9

# DESIGN GUIDE

## FOR STRUCTURAL HOLLOW SECTION COLUMN CONNECTIONS

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# Preface

Steel structural hollow sections, circular, square and rectangular, are some of the most efficient structural sections under compression loading. This design guide has been written to give the design engineer the information one needs to design hollow section column connections in the most efficient and economic way. Steel structural hollow sections are unique in the world of structural steel sections, because their geometry is such that their mass is distributed away from their longitudinal axis, making them ideal for use as columns.

This design guide is the 9th in a series that CIDECT has published under the general series heading “Construction with Hollow Steel Sections“. The previously published design guides in the series, which are all available in English, French, German and Spanish, are:

1. Design guide for circular hollow section (CHS) joints under predominantly static loading (1991)
2. Structural stability of hollow sections (1992, reprinted 1996)
3. Design guide for rectangular hollow section (RHS) joints under predominantly static loading (1992)
4. Design guide for structural hollow section columns exposed to fire (1995, reprinted 1996)
5. Design guide for concrete filled hollow section columns under static and seismic loading (1995)
6. Design guide for structural hollow sections in mechanical applications (1995)
7. Design guide for fabrication, assembly and erection of hollow section structures (1998)
8. Design guide for circular and rectangular hollow section welded joints under fatigue loading (2000)

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CIDECT
2004

# CONTENTS

# 1 Introduction

Steel structural hollow sections, whether they are circular, square or rectangular, are inherently more efficient as compression members than any other structural steel section, e.g. I-, H- or L-sections, etc., due to their geometric shape.

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.

Architects, design engineers, fabricators and erectors are all experts in their own particular fields, but traditionally have worked separately. The architect and the design engineer, between them, are responsible for the conceptual lay-out, the sizing of the members and, often with tubular construction, some initial detailing of the connections. All of these are generally aimed at the reduction of the material weight of the structure, often with too little thought to the fabrication, assembly and erection. This lack of communication between the various disciplines and, subsequently, an inadequate knowledge of the 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 other's requirements for the realisation of a satisfactory and cost effective structure.

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.

## 1.1 Design philosophy

The design philosophy, requirements and terminology can be considerably different from country to country, e.g.

- limit states or allowable stress design;
- requirements or not for robustness (also called structural integrity);
- material yield strength, tensile strength or a combination of both;
- the methodology and specific value of partial safety factors (or resistance factors) for both load and capacity;
- design details;
- the symbols used vary not only from country to country, but in some cases within the same country.

**Design method:** This design guide is written in terms of a limit states format, unless specifically stated otherwise. However, if the information given in this design guide is to be used in an allowable stress design, it is suggested that a safety factor of about 1.5 should be used on the capacities given in this design guide.

**Robustness:** In many countries the building codes and regulations have a robustness or structural integrity requirement. This requirement is that all connections, even simple shear ones with only vertical shear loads, must also have the ability to carry specified, nominal horizontal forces. This is to ensure that if accidental horizontal forces are present in a building, it and the individual connections will remain intact and will not collapse.

**Material strength:** Structural hollow sections are manufactured in many countries and product specifications can be quite different from one country to another. These differences can include method of manufacture (hot finished or cold formed), yield and tensile strength, elongation and impact properties, and also differences in the geometric properties and tolerances of the hollow section.

There are also differences in the definition of shear yield strength; some use 0.6 times the tensile yield strength and others the tensile yield strength divided by $\sqrt{3}$. In this design guide the latter, tensile yield strength divided by $\sqrt{3}$, has been generally used.

**Partial safety factors:** Different building codes use different philosophies for partial safety factors for resistance. Some codes, such as Eurocode No. 3 (CEN 1992), use partial ($\gamma$) safety factors for resistance which generally have values above or equal to 1.0 and are used as dividers, i.e. (nominal strength)/$\gamma_M$. Others, especially in North America and Australia, use resistance or capacity ($\phi$) factors with values equal to or less than 1.0 and are used as multipliers, i.e. $\phi$ (nominal strength). However, this difference in design methodology will usually make little difference to the design since the value of $1/\gamma_M$ is generally very nearly the same as $\phi$.

In this design guide, as with all previous CIDECT design guides, all the design expressions relating to the actual hollow section column already have the relevant partial safety (or resistance) factor incorporated ($\gamma = 1/\phi$) and as a result no further partial safety or resistance factors should be applied to them. However, for the beam members, plates, bolts, etc. the partial safety or capacity factors relevant to the design code being used by the design engineer should always be used. Thus, $\gamma_M$ or $\phi$ factors should only be used in this Design Guide where indicated.

**Design details:** Different codes and specifications use different design details for such items as bolt spacing, edge distances from bolt centres, effective lengths of welds, etc. This design guide does not attempt to lay down specific values for any of these and the values specified in the relevant code being used by the design engineer should always be applied. In some examples in this design guide, the authors have, for completeness, undertaken detail design using a local design code. These occasions are explicitly stated, and designers should still use their own local code when making similar checks.

