioned here that a weld of the concave type (Fig. 4.70c) produces a better fatigue behaviour on account of the more gradual transition of the weld to base metal.

In the case of a RHS joint with $b_1 = b_0$ (Fig. 4.71), it is the corner radius which determines the cross section of the weld. If it is possible, the gap "g" has to be made ≤ 3 mm by reducing b_1. In unavoidable cases, the weld can be built up, although this may be expensive.

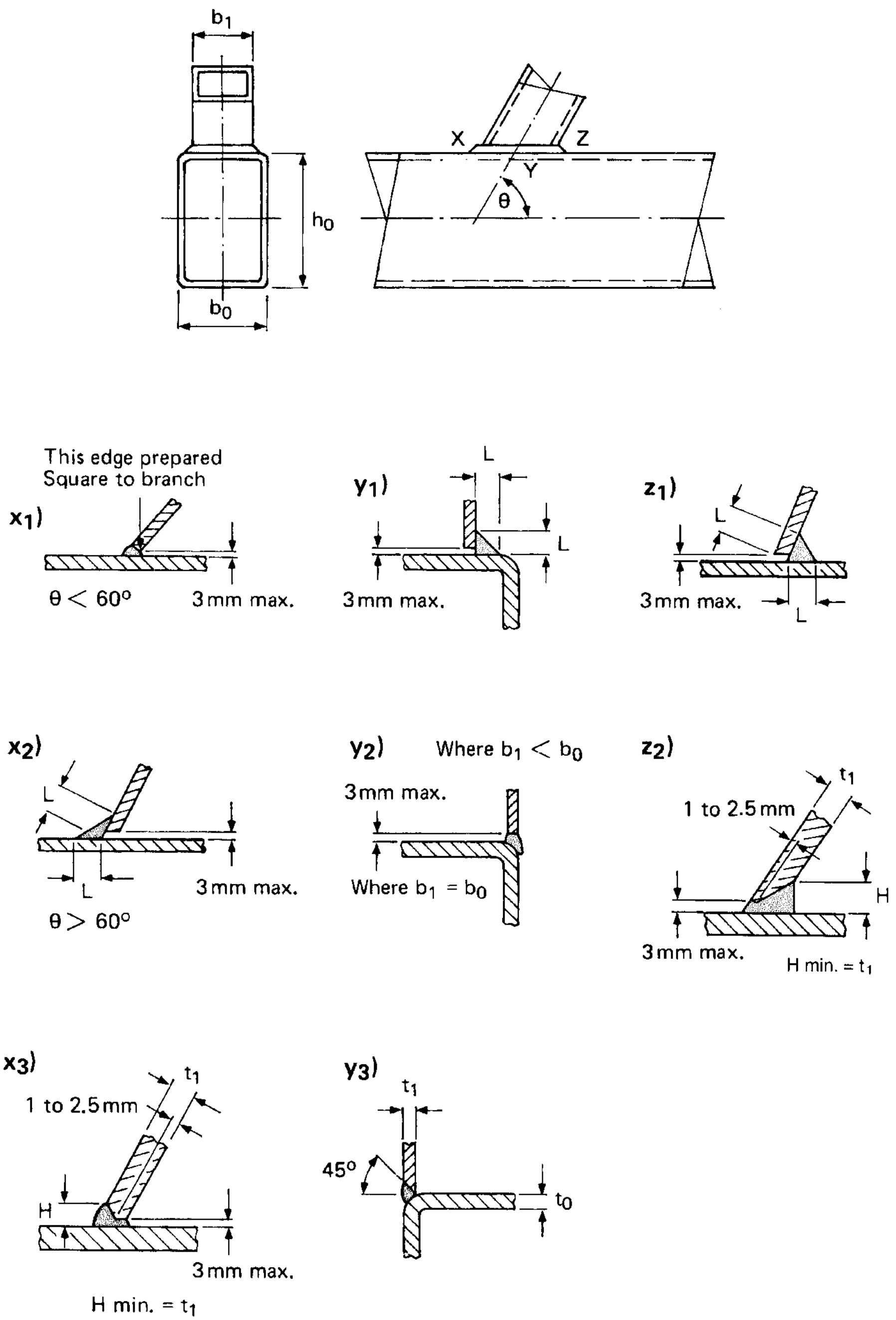


Fig. 4.67 – Fillet and butt welds in lattice joints between RHS members

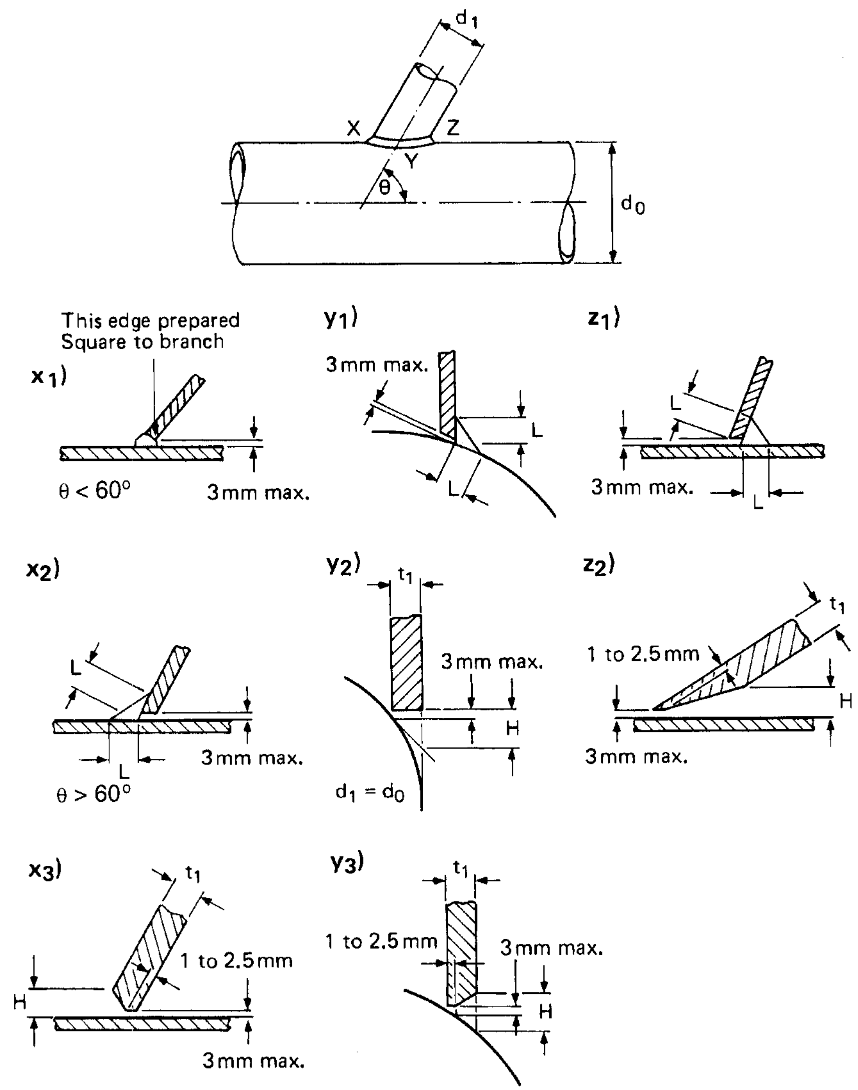


Fig. 4.68 – Fillet and butt welds in lattice joints between CHS members

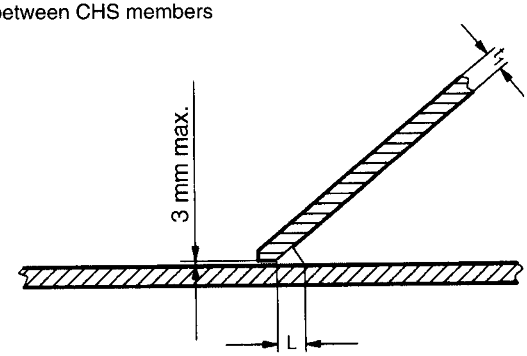


Fig. 4.69
Alternative to detail Z_2
in Figures 4.67 and 4.68

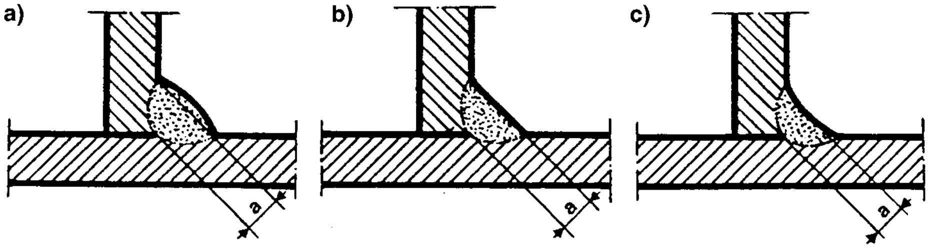


Fig. 4.70 – Fillet weld shapes a) convex b) plane c) concave defining throat thickness "a"

Table 4.7 – Fillet weld throat thicknesses according to [66]

Steel grade according to Table 2.2a (non-alloy steel)	
S235 S275 S355	$a/t_1 \geq 0.84\ \alpha^*$ $a/t_1 \geq 0.87\ \alpha^*$ $a/t_1 \geq 1.01\ \alpha^*$
Steel grade according to Table 2.2b (fine grain steel)	
S275 S355	$a/t_1 \geq 0.91\ \alpha^*$ $a/t_1 \geq 1.05\ \alpha^*$

* $\alpha = \dfrac{1.1}{\gamma_{Mj}} \times \dfrac{\gamma_{MW}}{1.25}$ where γ_{Mj} = Partial safety factor for joints in lattice structures (= 1.1 according to EC3 [66])

γ_{MW} = Partial safety factor for weld (= 1.25 according to EC3 [66])

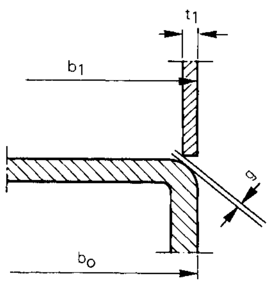


Fig. 4.71 – RHS joint layout with $b_1 = b_0$

From the viewpoint of the economy of fabrication, gap connections (K, N, KT situations) are preferred to partial overlap joints, because the members are easier to prepare, fit and weld (Fig. 4.72).
However, fully overlapped joints may provide a better joint strength with similar fabrication than gap joints, although with less tolerance for fitting.
The fabrication steps of partial overlap joints are worth special mentioning as they require more preparation in the workshop and it is necessary to choose the proper construction to obtain the lowest cost. Fig. 4.73 illustrates three solutions for K type joints with partial overlap in CHS.

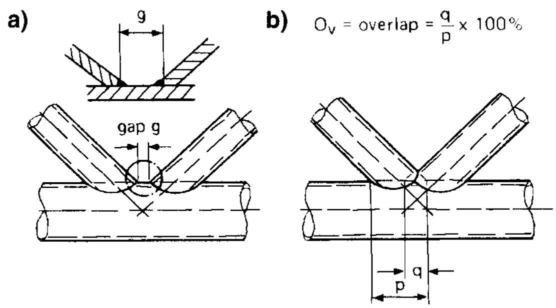


Fig. 4.72 – K joints with a) gap b) partial overlap

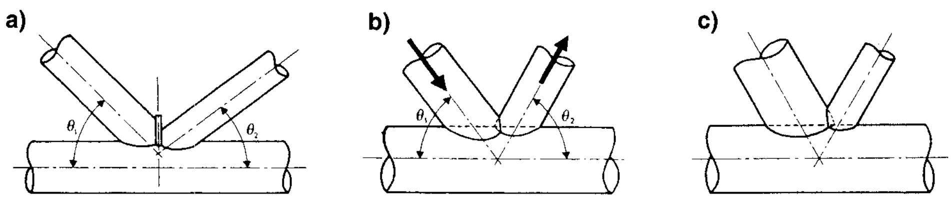


Fig. 4.73 – Various partial overlap joints of K type in CHS

a) Two bracing members are connected by welding through an intermediate plate (square cut for bracing at the intersection)
b) In case the diameters and wall thicknesses of the two bracing members are of the same order, the tensile member shall be welded to the chord first and the compression member shall partially cover the tensile one by welding (profile cut the bracing member in compression twice).
c) In case the difference between the bracing diameters is large, the larger one shall be first welded to the chord and the smaller one shall overlap the larger subsequently (double profile cut the smaller member).

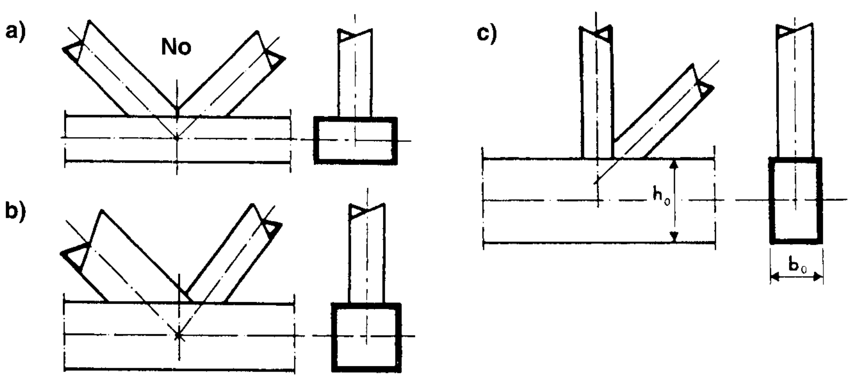
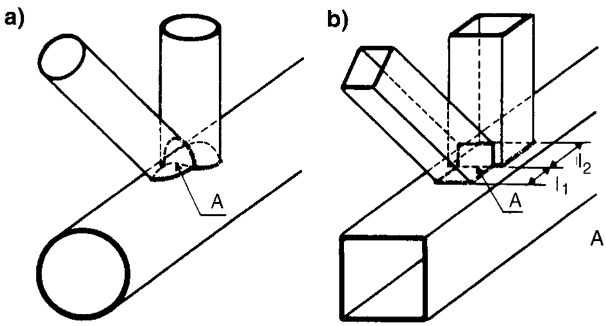


Fig. 4.74 – Various partial overlap joints of K and N type in RHS

a) Both bracing members have double cuts at the ends. This connection is **not acceptable** because the joint will have a strength lower then predicted.
b) The larger member shall be first welded to the chord. The smaller one shall cover the larger one (double cut the smaller member).
c) For N type, the vertical is first welded to the chord. The diagonal shall overlap the vertical with a double cut.
   This is however not always possible. In a $90^{0}/45^{0}$ joint the load in the diagonal member is about 40% higher than that in the vertical, which will probably mean that the diagonal member is bigger and/or thicker than the vertical one. In this case, the diagonal has to be welded first to the chord.

The various solutions for RHS joints (K and N type) are shown in Fig. 4.74.
A special comment is called for a joint with partial overlap (Fig. 4.75). In fabrication workshops, it is usual to mount the members of a lattice girder on an assembly jig and tack weld them. Final welding follows in a separate operation. This sequence makes it impossible to weld the seam in the covered part "A". However, experiments have shown that the strength of the joint is not generally affected by leaving the weld at "A" out.
However, if the vertical components of load in the two bracings are different by more than 20%, then the weld has to be made.
Fabrication steps and welding sequence are to be carried out in a manner so that the weld-initiated residual stress and deformation shall be kept as low as possible. In uniplanar and multiplanar hollow section joints, non-uniform heating of the chord member can lead to undesirable deformations and residual stresses. Primarily, the residual stresses can be reduced by selecting proper welding sequence, which allows free shrinkage. In lattice girders, for example, welding of the bracings should start from the centre and work outwards towards the end bracings; this results in compression residual stresses being initiated rather than undesirable tensile residual stresses.



A – Welding of the hidden toe may be dispensed with if the vertical load components are not more than 20% different

Fig. 4.75 – Welding of a joint with partial overlap

Fig. 4.76 indentifies the welding plan as recommended in [67] based on the following guidelines:

- Stop/start positions should not be located at or close to the toe position or lateral flank positions of a saddle weld joint between two circular hollow sections.
- Stop/start positions should not be located at or close to the corner positions of a joint between a square or rectangular hollow section bracing and a chord of a square or rectangular hollow section.

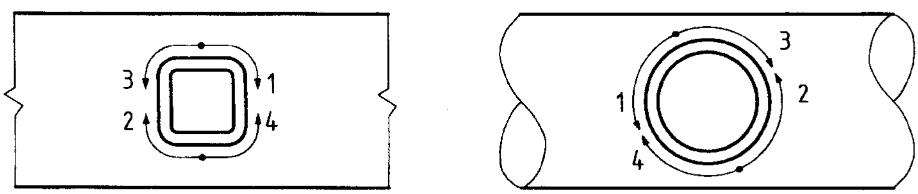


Fig. 4.76 – Recommended welding sequence for rectangular, square and circular hollow section joints

- Welding between hollow sections should be completed all round as a seal weld, even if this total length of weld is not required for strength reasons [3, 9].
- Stop/start positions of single pass welds should be chosen to avoid these positions coming under the location of a subsequent weld.
  However, intermediate stop/start positions can be necessary, where joint geometry, i. e. in overlap and reinforced joints, is such that welding cannot be executed continuously.