**Symbols:** A wide variety of symbols are used around the world for the same thing. For example, in different codes, the symbol used for yield strength may be $F_y$ or $f_y$ or $p_y$ or $Y_s$ or $R_e$, etc. A list of symbols used in this design guide is given in Chapter 12.

# 2 Advantages of hollow section columns

The radius of gyration, especially about the minor axis, of a structural hollow section is significantly higher than that of an open section of a similar size and area. This results in a much lower slenderness ratio for the same effective length, and hence a higher compression capacity. Any residual stresses that may be in the section due to the method of manufacture are generally also distributed in a much more favourable way than those in open sections because of the different shape characteristics and this can also result in an increase in the compression capacity.

Structural hollow sections are generally available in lengths up to 12 or 15 m (40 or 50 ft), but in some circumstances longer lengths, up to 20 m, may be available. This means that for buildings of up to about 4 storeys only one length per column is required.

An additional benefit of structural hollow sections is that for any given section size the outside dimensions remain the same irrespective of the thickness, unlike H-section columns, where the inside dimensions remain the same and the external dimensions change. This means that even if the column cross sectional area is reduced in higher storeys, the beam lengths can remain the same for the full height of the building, which should result in reduced beam fabrication and erection times and therefore reduced overall costs.

## 2.1 Plain columns

In most countries of the world, the current design codes and standards either are, or will be in the near future, based on a limit states design method. The majority of these use what are described as multiple column curves for the design of compression members. The designation of these curves varies. In Europe, for example, Eurocode 3 (CEN 1992) uses the designations a, b, c, etc. while others use 1, 2, 3, etc. However, in all cases hot finished structural hollow sections have been assigned to the highest curve (i.e. curve a or 1). In Eurocode 3, but not necessarily world-wide, cold formed structural hollow sections, based on the enhanced mechanical properties of the finished cold formed product, have been assigned to the third curve (i.e. curve c or 3). A graph of the buckling curves given in Eurocode 3 is shown in figure 2.1. This can result in either a much higher capacity or a considerable weight saving if a structural hollow section is used instead of an open structural column section. In addition, if columns are subject to moment loading about both axes, structural hollow sections generally have a higher moment of inertia and section modulus about the minor axis than a comparable H-section. The design of structural hollow section compression members is described in much more detail in the CIDECT Design Guide on Structural Stability of Hollow Sections (Rondal et al. 1992).

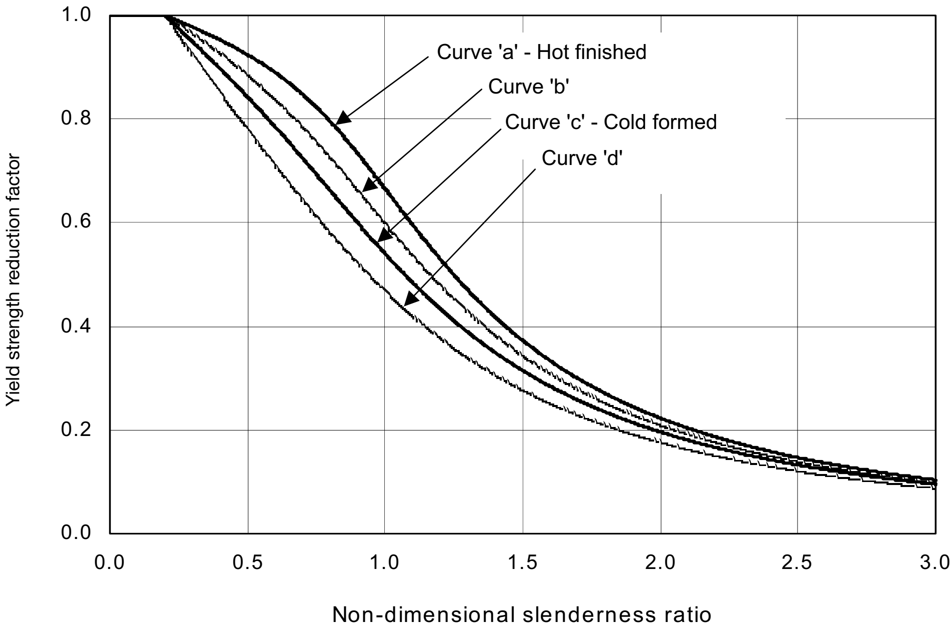



Figure 2.1 – Eurocode 3 column buckling curves

An example is given in figures 2.2 and 2.3. This comparison has been made based on an effective length of 5 m and designing to Eurocode 3, with the requirements of the UK national application document, DD ENV 1993 (BSI 1992). The sections used are a British universal column (UC, H-section), BS 4 (BSI 1993), and two European hot finished structural hollow sections, one square (RHS) and one circular (CHS), EN 10210 (CEN 1997), with all the sections having a nominal yield strength of 275 N/mm$^2$.