Truss joints, where CHS bracings with flattened ends are welded to CHS or RHS chords, are usually used to avoid complex profile cuts (CHS chords) or for easy fabrication. In general, CHS bracings with cropped (see Fig. 3.21A) or partially flattened (see Fig. 3.21D) ends are used; there are however also cases for full flattening (see Fig. 3.21B). Fig. 4.77 shows the following recommended joint configurations:

1. Joint with CHS bracings welded to CHS chord (Fig. 4.77a). Flattened faces of the bracings running parallel to the chord axis, overlap of the flattened ends is recommended.
2. Joint with CHS bracings welded to RHS chord (Fig. 4.77b): Flattened faces of the bracings perpendicular to the chord axis, prevents chord face deformation if the width of the flattened ends approximates that of the chord width.

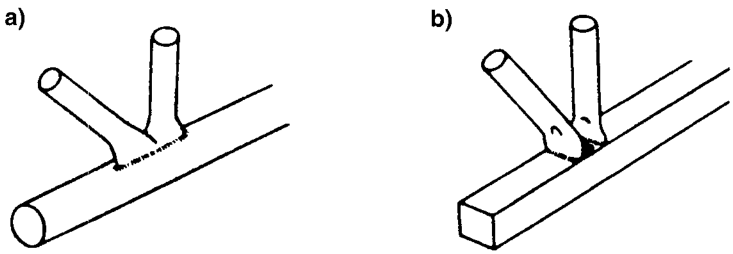


Fig. 4.77 – Joints with CHS flattened end bracings a) CHS chord b) RHS chord

The methods for the design calculations for joints with CHS flattened end bracings are given in [1, 8, 9].
Fig. 4.78 illustrates an interesting layout of a triangulated girder with a CHS upper chord and two RHS lower chords. This layout is sometimes applied, when the girders lie horizontally. The flattened ends of the CHS bracing members are then welded to the corners of the RHS chords.

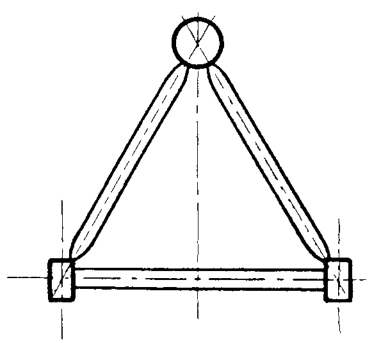


Fig. 4.78 – Triangulated girder (two RHS bottom and one CHS upper chord)

If the joint strength of welded truss joint is too low, a possibility of strengthening them is by using reinforcements, e.g. a stiffening plate welded to the chord flange or web, a vertical stiffener or a combination of vertical and horizontal stiffeners as shown in Fig. 4.79.
Reinforcements of Vierendeel joints have been described in Chapter 4.4.3.3.
Design calculation for reinforced hollow section joints have been discussed and recommended in [3, 9, 71]. They show the effect of the length, width and thickness of the reinforcement plates on the joint strength.
Designers should not normally use reinforced joints, because they are unnecessarily expensive involving high labour costs and may possibly disturb the aesthetic appeal of a structure. They are, in most cases, resorted to for a belated strengthening, if the dimensions

of the members of a joint prove to be inadequate. However, for hollow section joints under fatigue load, e.g. in cranes, the flange plate reinforcement comes often into application, as this may increase the life of a joint significantly [10].

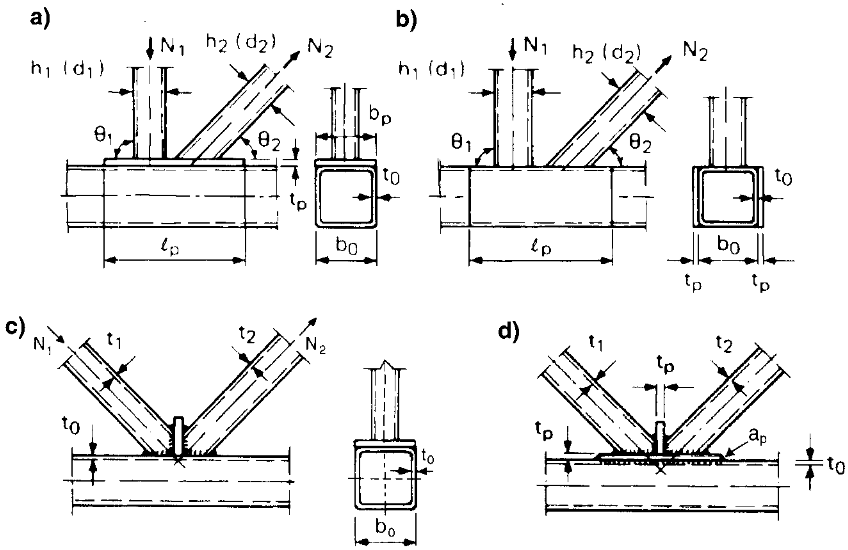


Fig. 4.79 – Reinforced truss joints a) Flange plate reinforcement b) Web plate reinforcement c) Vertical plate reinforcement d) Combination of flange and vertical plate reinforcement

All-round welding is required to connect the flange reinforcement plate to the chord. Care has to be taken that the weld seals the two inner surfaces to prevent corrosion.

4.4.4 Indirectly joined connections

The procedure for joining hollow sections by the indirectly joined connections consists predominantly of a combination of welding and bolting.

4.4.4.1 End-to-end connections

Fig. 4.80 a to g shows a variety of simple, bolted end-to-end connections, where the intermediaries, e.g. plates, tees, angles and channels are welded to hollow sections.
If used as connections for diagonals in lattice structures, they can also fulfil the function of closing the ends of a hollow section to avoid internal corrosion (see Fig. 4.80 a to d). It is important to note that the flange of the tee section (which can also be made by welding two flat plates) must be sufficiently thick to distribute the load effectively to the cross section of the hollow section [16] (see Fig. 4.80c).
Another solution, shown in Fig. 4.80e, consists of inserting a flat plate through a slot in the hollow section and welding it.
For larger loads, two plates can be welded to the two sides of the RHS (see Fig. 4.80f) instead of inserting a plate into it.
Fig. 4.80g shows a detail, where a more uniform load distribution on all four side walls of the RHS can be achieved by cutting the hollow section ends at a 60° angle and welding them to a "Y" piece made of two plates.

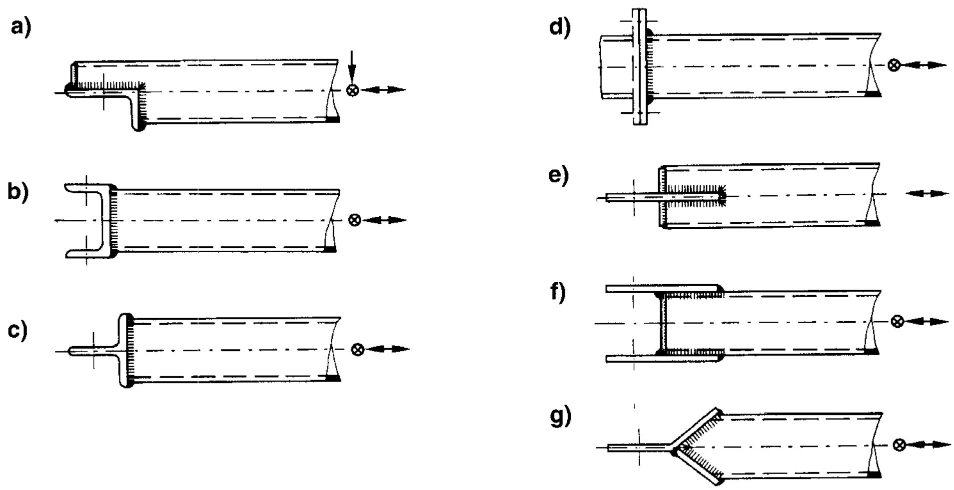


Fig. 4.80 – End-to-end bolted connections
a) Angle welded to hollow section end
b) Channel welded to hollow section end
c) Tee section welded to hollow section end
d) Flange plate welded to hollow section end
e) Flat plate inserted into hollow section slit and welded
f) Flat plates welded to two sides of RHS
g) "Y" piece made of flat plates welded to RHS

A flange plate connection is illustrated in Fig. 4.80d with further details in Figs. 4.81 and 4.82. Flange plates can be made of a ring (x) or solid (y) plate. Various shapes (z) can be adopted for both circular and rectangular hollow sections.
Ring flanges require to be very thick in order to retain their strength, while solid flange plates can be much thinner. Fillet or groove welds are used depending on the thickness of a hollow section.

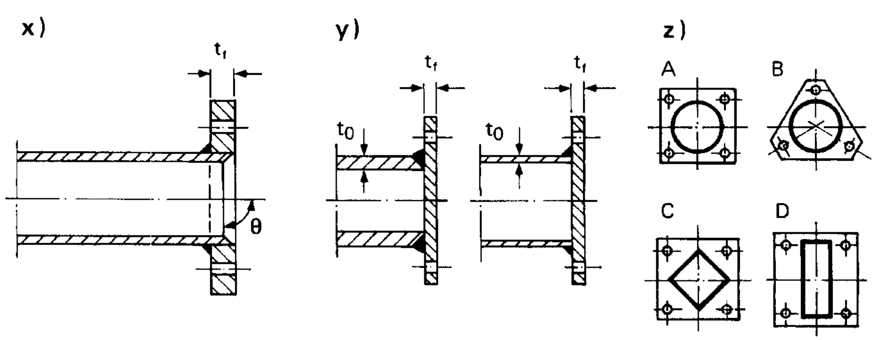


Fig. 4.81 – Flange plate connections

The reduction of the flange plate thickness by using stiffeners, as shown in Fig. 4.82, is not recommended due to the adverse effect of local bending in the hollow section walls at the top of the stiffeners as well as considering the additional costs for fabrication, protection and maintenance. A thick end plate instead of using stiffeners can provide sufficient resistance; however, care should be taken regarding the material quality. For very thick plates, a low sulphur content is essential to avoid lamellar tearing of the plate. Also, in many countries plate steel is generally of a lower strength than that of hollow sections, which may result in a much thicker plate than expected.

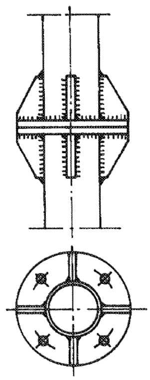


Fig. 4.82 – Flange plate connection with stiffening ribs

As an alternative to flange joints, in-line joints with bolted splice plates can be used. They can be either exposed (Fig. 4.83) or covered with cover plates (Fig. 4.84) to give a smooth external appearance.

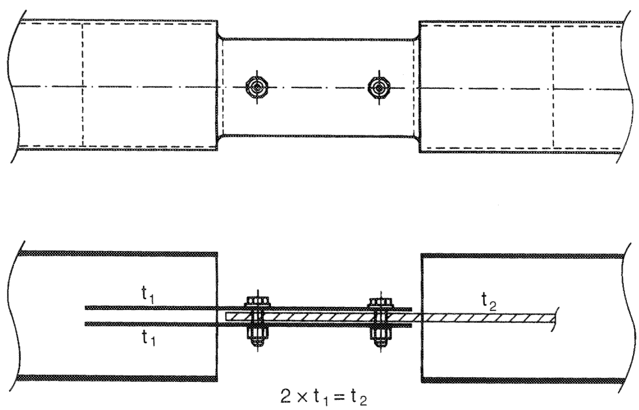


Fig. 4.83 – Splice joint (exposed)

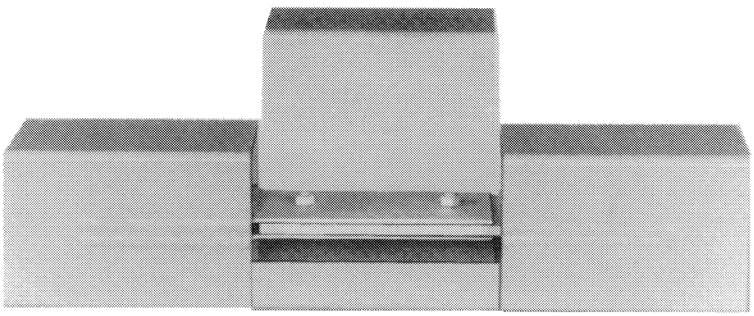


Fig. 4.84 – Splice joint (with cover plate)

Splice joints can also made using internal plates as shown in Fig. 4.85.

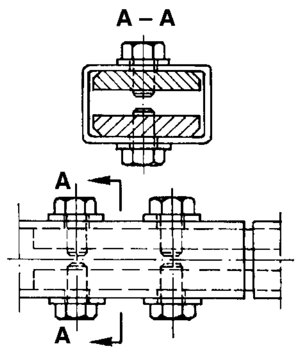


Fig. 4.85 – Splice joint with internal plates

Figures 4.86 and 4.87 show fish and splice plate connections of CHS and RHS respectively.

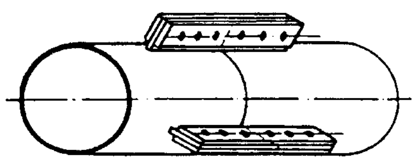


Fig. 4.86 – CHS splice plate connection

The former one consists of four, six or eight strips welded longitudinally on the periphery, which are then connected by sets of double fish plates, one on each side. These connections are suitable for larger parts and heavy loads. Care has to be taken to prevent corrosion.

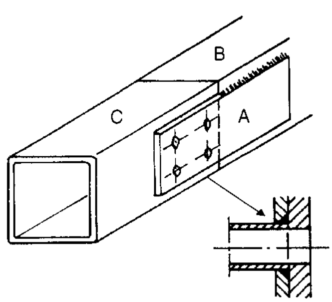


Fig. 4.87 – RHS splice plate connection

In the latter case, lateral strips (A) are welded to one of the sections (B) in the workshop and then bolted to the other section (C) on site. In the case of the danger of deformation of the hollow section wall by the tightening of through bolts, tubular spacers should be inserted and welded as shown in Fig. 4.87. The connection will be expensive if this operation is necessary. Measures to prevent corrosion have to be taken for this type of connection too.

Hollow sections with welded head plates, which do not protrude outside in order to keep a smooth appearance, have to be bolted within the hollow sections as shown in Fig. 4.88. The accessibility to the bolts is obtained by means of hand access holes cut in the members.

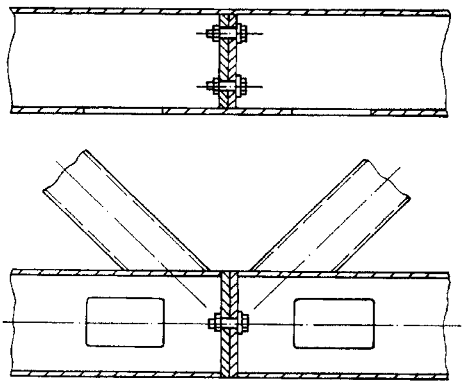


Fig. 4.88 – Hidden, bolted butt joint with head plates in the compression chord of a lattice girder

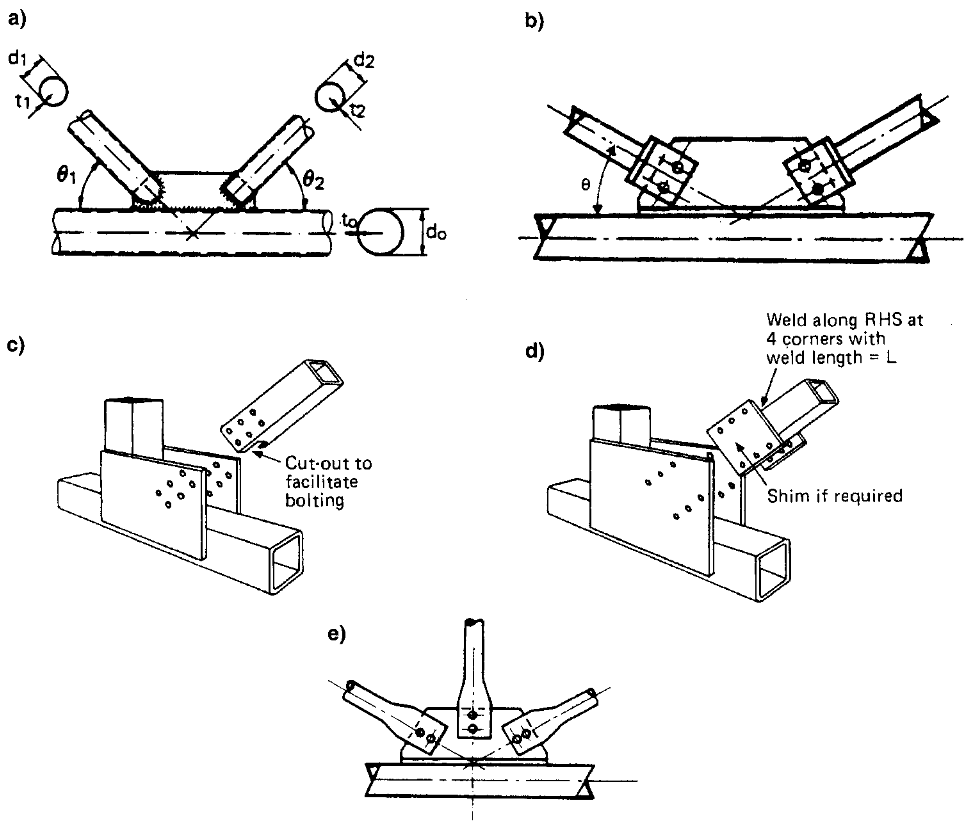


Fig. 4.89 – Various indirectly joined truss connections

a) Chord and bracings welded to gusset plate

b) Chord welded and bracings bolted to gusset plate

c) Chord and vertical welded and diagonal bolted to side plates (simple shear splice)

d) Chord and vertical welded and diagonal bolted to side plates via connection plates welded to it (modified shear splice)

e) Gusset plate welded to chord and the flattened ends of bracings bolted to the gusset plate

4.4.4.2 Indirectly joined truss connections

Fig. 4.89 illustrates a number of joints in a truss where the chord and the bracing members are connected via gusset or side plates to one another.

Fig. 4.89 a shows a solely welded design with a gusset plate welded to a CHS chord member. The bracings are then welded to the gusset plate to complete the connection. In Fig. 4.89b, the ends of the bracings are welded to the tees, which are bolted to the gusset plate connected to the chord.

It is usual that the fully welded version is fabricated and assembled in the workshop, while for the bolted version the assembly of the truss can be done on site.

Figs. 4.89c and d are alternatives using side plates welded to the chord and the sides of one bracing member, with the other bracing then bolted in place.

Diagonals are bolted to the side plates producing bolted shear connections. An important limitation to the use of these connections is the need to have closely matching RHS widths. Equal width members are connected directly as in Fig. 4.89c, but the side plates often need to be spread slightly by jacking, after welding is complete, in order to allow field assembly (welding contraction tends to pull the side plates inwards). Small width differences (tolerances) can be adjusted by the use of filler plates welded on the sides of the bracing member (Fig. 4.89d). Larger differences allow the further option of extra plates, which can be more convenient in the field.

Fig. 4.89e is another gusset plate version, where CHS bracings with flattened ends are bolted to a gusset plate, which is welded to the chord. The bolting can be done following normal bolting practice. If the structure is outside or exposed to corrosive atmosphere, the flattened ends have to be seal welded to prevent ingress of water leading to internal corrosion.

4.4.4.3 Purlin connections

Figures 4.92 through 4.94 show a number of connection details for purlins consisting of I-beam, channel or hollow sections (see Fig. 4.90) as well as of lattice girders (see Fig. 4.91) for large spans.

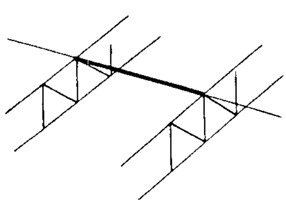


Fig. 4.90 – Purlin of single section

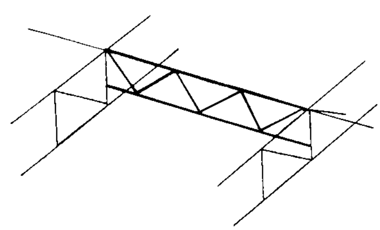


Fig. 4.91 – Purlin consisting of lattice girder

Figures 4.92 and 4.93 are the details for two simple attachments – the first one for a purlin of a rolled I-beam fixed by a folded corner plate and the second one for a purlin of a cold rolled section fixed by a bracket (an angle), which can have four holes and thus constitute a fish plate.

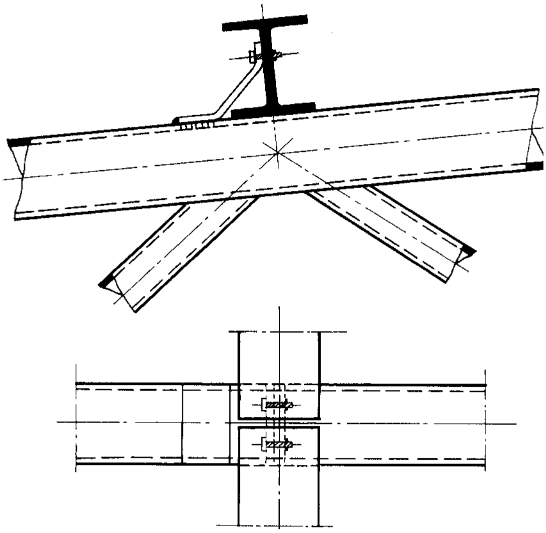


Fig. 4.92 – Purlin (rolled I-beam) attachment with a folded corner plate

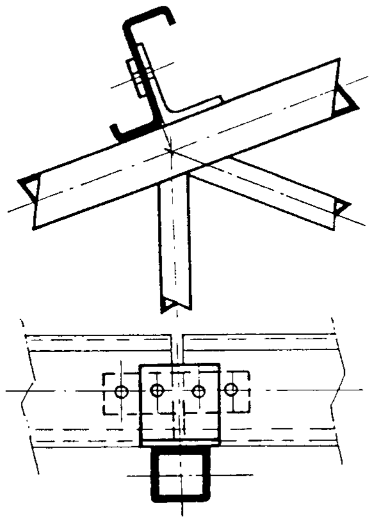


Fig. 4.93 – Purlin (cold rolled section) attachment with a bracket angle

A possibility of purlin construction with RHS is illustrated in Fig. 4.94. The application of RHS as purlins has the advantage that they show a more favourable behaviour against lateral buckling. As a consequence, intermediate ties can often be omitted.

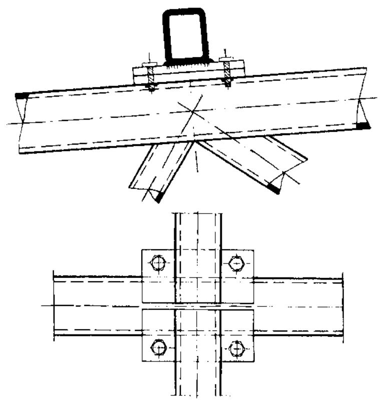


Fig. 4.94 – Purlin (RHS) attached with flange plates

Structural arrangements of lattice purlins are shown in Fig. 4.95. These purlins may be made continuous or the ties of the truss may be braced.

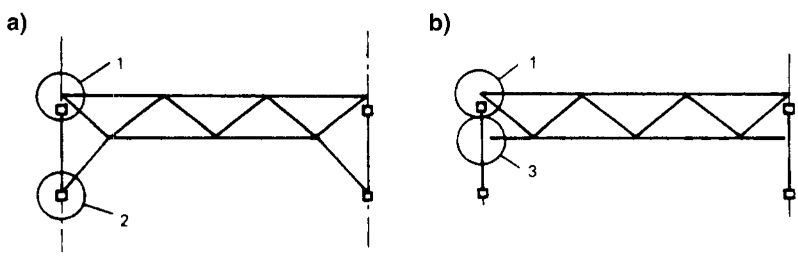


Fig. 4.95 – Structural arrangement of lattice purlins

Alternatives of the detail 1 are given in Fig. 4.96 and 4.97.

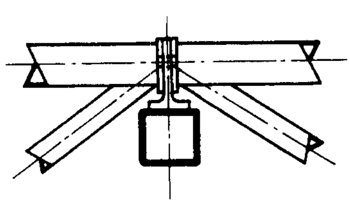


Fig. 4.96 – Detail 1 (length adjustment restricted to the bolt hole tolerances

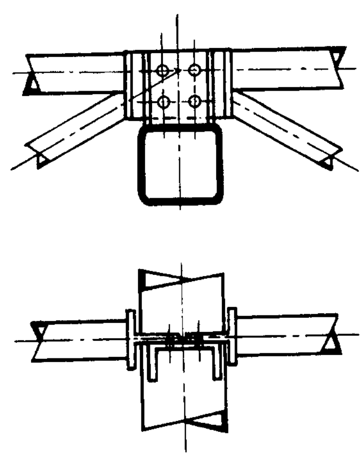


Fig. 4.97 – Detail 1 (length adjustment can be made by means of spacers)

Figures 4.98 and 4.99 describe the alternatives depending on the inclination θ of bracing members of detail 2.

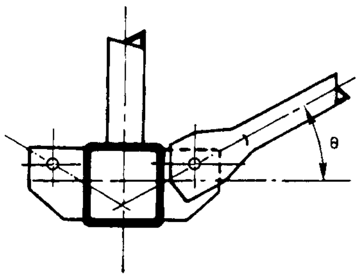


Fig. 4.98 – Detail 2 (centre lines offset)

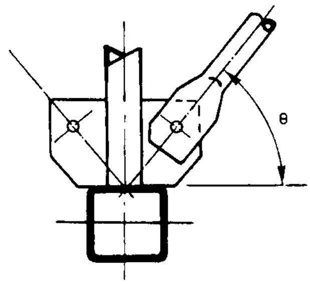


Fig. 4.99 – Detail 2 (without offset of centre lines by appropriate layout of main members)

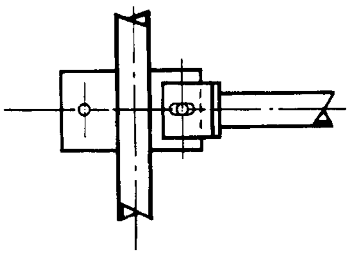


Fig. 4.100 – Detail 3

5 General procedure for the fabrication and assembly of hollow section structures

5.1 Fabrication and full scale drawings and their approval

The fabrication procedure starts with the preparation of the fabrication drawings based on the design drawing drawn by the designer and draughtsman. This describes each element of the structure as well as connections in detail taking also the equipment and technology available in the workshop into account.

It is often advisable to draw full-scale drawings of at least the small scale parts, even if full-scale drawings of the whole body are not made. This is recommended due to the facility, this offers, to carry out precise detailing of the structure and for checking the configurations and fittings of joints.

Construction components, selected steel grades for the structural elements and their dimensions as well as the fabrication procedures they have to undergo, should be recorded carefully in order to avoid any discrepancy between the fabrication and the design drawing. Fabrication labels giving the dimensions of hollow sections with wall thicknesses are important, as checking is difficult when the structure is in its final fabricated stage.

Before starting the real fabrication operation, it is necessary that the above mentioned points are checked by an inspector in charge, who can mediate in any controversy between designer and fabricator. Further, fabrication guidelines related to fabricating and welding methods, depending on the available equipment in the workshop, control facilities and skill and experience of the people involved with the fabrication, are to be approved by the qualified persons in charge.

5.2 Qualification of workshops and welders

As has been already described in Chapter 3.6.8, the following three points summarize the conditions for the eligibility of a workhop to fabricate hollow section structures and of a welder to perform welding in them:

1. Fabrication workshop must be properly equipped to fabricate a hollow section structure. Equipment may have to be adapted to the required design.
2. Persons involved with the fabrication must have adequate knowledge about the fabrication of hollow section structures and skill and experience to execute the required work.
3. Welders should be approved as qualified welders for hollow section structures by appropriate examinations.

5.3 Assembly

Proper choice of the rationalised assembling procedure is the key to the technical and economic production of a hollow section structure. This is significantly affected by the availability of jigs or assembling frames in the workshop or on site, which facilitate welding and bolting operations. Two jigs for lattice subassemblies of hollow sections are shown in Figs. 5.1 and 5.2, where the operations such as tack welding and then the final welding and also sometimes bolting are done. In order to offer a favourable welding position to the welder, the jigs allow the lattice subassemblies to be rotated.

Fig. 5.1 – Jig for a Warren type girder set on floor

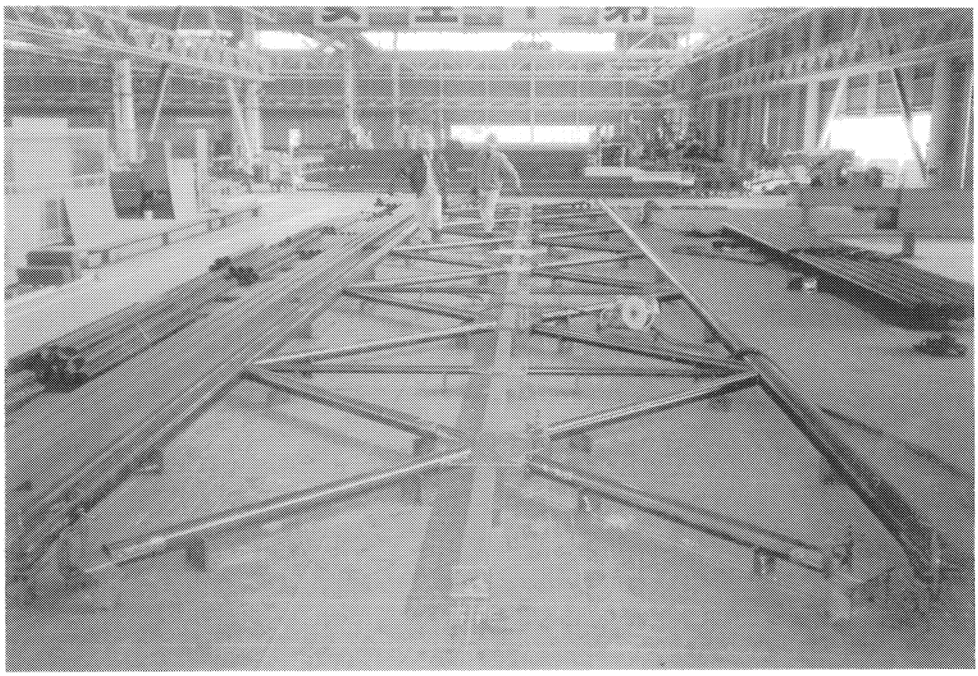


Fig. 5.2 – Jig for a Howe type girder set on floor

Fig. 5.3 demonstrates an interesting device for the assembly of a welded conveyor gantry truss with square hollow sections. The cross-bracing at the end of the truss is part of the fabrication jig and allows the truss to be rotated for easy handling.

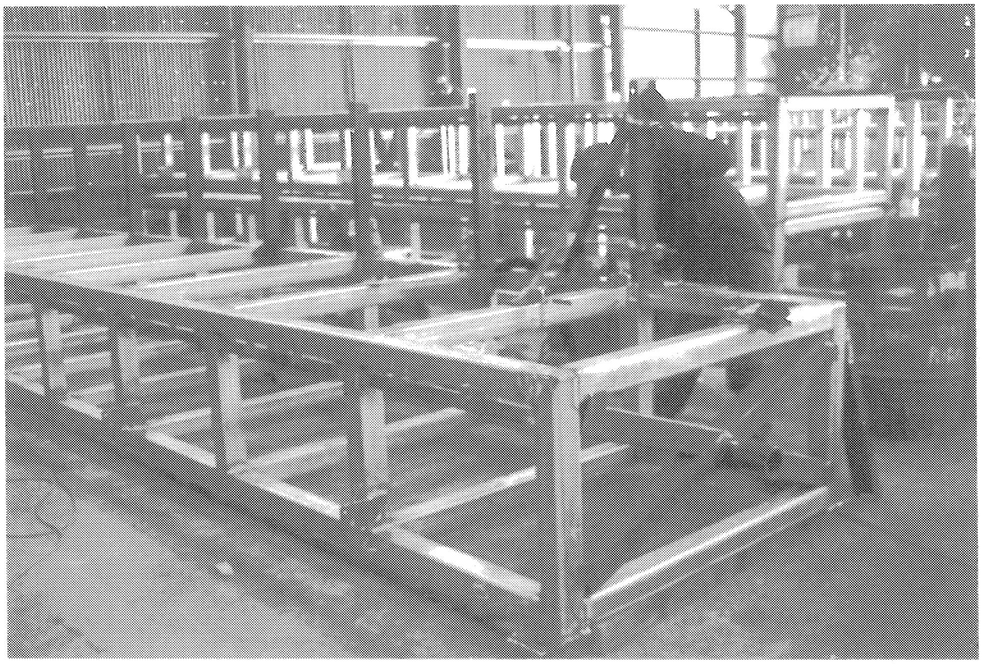


Fig. 5.3 – Welded conveyor gantry truss

A jig has to be manufactured taking account of weld shrinkage and considering distortion strain to ensure accuracy of product size and shape.
A jig can also be firmly set on a flat table, which usually consists of a frame made with shaped steel laid horizontally and is fixed with concrete and covered with a steel plate. Assembly is made by mounting material to be fixed on the table.
For hollow section trusses, it is essential that the order and sequence of assembly are planned before hand, especially for a joint intersection with more than two bracing members, where great care has to be taken regarding the assembly procedure.
Various assembly devices e.g. assembly frame with cradles, marking off slab and rotation frame, can be applied. Fig. 5.4 illustrates an assembly frame comprising of a main frame set at bench level and provided with cradles (A and B).
The cradles hold the members of a lattice in their proper positions relative to one another. If the members are already fitted with attachment plates (flanges, gussets etc.) welded at a preassembly stage, the cradles can be replaced by corresponding plates or brackets (c) for bolting up.
Another procedure using a "marking off slab" consists of simply marking the concrete floor of the workshop. This is applied when the construction of an assembly frame is not justified.
The equipment for a rotation jig (Fig. 5.1 and 5.2) is intended for clamping down the structural parts in their correct positions for welding together and the rotation of the whole workpiece around a pivot line. The economic application of a jig is very much dependent on the number of units to be manufactured as well as on the degree of manufacturing precision required for a project.
Tack welding has been adequately discussed in Chapter 3.6.3.