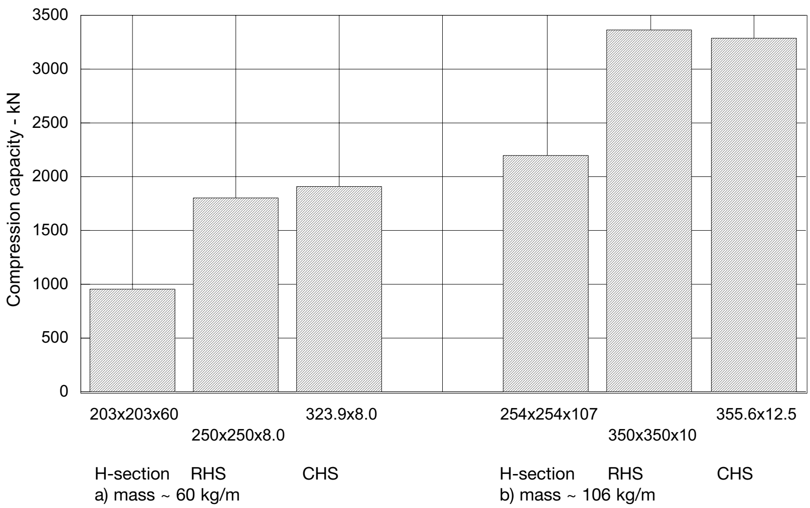



Figure 2.2 – Comparison of compression capacity for sections of equal mass

Based on the concept of equal masses, figure 2.2 shows that for section masses of about 60 kg/m a structural hollow section has a capacity almost twice that of a universal column and for masses of about 106 kg/m the capacity is about 50% higher. The converse of this is shown in figure 2.3, where for equal capacities a mass saving of nearly 40% can be achieved for a capacity of about 1000 kN and a saving of between 30% and 35% on a capacity of about 2100 kN.

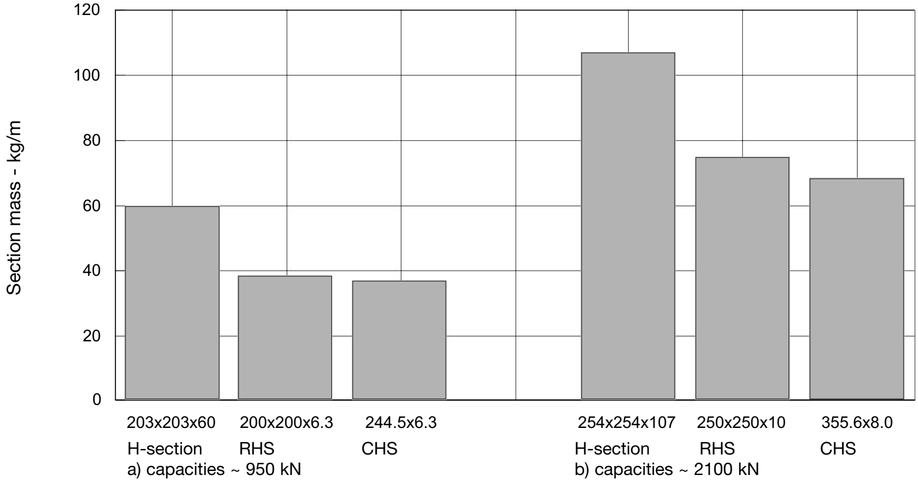



Figure 2.3 – Comparison of section masses for equal compression capacities

## 2.2 Concrete filled columns

Because of the hole in its centre a structural hollow section can be easily filled with concrete, either with or without rebar, to create a steel/concrete composite section, without the need for the temporary shuttering or formwork associated with composite columns made from open sections. Generally concrete with cylinder strengths in the range from 20 N/mm$^2$ to 50 N/mm$^2$ (cube strengths of 25 to 60 N/mm$^2$) has been used. It is possible to use higher strength concrete, but at the present time research work in this area is still underway and no definitive CIDECT design guidance is available. Concrete filled hollow section columns are much more ductile than a plain or reinforced concrete column and connections for beams, etc. can generally be designed and constructed using straight-forward steel design criteria. The ductility and rotation capacity of concrete filled hollow section columns is much better than that of other types of composite column because the concrete is contained within the steel shell and cannot split away even if the ultimate strength of the concrete is reached.

Figure 2.4 gives a comparison of the capacities of the same sections as those shown in figure 2.2.a), but also includes those for the two structural hollow sections when filled with concrete having a cube strength of 40 N/mm$^2$. The capacities of the hollow sections have been increased considerably and are now about 170% and 220% higher than that of the universal column section.