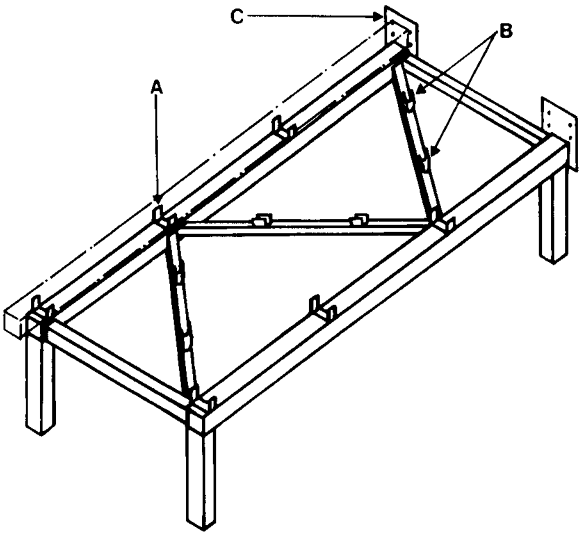


Fig. 5.4 – Assembly frame with cradles

The shrinkage and distortion, which evidently takes place during welding, should preferably be compensated for by jig-constraining, etc. (see Fig. 3.53, Chapter 3.6.5).

The following further factors have also to be considered while planning the fabrication:

- Working space
- Storage and stacking facilities for the structural elements
- Machine tools
- Skilled labour

Optimum cost effectiveness can only be reached when the above mentioned logistical points are properly considered.

One further item, which is also important for the economy of production, is the distance of the place for fabrication and assembly and the store for hollow sections with stacking facilities. The closeness of these two places, as well as the facilities for convenient and smooth transport, save time and labour and add to the economy.

Two alternatives are possible to fabricate a subassembly of hollow section construction:

1. The positions of the various structural components are marked off in relation to one another on the assembly frame. Proper inspection is required to prevent mistakes during this operation. The components are then assembled and joined by tack welding on the table.

   The subassembly is then moved to the welding shop, where welding is finalized following a pre-determined sequence, which promotes a reduced distortion.
2. Structural subassemblies are clamped down in position in the welding shop (possibly with allowance for subsequent distortion) and then the final welding is performed.

In some cases, the assembly of a structure is done on site, which necessitates the following special points to be accounted for:

1. In site construction, as the position and direction of the centre of gravity differs from that for the assembly in the workshop, it is necessary to keep the fittings such as cradles etc. until the subassembly is completed.
2. Auxiliary materials for construction such as erection pieces (see Fig. 5.5), which are often used, should be checked to determine their strength in accordance with the location, shape and size of the installation.

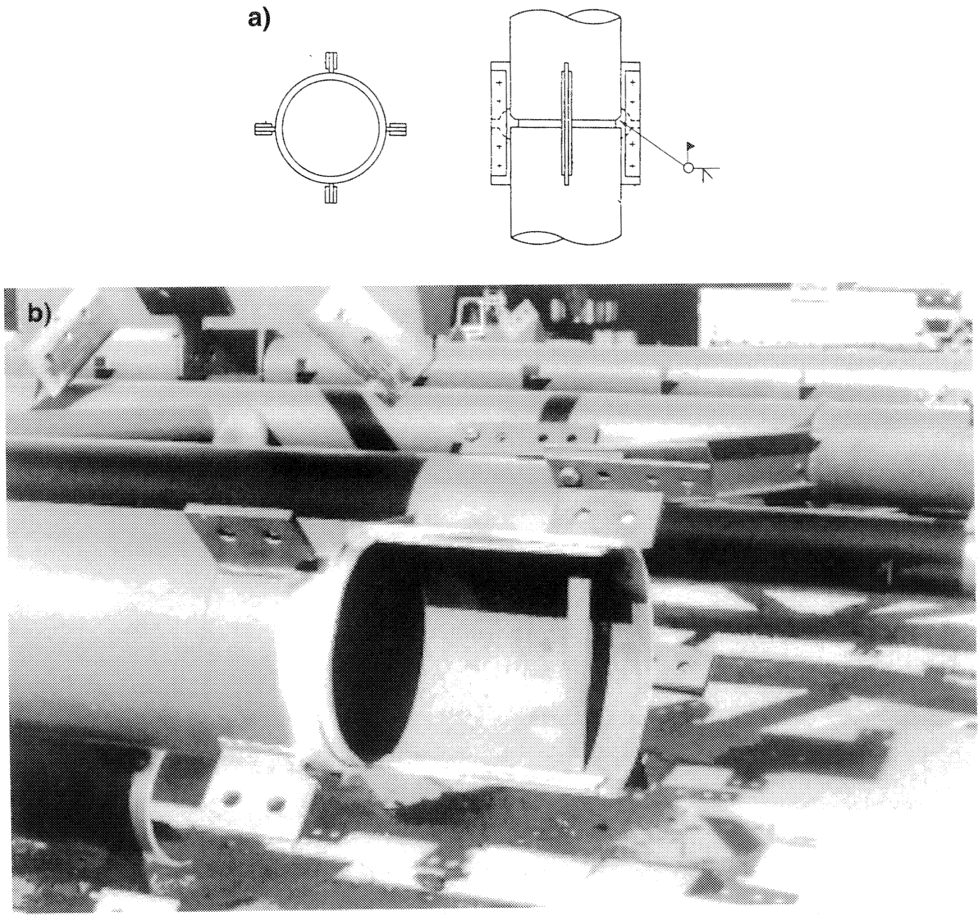


Fig. 5.5 – Erection piece

3. Besides selecting the most convenient welding method, weather conditions, such as wind, rain, temperature and humidity, are to be taken into account. Gas shielded arc welding requires protective equipment against the wind especially. Recently, the self shielded arc welding method is used more and more and semi-automatic or robotized equipment has been developed and applied. Protection from the weather is essential (see Fig. 5.6).
4. Often on the site the welding power source and welding locality are far from each other and it is necessary to have a remote control function, which enables the welder to adjust the welding condition at hand.

Fig. 5.6 – Site welding – protection from the weather is required for on-site welding

6 Transportation of hollow sections and structures

Depending on whether the structure is assembled in the workshop or on site, transportation of subassemblies or single structural elements to the site by road, rail, river, canal or sea has to be considered on the basis of cost effectiveness.

Stacking is one of the points which favours square and rectangular hollow sections with regard to the transportation by trucks, railways trains or ships (see Chapter 5.3). The mechanical properties of hollow sections, both CHS and RHS, are however very favourable for the transportation of single elements as well as large fabricated assemblies due to their high torsional rigidity, tensile and compressive strength and multi-axial bending resistance. The specific characteristics of the types of transport mentioned above are summarized below.

Road transport:
This type is convenient and mostly used, if the site is relatively near to the workshop. The authorised maximum heights, widths and lengths of the products are, however, limited and vary from country to country. Care must also be taken regarding local restrictions imposed by bridges, direct access to site, etc. These restrictions can be well met by making suitable subassemblies in the workshop and transporting them to the site, where they are fully assembled. Fig. 6.1 shows the configuration and construction of one type of subassembly, which can be economically stowed on a truck.

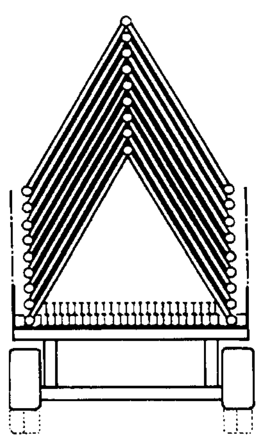


Fig. 6.1 – Transport of module units on a truck

Rail transport:
This type is the cheapest when the workshop and the jobsite are directly linked by a railway line. Size limits exist in this case also, which may vary from country to country.

Ship/boat transport:
This type of transport can be economic as the transportation of very large assembled units can be done. The condition is especially favourable when the jobsite and the workshop are directly on a major waterway or when the assemblies can be brought to a waterway without any problem. Fig. 6.2 shows the transport of the subassemblies for an offshore platform on a barge to be floated along to the location of oil exploration.

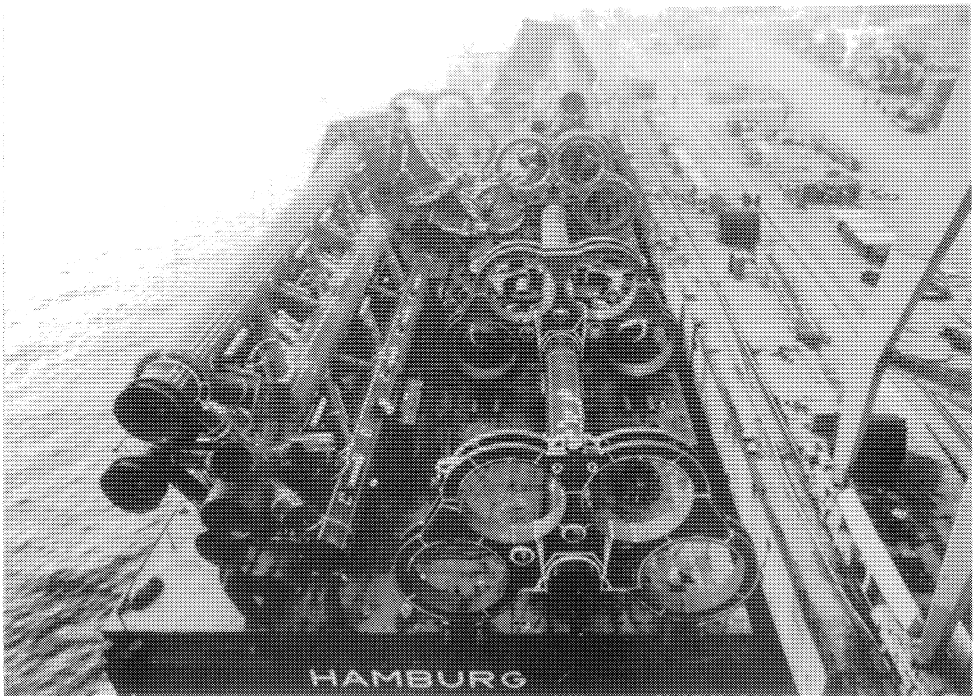


Fig. 6.2 –Preparation for the transport of the subassemblies for an offshore platform on a barge

7 Erection of hollow section structures

In principle there is no appreciable difference between the erection of hollow section structures and that of constructions with any other type of section. However, due to the superior torsional properties of hollow sections as described in Chapter 2.2, lifting and handling operations of hollow section assemblies are greatly facilitated. This is demonstrated by the comparison of the lifting of a lattice girder made of open sections with that of a hollow section lattice girder (Fig. 7.1). Temporary, transverse stiffeners or a lifting cradles are not necessary for a hollow section lattice girder, but most likely one would be required for a similar construction made of open sections.

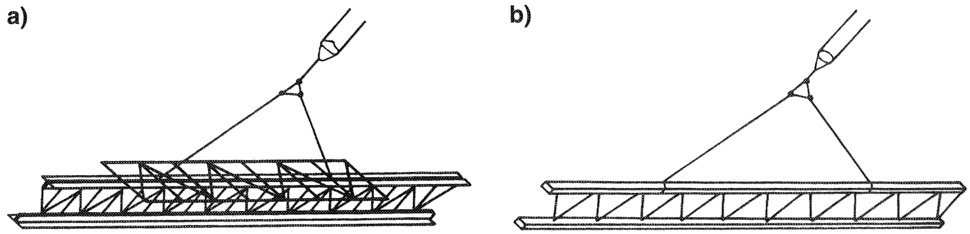


Fig. 7.1 – Lifting lattice girders a) Lattice girder of open sections with temporary traverse stiffener or lifting cradle b) Hollow section lattice girder without temporary transverse stiffener or lifting cradle

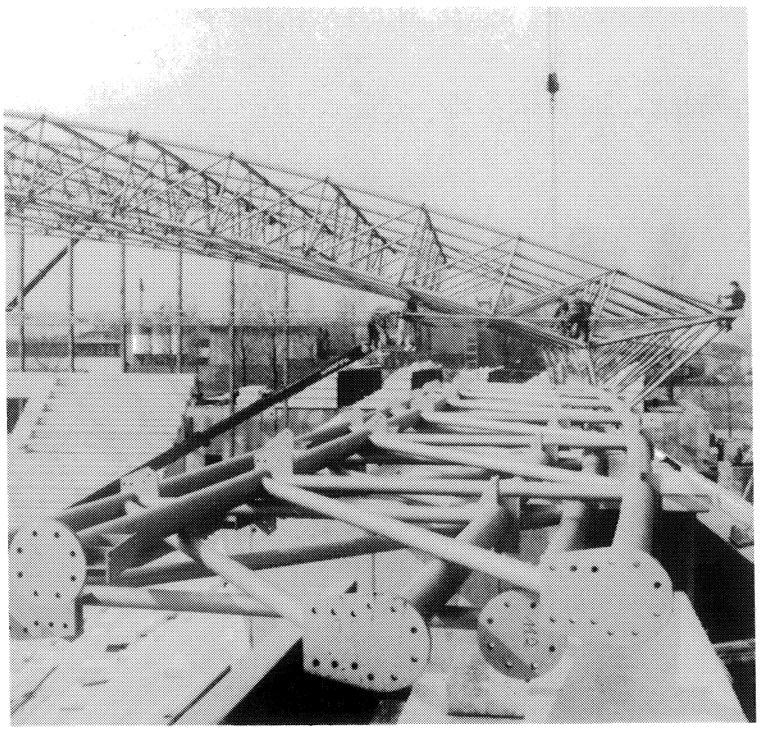


Fig. 7.2 – Erecting a CHS lattice structure for a stadium, prefabricated subassemblies are hoisted by a crane and flange-jointed

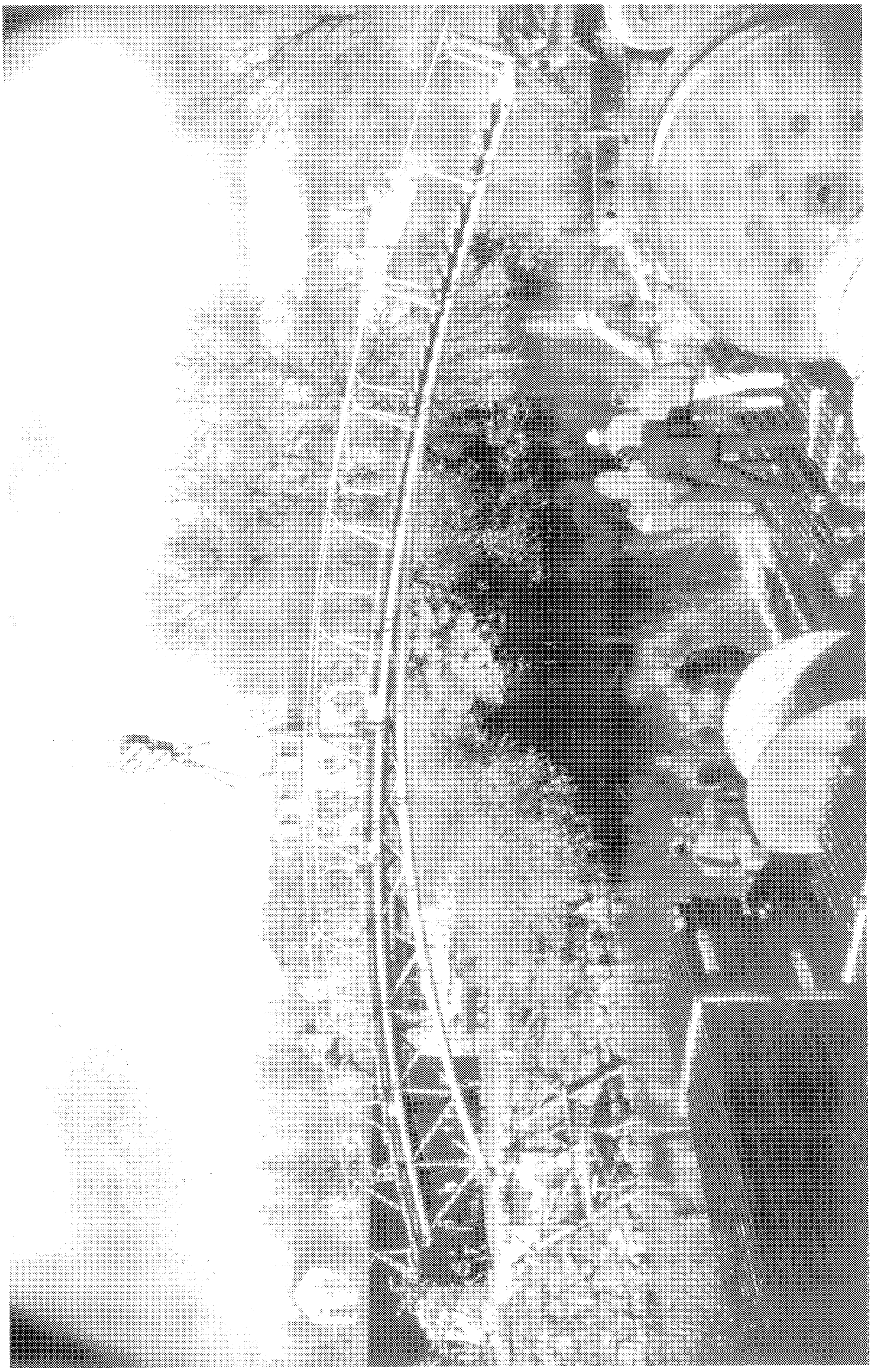


Fig. 7.3 – Erection of a lattice arched bridge

In general, the erection should start at a location where the access for the transport vehicles and handling equipment such as cranes is easier.
Mostly a laterally stable stanchion system in a row is gradually added with the other elements of the structure. Fig. 7.2 shows an erection operation, where lattice subassemblies are transported to the jobsite, hoisted by crane and joined to the other parts through bolted flanges.
The availability of powerful cranes allows lifting of a large unit transported directly from the workshop and bring it in position in one operation. Fig. 7.3 shows the erection of a CHS arched lattice bridge. The application of hollow sections in this case is favoured by the following factors:

- Relative lightness of the structure
- Extreme stiffness of the lattice structure
- Lighter resistance to the wind (although of marginal advantage), especially when CHS are used.

Ground level assembly of large units is widely used for space structures (see Fig. 7.4). This makes the erection operation easier and quicker and is safer for the personnel at the same time.

Fig. 7.4 – Assembling a "Delta" structure (see Chapter 3.8) on the ground at the jobsite, to be hoisted by a mobile crane to the final position

As has been already mentioned, the fabrication of uni-planar and multi-planar lattice structures is made in general in the workshops, when their sizes are relatively small and the required transport facilities are available. Otherwise the single elements have to be bolted or welded together on site. It may be useful to have the facility to carry out the assembly and the erection simultaneously at a height of 4 to 5 metres above the ground, so that small mobile lifting units can lift single structural elements to the final positions on the columns and there is enough free space under the construction for the movement of lifting units. Fig. 7.5 shows one such case.

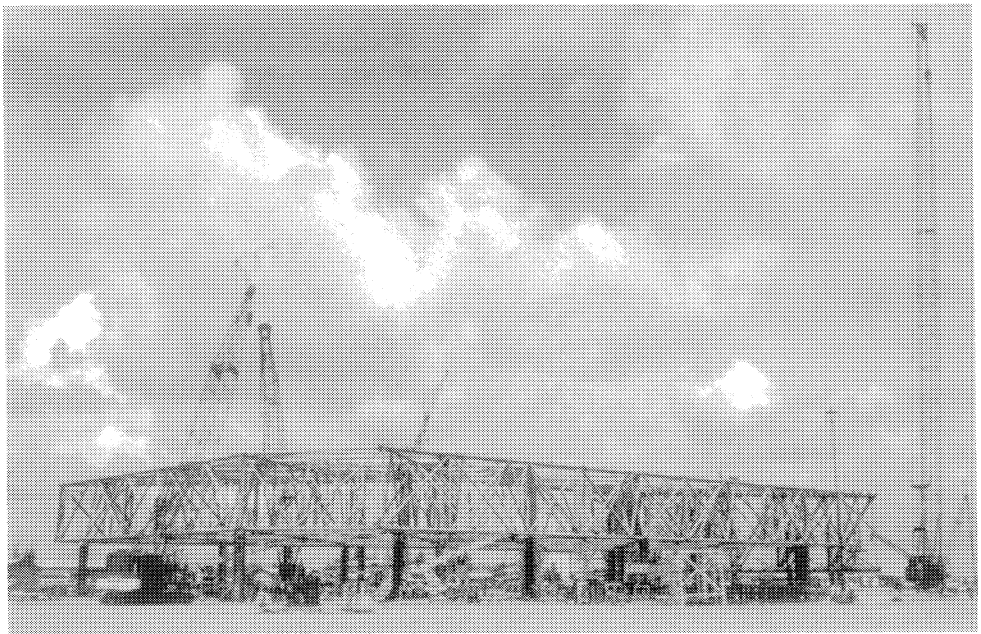


Fig. 7.5 – Assembly and erection of a structure at a height of 5 m with mobile cranes

A typical site assembly sequence, using bolted connections, is shown in Figs. 7.6a to 7.6c. Single members, Fig. 7.6a, which can reduce transportation costs, are delivered to site and bolted together to make the complete girder, Figs. 7.6b and 7.6c. When assembled the whole girder can be lifted into its final position.
With the correct equipment, very large hollow section constructions, weighing many tonnes, can be lifted into their final position. Figure 7.7 shows a very large hollow section lattice box girder being lifted into position using hydraulic rams at each end of the girder.

Fig. 7.6a – Bolted site assembly – ① individual components delivered to site

Fig. 7.6b – Bolted site assembly – ② simple site bolted connection

Fig. 7.6c – Bolted site assembly – ③ individual components bolted together to be erected in one large piece

Fig. 7.7 – Erection – very large pieces can be erected with the correct equipment. British Airways heavy maintenance hanger, Cardiff, Wales (dimensions 14.5 m × 8 m × 232 m; 1000 tonnes and 9 m × 5 m × 232 m; 600 tonnes)

8 Protection against corrosion

Corrosion is a phenomenon which is inherent to steel structures, when they are exposed to the atmosphere. The preventive measures can influence the economy of their use decisively in competition with concrete structures. The importance of protection against corrosion is increasing day by day with the rising agressivity of the atmosphere nowadays and its financial impact on the manufacture of steel structures has become considerable. It is therefore necessary to consider the merits and demerits of hollow sections with regard to this item and proper attention to it should start at an early stage of design.

Atmospheric corrosion is an electro-chemical process with reactions between steel and condensed humidity forming oxides or rusts. The notorious components of the atmosphere relevant to corrosion are H_2O, O_2 as well as CO_2, chlorides (in a sea climate), SO_2, nitric gases, H_2S and NH_3. The steel surface starts to rust at a relative atmospheric humidity above 60% and a temperature between -20^o and $+60^oC$. Dust deposits promote corrosion particularly as they absorb moisture. Circular hollow sections have a special advantage in this respect, as they allow lower dust deposits.

Due to the closed shape of hollow sections, corrosion can theoretically take place on the external as well as internal surface and both forms have to be accounted for. However, internal corrosion, see 8.2, is not normally a problem.

8.1 Protection against external corrosion

The methods of protection against external corrosion for hollow section structures are the same as for those for open sections, namely

1. paints and sprays
2. metal coatings, e.g. galvanising, metal spraying, etc.

Electro-chemical polarisation or cathodic/anodic protection, although normal for offshore structures and pipe lines, is not often applied in structural engineering.

Painting and spraying hollow sections enjoy significant advantages in comparison with open sections both technically and financially. These merits, which can be decisive for the choice of hollow section in a structure, are as follows:

1. Hollow sections offer, for a given load bearing capacity, an outside surface area that can be up to 50% less than that of the equivalent open sections. This can result in appreciable savings in material for corrosion protection and labour.
2. Sharp corners of an open profile are prone to rusting, since the coating is usually thinner there than elsewhere. Condensation also occurs at sharp corners leading to corrosion. As hollow sections do not actually have sharp corners, it is simpler to obtain uniform paint or spray thickness resulting in a better long time behaviour of corrosion protection (see Fig. 8.1).
3. As regards corrosion, steel structures should be designed in such a manner that they offer less or no surface for attack by corrosion agents, i.e. hollow space, dust-, water- or snow-traps etc. It is possible to design a hollow section structure with neat and uncluttered joints using no gusset or connection plates, where every part is readily accessible for initial protective treatment against corrosion and any maintenance work later (see Fig. 8.2).

Mainly these three points underline the economical competitiveness of hollow section structures not only for initial production but also for their maintenance in the future.

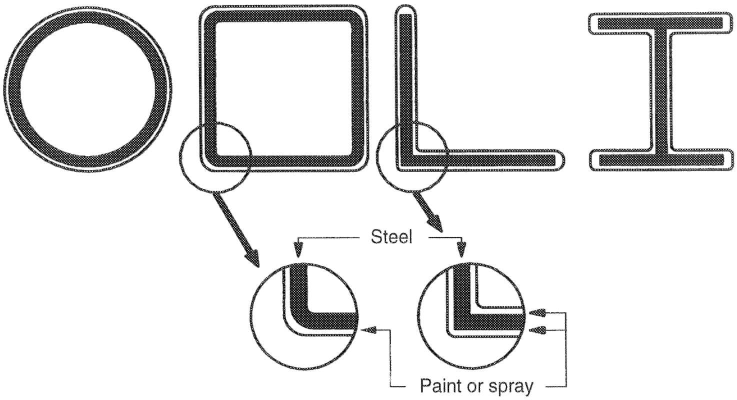


Fig. 8.1 – Comparison of the uniformity of layer thickness of paint and spray on open and hollow sections

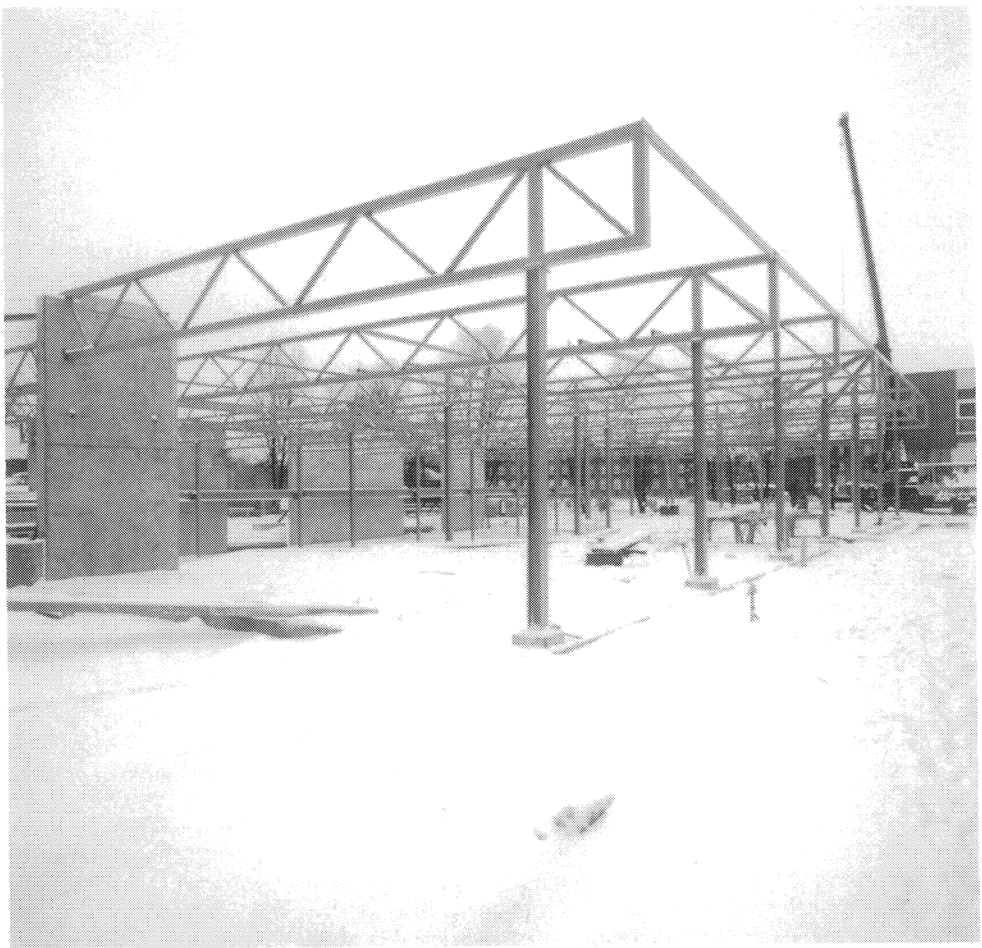


Fig. 8.2 – Hollow section structure with joints using no gusset or connection plates

8.1.1 Protection by painting and spraying

The choice of paints and sprays and their application methods has to be made taking the climate, the location, the prospective lifetime of the structure and the maintenance periods as well as the specific requirements for paints or sprays e.g. thickness, into consideration. Consisting normally of a primer-, an intermediate- and a top-coating, the paints are mostly combinations of organic bonding agents and inorganic pigments.
The primer coat has the vital function to inhibit the corrosion process on the steel surface by means of its high resistance to permeability and to ensure a good bond between the subsequent coating and steel surface. The intermediate coat, which is chemically inert and impervious to external atmosphere, acts to improve the bond between primer and top coat. The top coat is also impervious and resistant to mechanical and chemical actions, and it can also have a decorative function in some cases.
Appropriate surface preparation prior to the application of the corrosion protection is compulsory, since poor results are to be expected in cases of insufficiently clean or slightly rusty surfaces. Depending on the surface conditions, it is prepared as follows:
- Descaling by shot blasting, abrasive discs or grinding, manual or pneumatic hammering, needle gun scouring, depending on how tenacious the layers of the scales are.
- Cleaning by removing any foreign matter stuck to the wall, e.g. dust, mud, etc.
- Degreasing by spraying emulsions depending on the nature of grease to be removed. The emulsions should be environmentally permissible.

Paints are applied usually by the following techniques:
- Brushing
- Spraying (pneumatic by pulverisation or electrostatic)
- Dipping

Brushing is strongly recommended for the first coat. Metal coatings are made by gun metallising, which consists of spraying metallic zinc or aluminium, melted in an oxyacetylene or oxypropane flame, by a compressed air jet. This type of corrosion protection is highly suitable for large structures. Prior to spraying, the surface has to be shot blasted.
Zinc rich paints, with 34% metallic zinc content, are specially recommended here for their excellent protective quality and easy application, as they also dry and harden very quickly.
The life time of corrosion protection can be prolonged significantly by Duplex-coating, which consists of an organic layer onto a metallic coating.
Nowadays hollow sections are often delivered by the mills to the fabricators in a pre-protected state. For this purpose, they are descaled and cleaned in the mill and subsequently coated with a "shop primer". The coating has a thickness between 15 and 20 μm and is able to be overwelded directly. It has the function of protecting the surface against corrosion during transport, storage and fabrication. As the primer coat is applied directly onto the shop primer, they have to be compatible with each other.

8.2 Protection against internal corrosion

The "state of the art" recommendation is that no protective measure against internal corrosion of hollow sections is required, provided they are airtight sealed at all openings. This is done in general by welding cap plates at the ends or using sealing devices e.g. washer, gasket ring for the bolts.
In these cases, the corrosion process is quickly stopped by the lack of renewed oxygen after the initially closed-in oxygen is exhausted. This has been proven by CIDECT by extensive research works [29] as well as by a long term investigation by the German Railways [36].
The investigations have further demonstrated the following:
1. Although humid air might enter into a hollow section through the openings, which are not perfectly sealed (weld notches for example), rust can only develop in a very restricted small area around the opening.