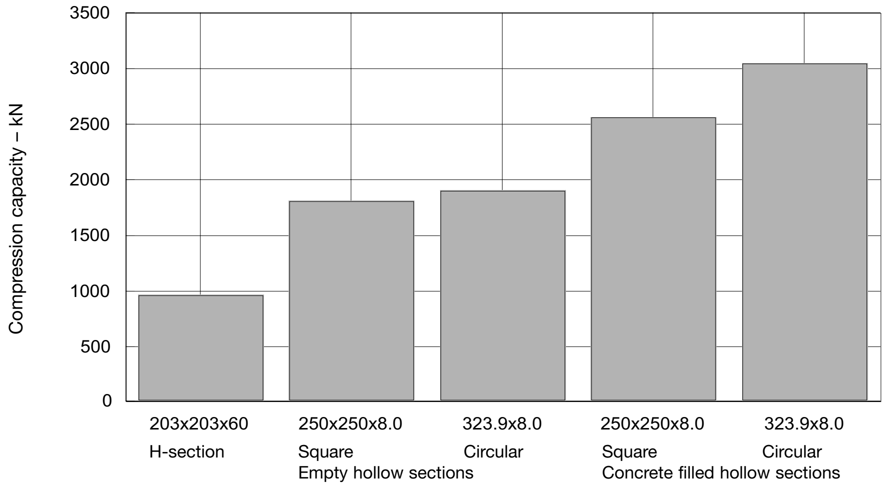



Figure 2.4 – Compression capacities for sections of equal mass (about 60 kg/m) with concrete filling

Most countries, for example Australia, Canada and those in Europe, now use limit states methods for the design of composite steel/concrete columns, although some, notably Japan, still use an allowable stress approach. The design of concrete filled structural hollow sections is fully described in the CIDECT Design Guide on Concrete Filled Hollow Section Columns (Bergmann et al. 1995).

## 2.3 Fire protection

Structural hollow sections are unique among structural steel profiles in that they can be protected from fire damage by using either internal or external methods of protection. As with other structural steel sections, in some cases where the required fire resistance time is quite short, about 15 to 30 minutes, it is possible that no fire protection of any type is needed.

CIDECT Design Guide No. 4 (Twilt et al. 1995) gives detailed information on the design requirements for both external and internal methods of fire protection for structural hollow sections.

### 2.3.1 External fire protection

This type of fire protection can be applied to all types of structural steel profiles. The degree of fire protection depends upon the properties and thickness of the insulation material, the shape factor (heated surface periphery divided by cross sectional area) of the steel profile and the load being carried.

If a sprayed or profile following external protection material is to be used, a structural hollow section will generally require a smaller volume of fire protection material than an equivalent H-section, because of its smaller exposed surface area. For example, consider the structural sections shown in figure 2.3 for a capacity of about 1000 kN. All have a shape factor of about 160 and will, therefore, all require about the same thickness of fire protection material. However, both of the hollow sections have a surface area about 35% less than the H-section, so the volume of fire protection material required will also be about 35% less.

### 2.3.2 Internal fire protection

The hole down the centre of a structural hollow section can be used to great effect as a means of providing the required fire protection to the section and still retain its original external dimensions. Two types of internal fire protection can be used: concrete filling and water filling.

Concrete filling of structural hollow sections has previously been described (section 2.2) to produce a composite steel/concrete column, but it can also be used as a method of fire protection. In a fire the temperature distribution in a concrete filled hollow section is significantly different to that in an empty hollow section. The combination of materials with markedly different thermal conductivities produces extreme transient heating behaviour and high temperature differentials across the section. As a result of these differentials reinforced concrete filled hollow section columns can be designed to have a fire resistance of up to 120 minutes, or more, without any external fire protection. In this situation the basic idea is that the steel plus reinforced concrete are designed to carry the normal factored loads under a no-fire situation, and the reinforced concrete is designed to carry the much lower service loads that need to be taken into account in a fire.

Water filling, using natural circulation, provides a safe and reliable fire protection method for structural hollow section columns provided that the system is self activating in a fire and that the system is also self controlling. In a properly designed system the natural circulation will be activated when the columns are locally heated by a fire. The lower density of the heated water, compared to that of the remaining cooler water, produces pressure differentials which cause natural circulation. As the fire develops this behaviour increases, which in turn increases the cooling effect and the system becomes self-controlling. Several methods of designing a water filled system are described in CIDECT Design Guide No. 4 (Twilt et al. 1995).

# 3 Single sided bolting systems

There are two main methods of making site connections: bolting and welding. Bolting is nearly always the preferred method, unless special circumstances dictate otherwise. Using standard bolts and nuts to make connections to structural hollow sections is difficult because there is normally no access to the inside of the section to tighten them. Unless on-site welding has been adopted, this has usually meant that some form of additional fabrication, and therefore cost, has been necessary to overcome the problem.

Although a number of single sided, or blind, bolting systems have been in existence for a number of years, they have not normally been used in general steel construction mainly because they have been too small in diameter for structural applications. There had, as a result, been very few investigations into their structural strength and behaviour. In recent years, however, a number of blind bolting systems have become available in structural sizes (up to M20 or even M24) and strengths (ISO grade 8.8, ASTM A325, etc.). Blind bolting systems make use of either special types of bolts or inserts or special drilling systems. As the name implies, these can be used when only one side of the connection is accessible, and, therefore, access to both sides is not necessary. This allows, for example, bolted beam to structural hollow section column connection details to be designed in a similar way to a beam to open section column connection.

As these blind bolting systems have become available, CIDECT and others have carried out various research and development projects, in conjunction with the system manufacturers. These projects have been used to determine the requirements for the design of connections to structural hollow section columns incorporating these different systems. Although other systems may be available, these research projects have concentrated on the following systems: the Flowdrill drilling system, the Lindapter HolloBolt insert and the Huck Ultra-Twist bolt, which are described in the following sections of this chapter.