2. If in any case surface water is sucked into the hollow section and drains down the inner wall, corrosion is restricted to the wetted locality only. This can be stopped by drilling a hole at a low spot to drain any water away (see Fig. 8.3a), or by using a drainage flange at the bottom of a column, if ring flanges are used (Fig. 8.3b).

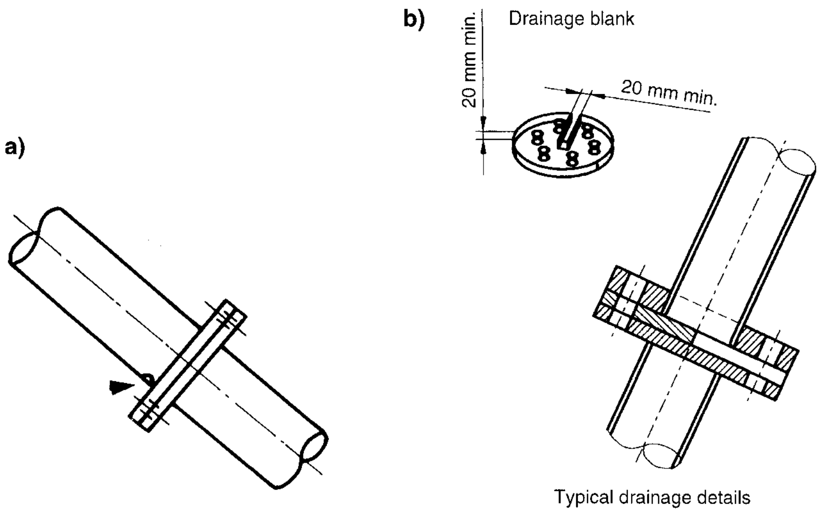


Fig. 8.3 – Vent hole drainage (a) or drainage flange (b)

Greater danger is expected in external structures, when it is possible for water to collect within a hollow section, because repeated freezing can lead to rupture of the wall through successive expansion and contraction until the elongation limit is reached [28]. For the constructions exposed to this danger, a hole with a drain plug is recommended to be drilled to remove the collected water from time to time; an alternative is to use a drainage flange.
When it is not possible to seal a hollow section due to constructional reasons, care has to be taken that air can pass through and water does not stagnate in the hollow section.

8.3 Protection against internal and external corrosion by hot dip galvanising

Hot dip galvanising is a reliable and economical process for the corrosion protection of both internal and external surfaces of hollow sections. The procedure consists of the following steps:

1. Steel surfaces are cleaned, degreased, pickled by chemical acids, rinsed and fluxed.
2. Hollow sections or subassemblies or finished assemblies (depending on the dimension of the available bath) are immersed in a bath of molten zinc at 470°C, where a non-porous layer of metallic zinc develops on the steel surface. The intermediate layer of iron-zinc-alloy between steel and pure zinc developed by diffusion acts as a bonding agent between them. The thickness of zinc coating in terms of grams per dm^2 depends on the environmental conditions i.e. rural, maritime, urban, industrial, tropical, etc. and structural requirements. Common values for coating thickness are between 4 and 6 grams per dm^2 surface.

The following points must be taken care of:

1. Hollow sections and assemblies must have holes and openings, so that air can get out of them without any problem, which can otherwise be a source of explosion. All connections in an assembly must be designed so that they allow unrestricted flow of pickling acids, rinsing agents and molten zinc through them as well as through the inner spaces of hollow

sections (Fig. 8.4). It has to be borne in mind that the smallest leak of pickling solution into the inner space will lead, on immersion into molten zinc at 470°C, to sudden vaporisation causing possible bursting of the hollow structural members. The arrangement of the outlet holes or openings will have to be made accordingly.

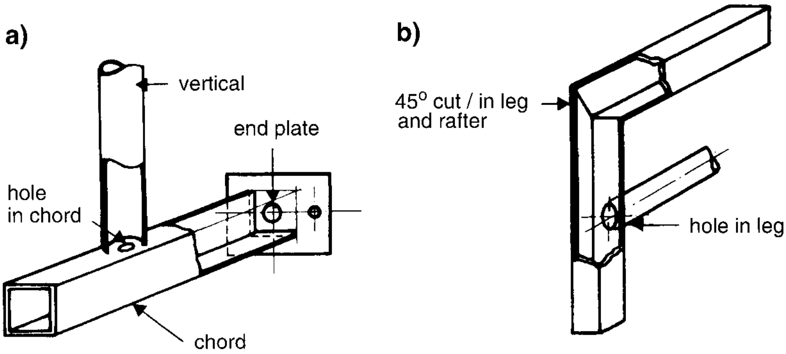


Fig. 8.4 – Arrangements of holes in hollow section connections a) T type joint b) Knee joint for hot dip galvanisation

3. In case the zinc bath is not large enough to accomodate an assembly, single hollow section members are hot dip galvanised and assemblies are made by welding or preferably bolting the members subsequently. The welding procedure for galvanised parts has been described in Chapter 3.6.9.
4. Residual stresses inherent to a welded structure are relieved by hot dip galvanising leading to deformations. They have to be accounted for while galvanising as best as possible. The deformations can be adjusted or compensated for within limits.

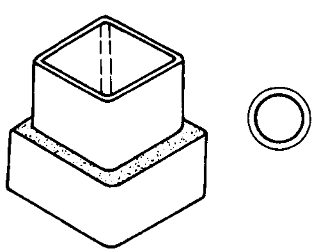
8.4 Protection of hollow section structures against corrosion inside buildings

In most cases, no protection measure is necessary for this type of structures e.g. concealed and visible deck frames, provided the following conditions are fulfilled:
- No condensation can take place (especially in roof areas) during the seasons
- The relative humidity is lower than 50%.

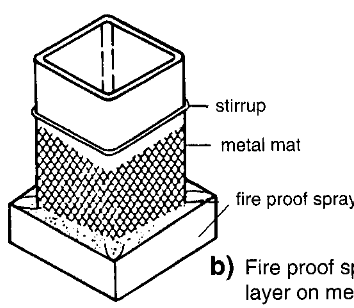
9 Protection against fire

As an outbreak of fire can never be excluded totally for any building, the client, the architect, the engineer and the fabricator must be aware of the level of fire safety of a steel structure directly starting from the initial design considering also the economical consequences. Laws, regulations and standards in various countries as well as international recommendations [41] aim to define and prescribe various security levels for structures as well as parts of them depending on their functions, e.g. single-storeyed, multi-storeyed, load bearing, minor or no loading, shielded against fire or not etc. The fundamental aim is to save human lives before preventing material losses. Prevention of fire and protection against it are the two aspects, which form the basis of the fire safety considerations. Prevention consists of dissemination of information about how to stop outbreak of fire and training people to act accordingly as well as to prevent the propagation of fire by providing buildings with alarm systems and fire extinguishers. Protection of human life is assured by the facilities for evacuation prescribing the number and sizes of emergency exits as well as by delaying the onset of mortal danger through prolonged retainment of the integrity of structures or parts of them e.g. beams, columns, lattice girders, etc. after fire outbreak.

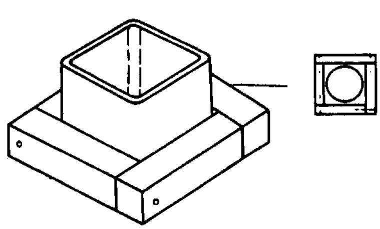
Integrity of a steel structure is determined on the basis of the critical temperature of 450 to 550°C, at which steel members fail and the fire resistance time before the critical temperature is reached at a certain load level. Steel structures, structural elements and fire protection materials are classified into the fire resistance classes F 30, F 60, F 90, F 120 and F 180, where the numbers designate the fire endurance time in minutes. Unprotected steel members fail in fire after some 15 to 30 minutes depending on the load and massivity, which is the ratio of the circumference to the cross sectional area. This means that the fire endurance time of thick walled hollow sections is greater than that of thin walled ones.



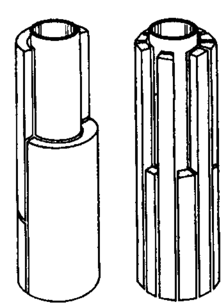
a) Plaster by in situ spraying



b) Fire proof sprayed layer on metal mat



c) Nailed board of fire proof material



d) Adhesive bonded fire proof boards

Fig. 9.1 – Hollow sections with external fire insulations

One major advantage of hollow sections with regard to fire protection is that this can be achieved not only by external insulation as for conventional open sections, but also by filling them with concrete, particularly reinforced concrete or by water circulation through the internal space. The design and calculation of these three fire protection procedures have been elaborately described in [4]. So they are dispensed with here except for showing Fig. 9.1, which illustrates the details of external insulations on hollow section columns.

These materials can be classified into the following groups:

- Insulating boards or panels (based mainly on gypsum or mineral fibre or lightweight aggregates such as perlite and vermiculite)
- Spray coating or plaster (based mainly on mineral fibre or lightweight aggregates such as perlite and vermiculite)
- Intumescent paints (mixtures applied directly to the steel surface, which swell up in fire to a multiple of their original thickness)

Fire protection requirements are a source of an additional expenditure for steel structures and can impair their competitiveness against concrete structures significantly. Therefore, strong effort has to be made to apply them as economically as possible. A few points in this respect are given below:

1. Due to external insulation of columns, especially in multi-storey buildings, the lettable floor area is reduced, thus decreasing the revenue. Care should be taken to keep the insulation as thin as possible, any unnecessary thickening should be avoided.
2. Concrete filling of hollow section columns, preferably with reinforcements, increases not only the load bearing capacity [5] but also the fire endurance. This can be achieved also without any external insulation leading to slender columns, which is architecturally pleasant and increases the revenue at the same time.
3. Application of water cooling of unprotected columns can be economical against external insulation starting from an eight-storey building upwards [37].

10 Economical aspects regarding building steel structures with hollow sections

A statement of international validity with respect to the overall competitiveness of hollow section structures against structures of conventional open sections is practically impossible to make. This is due to the fact that the cost situation for individual items such as material, labour and protection against corrosion and fire may be very different from country to country. As for example, the labour costs in the less industrialised and underdeveloped countries do not play such a big role relative to the material costs as they do in the highly industrialised countries, where also the level of the labour costs may differ significantly from one another.

As regards material costs, the price per tonne of hollow sections is usually higher than that of open profiles. However, here also the differences in price in different countries do not show a uniform level.

Costs for the protection against corrosion and fire are dependent on how stringent the requirements are prescribed by the standards or the regulations by the authorities of a particular country. This item can also considerably affect the overall economy of a steel structure in a country.

Because of the reasons mentioned above, this chapter assesses the economical aspects of hollow section structures with a technical approach i.e. by means of describing the technical advantages, which raise the competitive status of hollow section structures. Most of these points have already been mentioned in the previous chapters. However, they are summarized here for a final assessment:

1. Hollow sections of circular, square and rectangular cross sections offer outstanding static strength properties related to compression, torsion, multiaxial bending and lateral buckling (see Chapter 2, Fig. 2.4). These properties make it possible to build lighter constructions with hollow sections than those with open profiles. However, in some cases the saving in material can only partly compensate for the material cost.

   The superior properties of hollow sections can be best exploited when they are especially used as columns (see Chapter 4.2), lattice structures (see Chapter 4.1.2) as well as in mechanical applications [6]. Long span roof structures normally used for sports, swimming and exhibition halls and stadiums, as well as industrial buildings are increasingly designed as lattice structures by the architects. Hollow section structures are preferred not only for their lighter weight but also for their aesthetic appearance with their smooth surfaces without any sharp edges.

   Over and above, the direct welding of the members in the connections is an uncomplicated process, whereby gusset and stiffening plates can be dispensed with.

   In a lattice construction, most of the material is, generally, in the chords (approx. 75% of the overall weight of the lattice work). Optimum utilization of the chords is therefore most important for a minimum weight design. In case of the necessity of a new dimensioning due to inadequate load bearing capacity of a lattice girder joint during an intermediary check, it is recommended to check first whether the bracings can be changed. While the gradation of chords does not offer any advantage, it is helpful to combine bracings in individual groups. It is also important for the designer to check beforehand whether the hollow sections of the selected dimensions are available on the market.

2. The costs of grit blasting, primer coating (see Chapter 8.1.1) and the actual fabrication and assembly (see Chapter 3 and 5) constitute the costs of fabrication.

   For grit blasting and primer coating, hollow sections enjoy a more favourable position than open sections, as their surface areas are smaller then those of corresponding open sections and consequently the processing costs for hollow sections are lower (see Chapter 8.1).

   In a welded lattice girder construction, nearly all of the fabrication costs are in the bracings. Hence, the most economical solution for the fabrication can be attained by:

1. reducing the number of bracings by choosing the best girder layout
2. using an appropriate joint type, which is relatively easy to fabricate
3. using an appropriate type of hollow section (CHS or RHS)

The effect of girder layout is shown in Fig. 10.1 by means of three different bracing types: N-type, KT-type and K-type. It is demonstrated that the lowest number of bracings as well as of connections can be obtained by the K-type (see also Chapter 4.1.2).

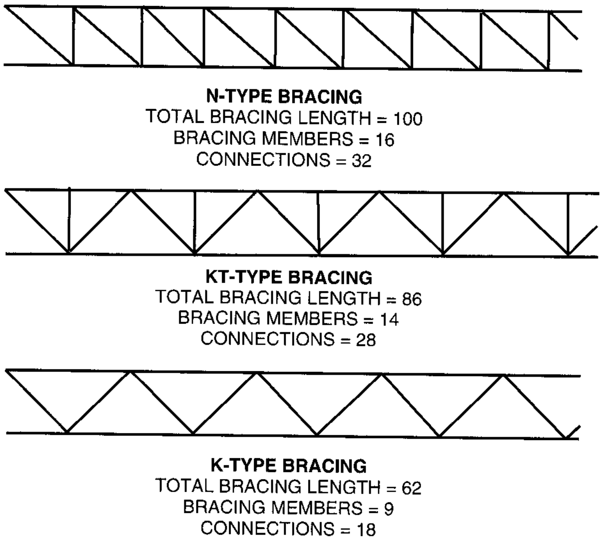


Fig. 10.1 – Girder layout

The influence of joint types with gap, 100% overlap and partial overlap on the fabrication of a lattice girder, as shown in Fig. 10.2, is quite significant (see also Chapter 4.4.3.4).

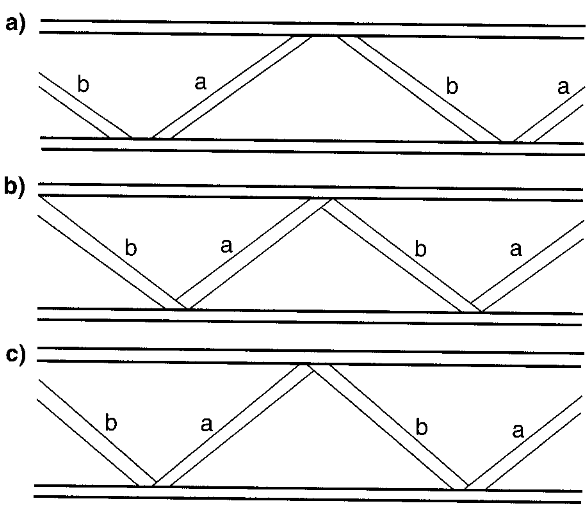


Fig. 10.2 – Joint types a) with gap b) with 100% overlap c) with partial overlap

Gap joints are the easiest to fabricate since there is only a single cut at each end of the bracing and for K-joints the angle of cut is the same at each end.

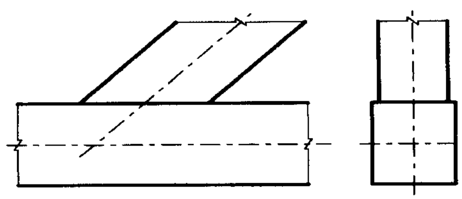
100% overlap joints also require a single cut at each end of bracing, but the angle of cut at each end will be different.

Partial overlap joints require double cuts at one end of each bracing and a single cut at the other end; again the angle of cut at each end will be different.