There is no intrinsic reason why these systems cannot be applied to both rectangular and circular hollow section columns. However, direct bolting to rectangular hollow section columns is an accepted procedure, but direct bolting to circular hollow section columns is not so usual because curved saddle plates, instead of flat ones, are required.

The following sections 3.1 to 3.5 describe these methods/systems and their capacities as individual bolts in a structural hollow section. In most connections incorporating a group of bolts loaded in tension the connection capacity will almost always be controlled by the deformation or yielding capacity of the face of the structural hollow section and not that of the individual bolt. The design methods and details for these practical connections are given in section 6.5.2.

## 3.1 Flowdrill drilling system

The Flowdrill system is a patented method for the extrusion of holes using a four lobed tungsten-carbide friction drill. Details of the drilling tools and procedure are available from the manufacturer – Flowdrill b.v. at *www.flowdrill.nl.*

The tungsten-carbide drill bit forms a truncated cone on the far side of the workpiece and a small upset on the near side, which can automatically be removed by a milling cutter incorporated into the drill bit. The hole can then be threaded using a roll (or forging) tap, rather than a cutting tap, to produce a threaded hole, which has an effective thread length of 1.5 to 2.0 times the material thickness. The Flowdrill process is shown schematically in figure 3.1.

The advantages of this system are that the specialist equipment is fabrication shop based, only standard fully threaded bolts are used (no nuts are needed), virtually standard beam and column bolt hole layouts can be used and no specialist equipment is required on site.

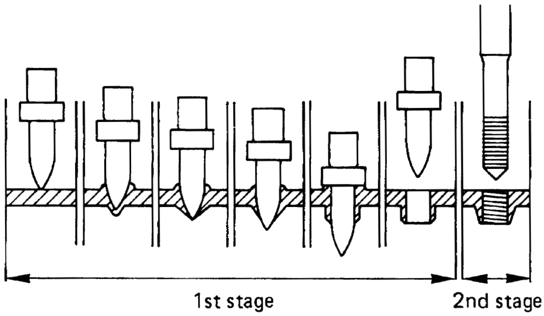



Figure 3.1 – Schematic of the Flowdrill process

The results of a series of tests on individual flowdrilled holes and on connections made using the Flowdrill system (Yeomans 1996a and 1996b) have shown that they are suitable for structural applications. These tests have shown that:

- flowdrilled holes can be produced in both hot finished and cold formed hollow sections from 5.0 to 12.5 mm thick;
- threaded roll tapped holes with M16, M20 and M24 ISO course thread profiles can be made;
- the full tension capacity of grade 8.8 (similar to ASTM A325) bolts can be carried by flowdrilled and roll tapped holes, provided that the RHS thickness is equal to or greater than the minimum thickness shown in Table 3.1 and the RHS has a nominal yield strength in the range 275 to 355 N/mm$^2$;

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

Table 3.1 – Minimum RHS thickness for full grade 8.8 bolt tension capacity

- the shear and bearing capacities of the hole and bolt can be calculated in the normal manner;
- in most applications in which the bolts are loaded in tension, the deformation or yielding of the RHS face will determine the overall connection capacity and not the capacity of each individual bolt. The design criteria for this are given in section 6.5.2.

### 3.2 Lindapter HolloBolt insert

The HolloBolt is a three part pre-assembled unit consisting of a main body, a threaded truncated cone and a standard grade 8.8 bolt and is shown in figure 3.2. A five part system is also available. Details of dimensions, hole tolerances, torque requirements, etc. are available from the insert manufacturer – Lindapter International plc at *www.lindapter.com.*

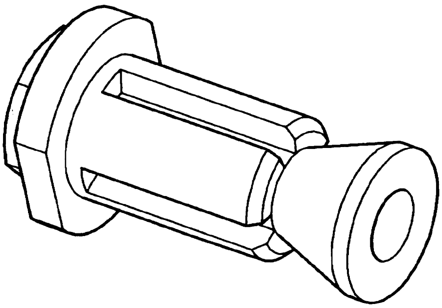


Figure 3.2 – The Lindapter HolloBolt insert

The operating principle of the HolloBolt insert is that once placed in the hole, through the materials being joined, the tightening of the bolt draws the tapered cone into the legs of the body. As this happens the legs of the body are splayed out and provide the mechanical interlock necessary to prevent the insert being pulled out. The tension and shear capacities of the insert are at least equal to that of the corresponding grade 8.8 bolt, but it is suggested that the grade 8.8 bolt capacities should be used for design purposes (Occhi 1996).

As with the Flowdrill system in connections in which the bolt, or bolts, are loaded in tension the RHS face deformation (or yielding) capacity will usually be the determining factor, and not that of the individual insert (Yeomans 1998) unless the hollow section face is reinforced. The design criteria for this are given in section 6.5.2.