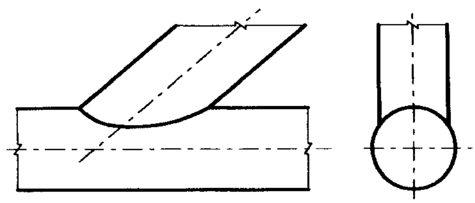
The cost of a joint depends on the joint type and the type of hollow section (CHS or RHS) as well as end shaping (plane cut or profile cut). Another item that can well make a difference in fabrication costs is the type of equipment that the fabricator uses. An idea of the relative costs in rising order is shown below:

Cheapest:	RHS chord gap joint bracing
	RHS chord 100% overlap bracing
	CHS chord gap joint bracing
	RHS chord partial overlap bracing
	CHS chord 100% overlap bracing
Dearest:	CHS chord partial overlap bracing

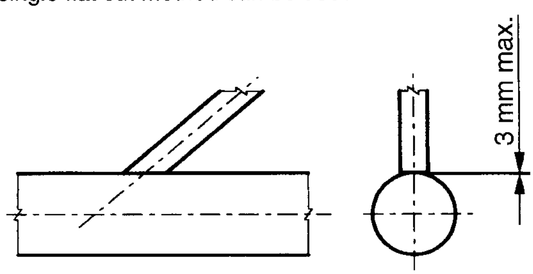
RHS CHORD GAP JOINT BRACING – a single plane saw cut



CHS CHORD GAP JOINT BRACING – profile shaping is nearly always necessary, unless the bracing is much smaller than the chord

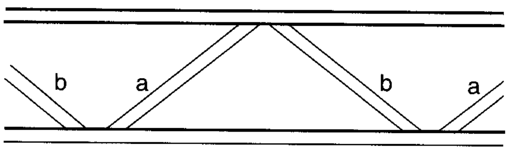


CHS CHORD GAP JOINT WITH VERY SMALL BRACING – a single plane saw cut is possible. As a rough guide the bracing diameter needs to be less than a third of the chord diameter before this single flat cut method can be used.

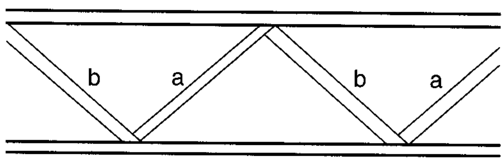


The choice of joint types, as well as itself affecting the cost of member and preparation, can also have an effect on the ease and hence the cost of fabricating the girder itself depending on the fit-ups and tolerances.

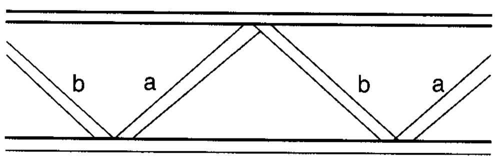
GAP JOINTS – these are the easiest to fabricate. The allowable tolerances are quite high since small adjustments can be made at each joint to ensure that the panel point positions are maintained



100% OVERLAP JOINTS – more difficult to fabricate than gap joints, since much less in the way of adjustment is possible, and if the bracing member lengths are not accurate, accumulated errors in panel point positions can result



PARTIAL OVERLAP JOINTS – these tend to be more difficult to fabricate than 100% overlap joints, since there is even less adjustment possible and if the bracing member lengths are not accurate, accumulated errors in panel point positions can result



The fabrication of lattice girders made of open sections, the dimensions of which are larger than those of the hollow sections in a corresponding girder, is in most cases more complicated since the gusset or connection plates can seldom be dispensed with. The total weld lengths are always larger making the total labour costs higher.
The surface area of a lattice girder of open sections is always larger than that of a corresponding lattice girder of hollow sections. This increases the cost for grit or shot blasting and primer coating in comparison with the hollow section girder.

3. The difference in painting costs for the finishing coat due to the big difference in surface area between hollow and open sections is the outcome of less painting material and lower labour cost for the hollow section structures (see also chapter 8.1). This can play a decisive role economically for the choice of hollow sections for a structure.
4. Other costs such as for static calculations, workshop drawings and overheads are more or less independent of dimensioning with open profiles or hollow sections. Transportation and erection of hollow section structures may in some cases be more economical than those of structures in open sections (see also Chapter 6 and 7).

In order to give an idea of the economical competitiveness of a lattice girder of 25 m span in RHS against that in open sections, Fig. 10.3 shows a comparison of various costs, the validity of which is restricted to Germany [73].

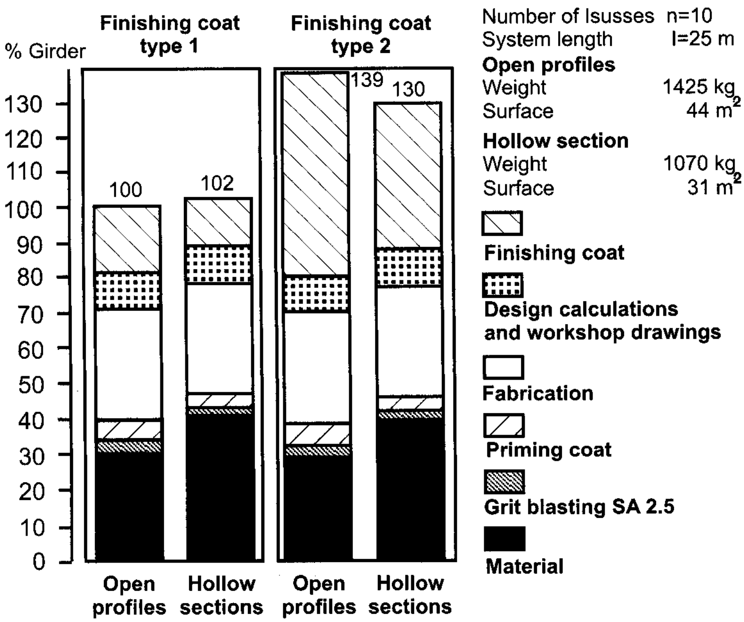


Fig. 10.3 – Comparison of the various costs for welded lattice girders with open profiles and hollow sections

In this case, it is interesting to note that the painting costs are decisive for the choice of RHS. While using a simple painting material at a lower price, the total costs for the lattice girders of open and hollow sections are more or less equal. The use of a high-grade variant of painting material for a higher price on the other hand leads to a more economical solution by RHS than by open sections.

11 List of Symbols

CHS Circular hollow section
RHS Rectangular hollow section
HF Hot finished
CF Cold formed
ASTM American Society for Testing and Materials
AISC American Institute of Steel Construction
AWS American Welding Society
AS Australian Standard
AIJ Architectural Institute of Japan
CSA Canadian Standards Association
CEN Comité Européen de Normalisation (European Committee for Standardisation)
EC Eurocode
EN Euronorm (European Standard)
ENV Europäische Vornorm (European prestandard)
IIW International Institute of Welding
JIS Japanese Industrial Standard
ISO International Standards Organisation
ERW Electric Resistance Welding
SAW Submerged Arc Welding
a throat thickness of a weld
a_p throat thickness of the weld to a reinforcement plate
B, b external width
b_i external width of square or rectangular hollow section for member (i = 0, 1, 2, 3)
D, d external diameter
d_i external diameter of circular hollow section for member i (i = 0, 1, 2, 3)
$d_{o.in}$ internal diameter of the chord member
$d_{i.in}$ internal diameter for member i (i = 1, 2)
e noding eccentricity for a connection
f_b buckling strength
f_u ultimate tensile strength of the basic material
f_y tensile yield strength
g gap between the bracing members ignoring welds of a K, N or KT connection at the face of the chord
H, h external depth
h_i external depth of square or rectangular hollow section for member i (i = 0, 1, 2, 3)
I moment of inertia for bending
I_t moment of inertia for torsion
M mass
N_i applied axial force in member i (i = 0, 1, 2, 3)
N_{op} chord preload (additional axial force in the chord member at a connection which is not necessary to resist the horizontal components of the bracing member forces)
O_v overlap
R radius of curvature

r_o	external radius of circular hollow section for the chord or external corner radius of rectangular hollow section
r_i'	internal radius of circular hollow section for member i (i = 1, 2, 3)
r_i	internal corner radius of rectangular hollow section
T, t	thickness
t_f	thickness of flange plate
t_i	thickness of hollow section for member i (i = 0, 1, 2, 3)
t_p	thickness of gusset or face reinforcement plate
t_w	thickness of web of I-beam
α, θ_i	included angle between bracing member i (i = 1, 2, 3)
β	angle of welding chamfer
γ_M	partial safety factor for the resistance

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Appendix A

Chemical compositions of structural steels (ladle analysis) acc. to ISO 630 [38]

Steel designation	Quality	Thickness in mm	C % max.	P % max.	S % max.	N_2 %[1)] max.	Method of deoxidation[2)]
Fe 360	B	≤ 16	0.18	0.050	0.050	0.009	
		> 16	0.20				
	C		0.17	0.045	0.045	0.009	NE
	D		0.17	0.040	0.040		GF
Fe 430	B	≤ 40	0.21	0.050	0.050	0.009	NE
		> 40	0.22				
	C		0.20	0.045	0.045	0.009	NE
	D		0.20	0.040	0.040		GF
Fe 510[3)]	B		0.22	0.050	0.050		NE
	C	≤ 16	0.20	0.045	0.045		NE
		> 16	0.22				
	D	≤ 35	0.20	0.040	0.040		GF
		> 35	0.22				

1) For steel treated with aluminium, the maximum nitrogen content may be increased to 0.015%. The nitrogen contents are specified, but they will be verified only if this is stated on the order. For steel made in an electric furnace, the maximum value can be 0.012%

2) NE = non-rimming
GF: These steels shall have a content of elements sufficiently high to produce a fine-grain structure, for example total aluminium greater than 0.02%

3) The contents for Mn and Si shall not exceed 1.60% and 0.55% respectively

Permissible deviation for the product anlaysis in relation to the ladle analysis specified

Element	Permissible deviation	
	Rimming steel	Non-rimming steel
C	+ 0.05	+ 0.03
P	+ 0.015	+ 0.005
S	+ 0.015	+ 0.005
Mn		+ 0.10
Si		+ 0.05
N_2	+ 0.002	+ 0.002

Appendix B

Chemical composition of hot finished and cold formed structural hollow sections of non-alloy steels acc. to EN 10210-1 (HF) [61] and EN 10219-1 (CF) [64]

	Steel designation	Type of deoxidation	C % max. Nominal thickness $t \leq 40$ mm	C % max. Nominal thickness $40 < t \leq 65$ mm	Si % max.	Mn % max.	P % max.	S % max.	N_2 % max.
EN 10210	S 235 JRH + HF	FN	0.17	0.20	–	1.40	0.045	0.045	0.009
	S 275 JOH + HF	FN	0.20	0.22	–	1.50	0.040	0.040	0.009
	S 275 J2H + HF	FF	0.20	0.22	–	1.50	0.035	0.035	–
	S 355 JOH + HF	FN	0.22	0.22	0.55	1.60	0.040	0.040	0.009
	S 355 J2H + HF	FF	0.22	0.22	0.55	1.60	0.035	0.035	–
EN 10219			Nominal thickness $t \leq 40$ mm						
	S 235 JRH + CF	FF	0.17		These values are identical to those according to EN 10210-1				
	S 275 JOH + CF	FF	0.20						
	S 275 J2H + CF	FF	0.20						
	S 355 JOH + CF	FF	0.22						
	S 355 J2H + CF	FF	0.22						

FN = Rimming steel not permitted
FF = Fully killed steel containing nitrogen binding element

Chemical composition of hot finished and cold formed structural hollow sections of fine grain steels acc. to EN 10210-1 (HF) [62] and EN 10219-1 (CF) [64]

	Steel designation	Type of deoxidation	C % max.	Si % max	Mn %	P % max.	S % max.	Nb % max.	V % max.	Al_{total} min.	Ti % max.	Cr % max.	Ni % max.	Mo % max.	Cu % max.	N % max.
EN 10210	S 275 NH + HF*	FF	0.20	0.40	0.50–1.40	0.035	0.030	0.050	0.05	0.020	0.03	0.30	0.30	0.10	0.35	0.015
	S 275 NLH + HF*					0.030	0.025									
	S 355 NH + HF*	FF	0.20	0.50	0.90–1.65	0.035	0.030	0.050	0.12	0.020	0.03	0.30	0.50	0.10	0.35	0.015
	S 355 NLH + HF*		0.18			0.030	0.025									
	S 460 NH + HF*	FF	0.20	0.60	1.00–1.70	0.035	0.030	0.050	0.20	0.020	0.03	0.30	0.80	0.10	0.70	0.025
	S 460 NLH + HF*					0.030	0.025									
	* Wallthickness ≤ 65 mm															
EN 10219	S 275 NH + CF**	All values are identical to those acc. to EN 10210-1														
	S 275 NLH + CF**															
	S 355 NH + CF**															
	S 355 NLH + CF**															
	S 460 NH + CF**															
	S 460 NLH + CF*															
	** Wallthickness ≤ 40 mm; Thickness over 24 mm is available only in CHS															

Appendix C

Formulae to calculate the geometrical properties of hot finished structural hollow sections according to EN 10210-2 [63]

Circular hollow section:

Nominal outside diameter	D (mm)
Nominal thickness	T (mm)
Nominal inside diameter	$d = D - 2\,T$ (mm)
Cross sectional area	$A = \dfrac{\pi\,(D^2 - d^2)}{4 \times 10^2}$ (cm^2)
Surface area per metre length	$A_s = \pi D/10^3$ (m^2/m)
Mass per unit length	$M = 0.785\,A$ (kg/m)
Second moment of area	$I = \dfrac{\pi\,(D^4 - d^4)}{64 \times 10^4}$ (cm^4)
Radius of gyration	$i = \sqrt{\dfrac{I}{A}}$
Elastic section modulus	$W_{el} = \dfrac{2\,I \times 10}{D}$ (cm^3)
Plastic section modulus	$W_{pl} = \dfrac{D^3 - d^3}{6 \times 10^3}$ (cm^3)
Torsional inertia constant (Polar moment of inertia)	$I_t = 2\,I$ (cm^4)
Torsional modulus constant	$C_t = 2\,W_{el}$ (cm^3)

Rectangular, including square, hollow section:

Nominal width of RHS	B (mm)
Nominal height of RHS	H (mm)
Nominal thickness	T (mm)
Nominal external corner radius for calculation	$r_0 = 1.5\,T$ (mm)
Nominal internal corner radius for calculation	$r_i = 1.0\,T$ (mm)
Surface area per metre length	$A_s = \dfrac{2}{10^3}(H + B - 4\,r_0 + \pi r_0)$ (m^2/m)
Cross sectional area	$A = [2\,T\,(B + H - 2\,T) - (4 - \pi)\,(r_0^2 - r_i^2)]\dfrac{1}{10^3}$ (cm^2)
Mass per unit length	$M = 0.785\,A$ (kg/m)

Second moment of area:

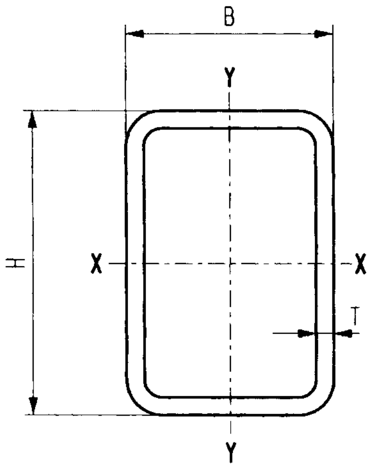
$$I_{xx} = \frac{1}{10^4}\left[\frac{BH^3}{12} - \frac{(B-2T)\,(H-2T)^3}{12} - 4\left(I_{zz} + A_z h_z^2\right) + 4\left(I_{\xi\xi} + A_\xi h_\xi^2\right)\right] \quad (\mathrm{cm}^4)$$

$$I_{yy} = \frac{1}{10^4}\left[\frac{HB^3}{12} - \frac{(H-2T)\,(B-2T)^3}{12} - 4\left(I_{zz} + A_z h_z^2\right) + 4\left(I_{\xi\xi} + A_\xi h_\xi^2\right)\right] \quad (\mathrm{cm}^4)$$

Radius of gyration:

$$i_{xx} = \sqrt{\frac{I_{xx}}{A}} \quad (\mathrm{cm})$$

$$i_{yy} = \sqrt{\frac{I_{yy}}{A}} \quad (\mathrm{cm})$$



Elastic section modulus:

$$W_{el_{xx}} = \frac{2I_{xx}}{H} \;(\times 10) \quad (\mathrm{cm}^3)$$

$$W_{el_{yy}} = \frac{2I_{yy}}{B} \;(\times 10) \quad (\mathrm{cm}^3)$$

Plastic section modulus:

$$W_{pl_{xx}} = \frac{1}{10^3}\left[\frac{BH^2}{4} - \frac{(B-2T)\,(H-2T)^2}{4} - 4\,(A_z h_z) + 4\,(A_\xi h_\xi)\right] \quad (\mathrm{cm}^3)$$

$$W_{pl_{yy}} = \frac{1}{10^3}\left[\frac{HB^2}{4} - \frac{(H-2T)\,(B-2T)^2}{4} - 4\,(A_z h_z) + 4\,(A_\xi h_\xi)\right] \quad (\mathrm{cm}^3)$$

Torsional inertia constant $\quad I_t = \frac{1}{10^4}\left[T^3\,\frac{h}{3} + 2KA_h\right] \quad (\mathrm{cm}^4)$