### 3.3 Huck Ultra-Twist bolt

The Ultra-Twist bolt is a pre-assembled unit manufactured by Huck International Inc. at *www.huck.com/industrial*, from whom details of dimensions, tolerances, torque requirements etc. are available. An exploded view of the bolt is shown in figure 3.3. The Ultra-Twist bolt is installed using an electric bolting wrench in holes 2 mm larger than the outside diameter of the bolts, which provides conventional clearances for fit-up.

These bolts have tensile strengths, installed tensions and shear capacities meeting the requirements of ASTM A325 bolts (equivalent to ISO grade 8.8, Sadri 1994 and Korol et al. 1993), so that the tension, shear and bearing capacities of individual fasteners can be calculated in the normal way. However, as stated previously, in applications where a group of bolts are used in tension the deformation or yielding of the hollow section face will nearly always be the determining factor in design (see section 6.5.2) unless the hollow section face is reinforced in some way.

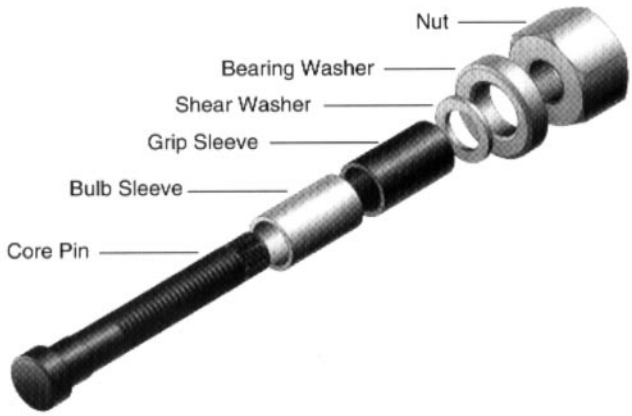



Figure 3.3 – Exploded view of Huck Ultra-Twist bolt

**3.4 Stud welding**

Threaded studs welded to structural hollow section columns can also be used to produce connections. Various types of studs are available, from many manufacturers, who should be consulted concerning requirements for their installation and their capacities.

Some research has been carried out (Maquoi et al. 1985) to investigate welding parameters and connection capacities. Provided that the weld is adequate and the studs are certified in a similar manner to bolts, the capacity of individual studs can be based on normal bolt and nut design methods, but with additional checks for punching shear and tear out of the hollow section. If studs are to be welded onto the hollow sections in the fabrication workshop, then special care is needed to prevent damage during transit to site.

Again, in connections containing a group of studs in tension deformation or yielding of the face of the hollow section will nearly always be the determining criterion (see chapters 5, 6 and 8) unless the face of the hollow section is reinforced.

**3.5 Other methods**

There are several other methods available for making bolted connections, which can be fixed from one side only. Two of these are briefly described below.

The first method is simply drilling and tapping the hollow section, but this generally needs a wall thickness of 16 mm or more to generate enough pull out capacity.

Another method (Kato 1988) is to drill holes in the hollow section large enough for a nut of the required size to be inserted and then to weld the nut to the hollow section flush with the outside surface (see figure 3.4).

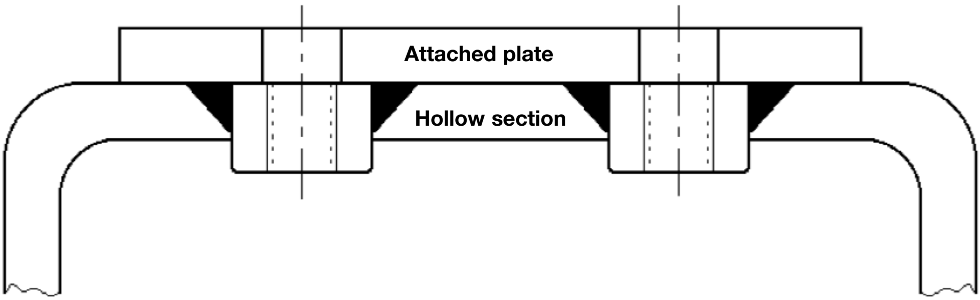



Figure 3.4 – Nuts welded into hollow section wall

# 4 Connection classification

This chapter gives some general background information on the classification of connections. The subsequent chapters 5, 6 and 8 give actual design guidance on connections to hollow section columns for simple shear (pinned), semi-rigid and rigid connections respectively. Chapter 9 contains specific design guidance on connections to concrete filled columns.

## 4.1 Introduction

In the past, most designers have designed beam-to-column connections either as pinned or as rigid. However, in reality, the actual stiffness of a connection will nearly always be somewhere between these two extremes, i.e. the connection will behave in a semi-rigid manner. Also the capacity of an unstiffened connection might be less than that of the connected beam, in which case it is termed *“partial strength”*.

The use of semi-rigid connections may offer a considerable reduction in overall frame costs, because they generally have either no stiffeners or much fewer stiffeners than rigid connections. Cost calculations for semi-rigid frames made of I- or H-beams show reductions in costs of 10 to 20% over rigid frames, depending on the structural arrangement and the sections used. However, it is not only the costs of material and labour for columns, beams and connections that should be taken into consideration, additional effects, such as beam depth or avoidance of temporary bracing, should also be considered.

*Note: in this design guide the terms “joint” and “connection” use the definition given in AISC (1997), and not that in Eurocode 3: Annex J (CEN 1992) which uses them the other way around.*

### 4.1.1 Elastic behaviour

The effect of the connection stiffness on the elastic moment distribution for a beam with a uniformly distributed load is represented in figures 4.1 and 4.2. Figure 4.1 shows the elastic distribution in the beam for the pin end, the fixed end and the semi-rigid end conditions. It can be seen that with semi-rigid connections the elastic moment distribution can be influenced considerably.

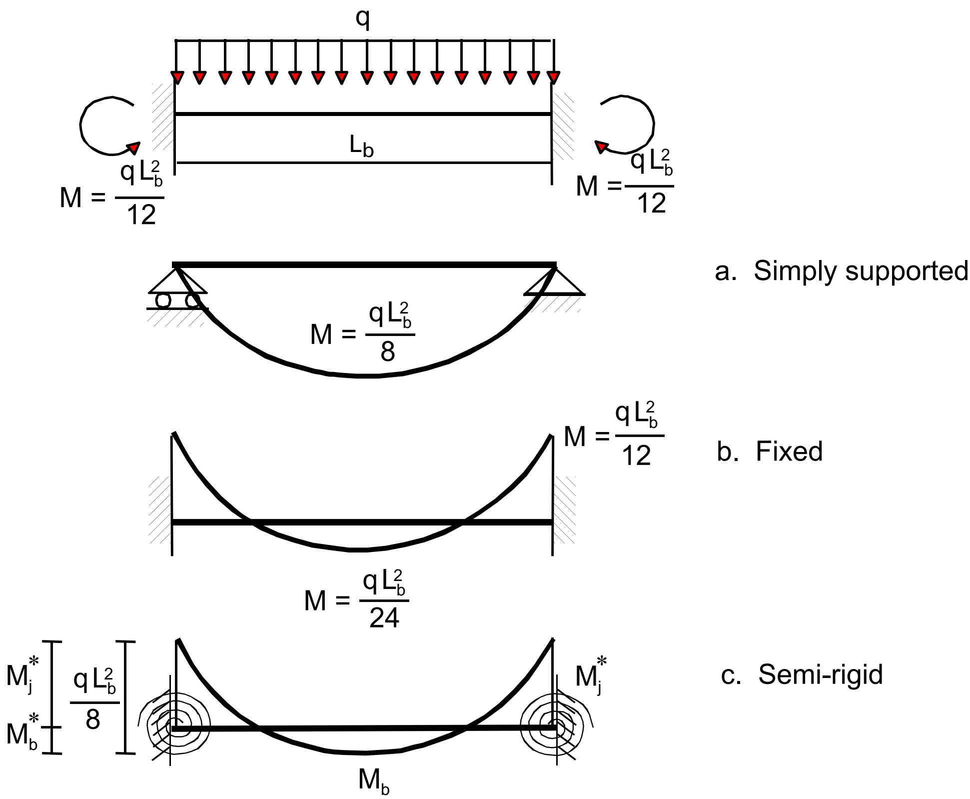



Figure 4.1 – Beam with various end conditions

The joint rotation $\phi_j$ is given by:

$$\phi_j = \frac{q \cdot L_b^3}{24EI_b} - \frac{M_j \cdot L_b}{2EI_b} \qquad \text{4.1}$$

with $M_j = S_j \cdot \phi_j$ and $K_b = 2EI_b / L_b$ ..................... 4.2

$$\text{or } \phi_j = \frac{q \cdot L_b^3}{24EI_b} - \frac{S_j \cdot \phi_j \cdot L_b}{2EI_b} \qquad \text{4.3}$$

$$\text{and } \frac{M_j}{S_j} = \frac{q \cdot L_b^3}{24EI_b} - \frac{M_j \cdot L_b}{2EI_b} \qquad \text{4.4}$$

$$\text{or } M_j = \frac{qL_b^2}{12} \cdot \frac{S_j}{(K_b + S_j)} \qquad \text{4.5}$$

$$M_b = \frac{q \cdot L_b^2}{8} - M_j \qquad \text{4.6}$$

Based on these relationships, in figure 4.2 the elastic moment at the beam centre $M_b$ and the moments at the connections $M_j$ are given for different joint stiffnesses $S_j$.

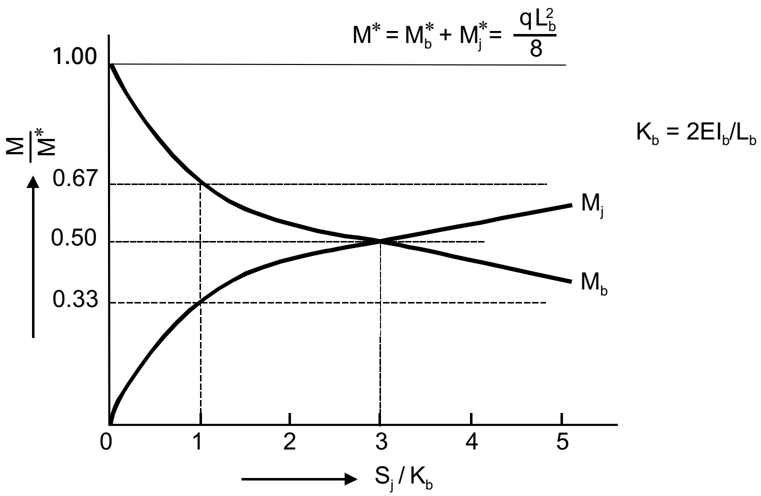



Figure 4.2 – Variation of elastic moment distribution with connection stiffness (Anderson et al. 1997)