Torsional modulus constant $\quad C_t = 10\left[\frac{I_t}{T + K/T}\right] \quad (\mathrm{cm}^3)$

where

$$A_z = \left(1 - \frac{\pi}{4}\right) r_0^2 \quad (\mathrm{mm}^2)$$

$$A_\xi = \left(1 - \frac{\pi}{4}\right) r_i^2 \quad (\mathrm{mm}^2)$$

$$h_z = \frac{H}{2} - \left(\frac{10-3\pi}{12-3\pi}\right) r_0 \quad (\mathrm{mm})$$

$$h_\xi = \frac{H-2T}{2} - \left(\frac{10-3\pi}{12-3\pi}\right) r_i \quad (\mathrm{mm})$$

$$I_{zz} = \left(\frac{1}{3} - \frac{\pi}{16} - \frac{1}{312-3\pi}\right) r_0^4 \quad (\mathrm{mm}^4)$$

$$I_{\xi\xi} = \left(\frac{1}{3} - \frac{\pi}{16} - \frac{1}{312-3\pi}\right) r_i^4 \quad (\mathrm{mm}^4)$$

$$h = 2\,[\,(B-T) + (H-T)\,] - R_c\,(4-\pi) \quad (\mathrm{mm})$$

$$A_h = (B-T)\,(H-T) - 2\,R_c\,(4-\pi) \quad (\mathrm{mm}^2)$$

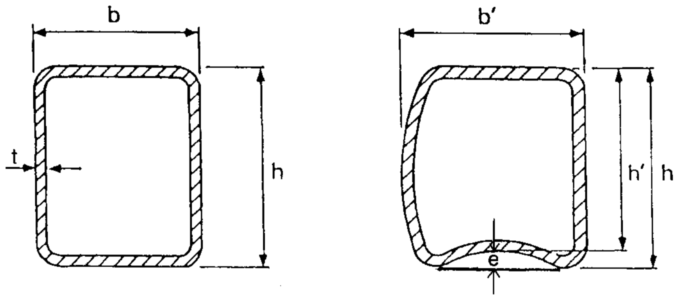
$$K = \frac{2A_hT}{h} \quad (mm^2)$$

$$R_c = \frac{r_0 + r_i}{2} \quad (mm)$$

Formulae to calculate the geometrical properties of cold formed hollow sections according to EN 10219-2 [65]:
All formulae are identical to those according to EN 10210-2 [63]. Except that the following external and internal corner radii are to be taken for calculation:

For $T \leq 6$ mm, $r_0 = 2.0\ T$ (mm) and $r_i = 1.0\ T$ (mm)
For $6 < T \leq 10$ mm, $r_0 = 2.5\ T$ (mm) and $r_i = 1.5\ T$ (mm)
For $T > 10$ mm, $r_0 = 3.0\ T$ (mm) and $r_i = 2.0\ T$ (mm)

Appendix D



$P_e = e/h \cdot 100\%$
$P_b = (b' - b)/b \cdot 100\%$

Minimum bending radii for square and rectangular hollow sections for cold bending with 3-roller bender [39]

Hollow section size	P_b		P_e		
h × b × t (mm) (mm) (mm)	1%	2%	0.5%	1%	2%
	Min. bending radius (m)		Min. bending radius (m)		
20 × 20 × 2.0	0.22	0.22	0.22	0.22	0.22
2.6	0.22	0.22	0.22	0.22	0.22
30 × 30 × 2.0	0.71	0.22	0.22	0.22	0.22
2.6	0.66	0.22	0.22	0.22	0.22
3.2	0.63	0.22	0.22	0.22	0.22
40 × 40 × 2.6	1.61	0.50	0.63	0.23	0.22
3.2	1.53	0.47	0.35	0.22	0.22
4.0	1.44	0.45	0.22	0.22	0.22
50 × 50 × 3.2	3.05	0.95	1.47	0.55	0.22
4.0	2.88	0.90	0.79	0.29	0.22
5.0	2.72	0.85	0.43	0.22	0.22
60 × 60 × 3.2	5.36	1.67	4.74	1.76	0.66
4.0	5.07	1.57	2.55	0.95	0.35
5.0	4.79	1.49	1.37	0.51	0.22
70 × 70 × 3.2	8.65	2.69	12.74	4.74	1.76
3.6	8.39	2.61	9.18	3.41	1.27
4.0	8.17	2.54	6.85	2.55	0.95
5.0	7.72	2.40	3.68	1.37	0.51
80 × 80 × 3.2	13.08	4.06	29.99	11.15	4.14
3.6	12.69	3.94	21.61	8.03	2.99
4.0	12.36	3.84	16.12	5.99	2.23
5.0	11.68	3.63	8.66	3.22	1.20
6.0	11.02	3.42	4.55	1.69	0.63

Hollow section size	P_b		P_e		
h × b × t (mm) (mm) (mm)	1%	2%	0.5%	1%	2%
	Min. bending radius (m)		Min. bending radius (m)		
90 × 90 × 3.2	18.83	5.85	63.82	23.72	8.82
3.6	18.28	5.68	45.98	17.09	6.35
4.0	17.80	5.53	34.29	12.75	4.74
5.0	16.83	5.23	18.42	6.85	2.55
6.3	15.87	4.93	9.68	3.60	1.34
8.0	14.94	4.64	4.98	1.85	0.69
100 × 100 × 3.2	26.10	8.11	125.40	46.61	17.33
4.0	24.67	7.67	67.38	25.05	9.31
5.0	23.32	7.25	36.20	13.46	5.00
6.3	22.00	6.83	19.03	7.07	2.63
8.0	20.71	6.43	9.78	3.64	1.35
10.0	19.58	6.08	5.26	1.95	0.73
120 × 120 × 3.2	45.92	14.27	403.57	150.01	55.76
4.0	43.41	13.48	216.84	80.60	29.96
5.0	41.03	12.75	116.51	43.31	16.10
6.3	38.70	12.02	61.23	22.76	8.46
8.0	36.44	11.32	31.49	11.70	4.35
10.0	34.44	10.70	16.92	6.29	2.34
140 × 140 × 3.6	71.87	22.23	781.09	290.34	107.92
5.0	66.15	20.55	313.00	116.34	43.25
6.3	62.40	19.38	164.48	61.14	22.73
8.0	58.75	18.25	84.59	31.44	11.69
10.0	55.53	17.25	45.45	16.89	6.28
150 × 150 × 4.0	86.66	26.92	906.58	336.99	125.26
5.0	81.91	25.45	487.11	181.06	67.30
6.3	77.27	24.00	255.98	95.15	35.37
8.0	72.75	22.60	131.64	48.93	18.19
10.0	68.76	21.36	70.73	26.29	9.77
160 × 160 × 4.0	183.32	46.29	2395.69	725.12	219.48
5.0	124.15	31.35	1255.53	380.02	115.02
6.3	82.92	20.94	642.98	194.62	58.91
8.0	54.64	13.79	321.95	97.45	29.50
10.0	37.00	9.34	168.73	51.07	15.46
180 × 180 × 4.0	273.35	69.02	3098.46	937.84	283.86
5.0	185.13	46.74	1623.83	491.50	148.77
6.3	123.65	31.22	831.60	251.71	76.19
8.0	81.47	20.57	416.40	126.03	38.15
10.0	55.18	13.93	218.22	66.05	19.99
200 × 200 × 5.0	264.66	66.82	2043.97	618.67	187.26
6.3	176.76	44.63	1046.76	316.83	95.90
8.0	116.74	29.41	524.13	158.64	48.02
10.0	78.88	19.92	274.69	83.14	25.17

Hollow section size	P_b		P_e		
h × b × t (mm) (mm) (mm)	1%	2%	0.5%	1%	2%
	Min. bending radius (m)		Min. bending radius (m)		
220 × 220 × 5.0	365.67	92.33	2516.95	761.83	230.59
6.3	244.23	61.67	1288.99	390.15	118.09
8.0	160.92	40.63	645.42	196.35	59.13
10.0	108.98	27.52	338.25	102.38	30.99
250 × 250 × 5.9	422.55	106.69	2060.56	623.69	188.78
6.3	376.81	95.14	1704.11	515.80	156.12
8.0	248.27	62.69	853.27	258.27	78.17
10.0	168.15	42.46	447.18	135.35	40.97
260 × 260 × 5.9	482.68	121.87	2244.85	679.47	205.66
6.3	430.43	108.68	1856.51	561.93	170.08
8.0	283.60	71.61	929.58	281.37	85.15
10.0	192.04	48.49	487.03	147.41	44.62
300 × 300 × 7.1	567.46	143.28	1794.40	543.13	164.39
8.0	460.71	116.32	1270.17	384.45	116.37
10.0	312.03	78.79	665.71	201.50	60.99
350 × 350 × 8.0	777.00	196.19	1777.92	538.14	162.88
10.0	526.28	132.88	931.90	282.07	85.38
400 × 400 × 10.0	827.66	208.98	1247.09	377.47	114.25

Hollow section size	P_b		P_e		
h × b × t (mm) (mm) (mm)	1%	2%	0.5%	1%	2%
	Min. bending radius (m)		Min. bending radius (m)		
50 × 30 × 2.6	2.43	0.76	0.87	0.32	0.22
3.2	2.31	0.72	0.49	0.22	0.22
4.0	2.18	0.68	0.26	0.22	0.22
60 × 40 × 3.2	4.30	1.34	1.97	0.73	0.27
4.0	4.07	1.26	1.06	0.39	0.22
5.0	3.84	1.19	0.57	0.22	0.22
70 × 40 × 3.2	6.38	1.98	3.79	1.41	0.52
4.0	6.03	1.87	2.04	0.76	0.28
5.0	5.70	1.77	1.10	0.41	0.22
80 × 40 × 3.2	8.97	2.79	6.69	2.49	0.92
4.0	8.48	2.73	3.59	1.34	0.50
5.0	8.01	2.49	1.93	0.72	0.27
90 × 50 × 3.2	13.68	4.25	17.88	6.65	2.47
3.6	13.28	4.12	12.88	4.79	1.78
4.0	12.93	4.02	9.61	3.57	1.33
5.0	12.22	3.80	5.16	1.92	0.71

Hollow section size	P_b		P_e		
h × b × t (mm) (mm) (mm)	1%	2%	0.5%	1%	2%
	Min. bending radius (m)		Min. bending radius (m)		
100 × 50 × 3.2	17.90	5.56	27.96	10.39	3.86
3.6	17.38	5.40	20.14	7.49	2.78
4.0	16.92	5.26	15.02	5.58	2.08
5.0	15.99	4.97	8.07	3.00	1.12
100 × 60 × 3.2	19.77	6.14	41.50	15.42	5.73
3.6	19.19	5.96	29.89	11.11	4.13
4.0	18.69	5.80	22.30	8.29	3.08
5.0	17.66	5.49	11.98	4.45	1.66
6.3	16.66	5.18	6.30	2.34	0.87
120 × 60 × 3.2	31.49	9.78	89.99	33.45	12.43
3.6	30.57	9.50	64.83	24.10	8.96
4.0	29.77	9.25	48.35	17.97	6.68
5.0	28.14	8.74	25.98	9.66	3.59
6.3	26.54	8.25	13.65	5.07	1.89
8.0	24.99	7.76	7.02	2.61	0.97
120 × 80 × 3.2	36.83	11.44	167.76	62.36	23.18
4.0	34.81	10.81	90.14	33.50	12.45
5.0	32.90	10.22	48.43	18.00	6.69
6.3	31.04	9.64	25.45	9.46	3.52
8.0	29.22	9.08	13.09	4.86	1.81
10.0	27.62	8.58	7.03	2.61	0.97
140 × 80 × 3.2	54.60	16.96	322.79	119.98	44.60
4.0	51.61	16.03	173.43	64.47	23.96
5.0	48.78	15.15	93.19	34.64	12.88
6.3	46.01	14.29	48.97	18.20	6.77
8.0	43.32	13.46	25.18	9.36	3.48
10.0	40.95	12.72	13.53	5.03	1.87
150 × 100 × 3.2	73.53	22.84	701.37	260.70	96.91
4.0	69.50	21.59	376.84	140.08	52.07
5.0	65.69	20.41	202.48	75.26	27.88
6.3	61.97	19.25	106.41	39.55	14.70
8.0	58.34	18.12	54.72	20.34	7.56
10.0	55.14	17.13	29.40	10.93	4.06
160 × 80 × 3.2	260.74	65.84	466.13	141.09	42.70
4.0	176.59	44.59	244.29	73.94	22.38
5.0	119.60	30.20	128.03	38.75	11.73
6.3	79.88	20.17	65.56	19.85	6.01
8.0	52.63	13.29	32.83	9.94	3.01
10.0	35.64	9.00	17.21	5.21	1.58
180 × 100 × 3.6	318.32	80.37	606.49	183.57	55.56
5.0	179.35	45.28	234.28	70.91	21.46
6.3	119.79	30.25	119.98	36.31	10.99
8.0	78.93	19.93	60.07	18.18	5.50
10.0	53.45	13.50	31.48	9.53	2.88

Hollow section size	P_b		P_e		
h × b × t	1%	2%	0.5%	1%	2%
(mm) (mm) (mm)	Min. bending radius (m)		Min. bending radius (m)		
200 × 100 × 4.0	376.44	95.05	397.69	120.37	36.43
5.0	254.95	64.37	208.42	63.09	19.09
6.3	170.28	42.89	106.74	32.31	9.78
8.0	112.19	28.33	53.45	16.18	4.90
10.0	75.98	19.19	28.01	8.48	2.57
200 × 120 × 4.0	380.16	95.99	725.03	219.45	66.42
5.0	257.47	65.01	379.97	115.01	34.81
6.3	171.96	43.42	194.59	58.90	17.83
8.0	113.30	28.61	97.44	29.49	8.93
10.0	76.74	19.37	51.06	15.46	4.68
220 × 140 × 4.0	526.93	133.04	1083.74	328.03	99.29
5.0	356.87	90.11	567.96	171.91	52.03
6.3	238.35	60.18	290.87	88.04	26.65
8.0	157.05	39.65	145.64	44.08	13.34
10.0	106.36	26.85	76.33	23.10	6.99
250 × 150 × 5.0	548.84	138.58	618.58	187.23	56.67
6.3	366.57	92.56	316.79	95.89	29.02
8.0	241.53	60.98	158.62	48.01	14.53
10.0	163.58	41.30	83.13	25.16	7.62
300 × 200 × 5.9	767.31	193.74	807.06	244.28	73.94
6.3	684.25	172.77	667.45	202.02	61.15
8.0	450.84	113.83	334.20	101.16	30.62
10.0	305.34	77.09	175.15	53.01	16.05
400 × 200 × 7.1	1450.81	366.32	343.11	103.85	31.43
8.0	1177.85	297.40	242.86	73.51	22.25
10.0	797.71	201.41	127.28	38.52	11.66
450 × 250 × 8.0	1766.34	445.98	444.41	134.51	40.71
10.0	1196.27	302.04	232.90	70.50	21.34
500 × 300 × 10.0	1717.29	433.60	377.75	114.34	34.61

Comité International pour le Développement et l'Etude de la Construction Tubuliare

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 Publications

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

The list of the Design Guides in English, French, German and Spanish already published and in preparation, is given below:

1. Design guide for circular hollow section (CHS) joints under predominantly static loading
2. Structural stability of hollow sections
3. Design guide for rectangular hollow section (RHS) joints under predominantly static loading
4. Design guide for structural hollow section columns exposed to fire
5. Design guide for concrete filled hollow section columns under static and seismic loading
6. Design guide for structural hollow sections for mechanical applications
7. Design guide for fabrication, assembly and erection of hollow section structures
8. Design guide for circular and rectangular hollow section joints under fatigue loading (in preparation)

In addition, taking into account the ever growing place of steel hollow sections into international high tech constructions a new book "Tubular Structures in Architecture" has been published with the sponsorship of the European Community.

Copies of the Design Guides or the Architectural Book may be obtained from members or from:

The Steel Construction Institute
Silwood Park
Ascot
Berkshire SL5 7QN
England

Tel +44 (0) 1344 23345
Fax +44 (0) 1344 22944
e-mail Farooq@steel-sci.com

Copies of Cidect Research Papers can also be obtained from The Steel Construction Institute. Please contact Dr Farooq Awan.

CIDECT Organization (1997):

- President: T. G. Wheeler (United Kingdom)
  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 execution 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:
(1998)

- Aceralia Transformados, Spain
- BHP Structural and Pipeline Products, Australia
- British Steel Tubes and Pipes, United Kingdom
- EXMA, France
- Hoogovens Buizen, The Netherlands
- IPSCO Inc., Canada
- Laminoirs de Longtain, Belgium
- Mannstaedt Werke GmbH, Germany
- Nippon Steel Metal Products Co. Ltd., Japan
- Rautaruukki Oy, Finland
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