### 4.1.2 Plastic behaviour

If a rigid-plastic analysis is used, the moment capacity of the connections is of primary importance, but the rotation capacity is also important. For example, if the stiffness of the connections of the beam in figure 4.1 is very low, the plastic moment capacity of the beam at mid-span $M_{pl}$ may be reached first. As a result the moment capacity of the end connections $M_j$ can only be reached if the beam has sufficient rotation capacity at the location of the plastic hinge. In the case of connections with a very low stiffness this might not be the case, e.g. see connection “e” in figure 4.3.

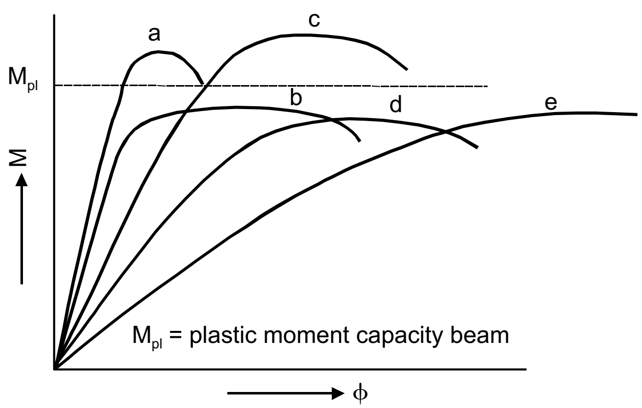



Figure 4.3 – Various M-$\phi$ characteristics

If the stiffness of the connection is high, the (partial) strength capacity of the end connections (e.g. connection "b" in figure 4.3) may be reached first. Now these connections should have sufficient deformation capacity to develop, with increasing load, the plastic moment capacity of the beam at mid-span.

Thus, for a proper analysis of frames with semi-rigid connections, a description of the moment-rotation behaviour is required. Thus, evidence is required regarding:

- stiffness (serviceability and at the ultimate limit state),
- strength (ultimate limit state) and
- rotation capacity.

However, all this information is not yet generally available for tubular beam-to-column connections. Other options are that the stiffness is such that the connections can be classified as (nearly) rigid or (nearly) pinned as discussed in other chapters. For both cases, limits can be given. However, the deflections can only be determined properly if the joint stiffness is available.

$$\delta = \frac{5q \cdot L_b^4}{384EI_b} - \frac{M_j \cdot L_b^2}{8EI_b} \qquad \text{.......... 4.7}$$

Combined with equation 4.5 gives:

$$\delta = \frac{5q\, L_b^4}{384EI_b}\left[1 - \frac{4S_j}{5(K_b + S_j)}\right] \qquad \text{.......... 4.8}$$

Figure 4.4 shows this relationship between the mid-span deflection of the beam of figure 4.1 and the connection stiffness $S_j$.

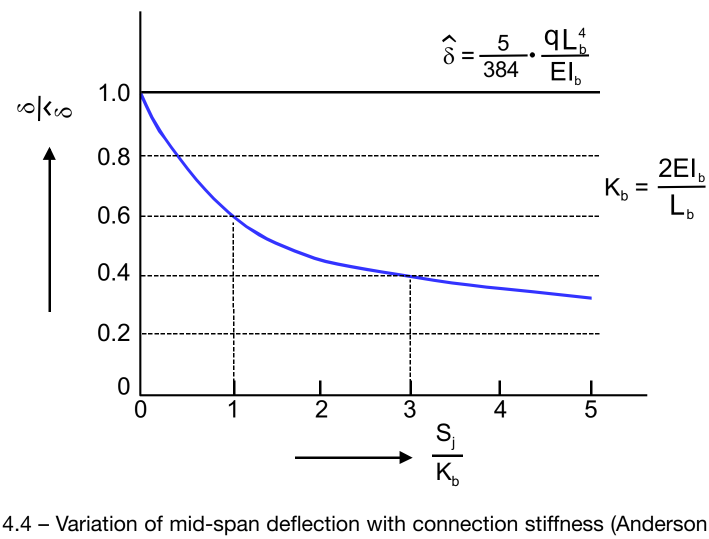



Figure 4.4 – Variation of mid-span deflection with connection stiffness (Anderson et al. 1997)

## 4.2 Semi-rigid connection design according to Eurocode 3

In this section, the analysis method used in Eurocode 3: Annex J (CEN 1992) for semi-rigid connections between I- and H-sections is briefly presented. The method is generally known as the "component method" and is used to determine the strength and stiffness of semi-rigid connections. These connections are principally for moment-resisting connections and can be welded or bolted (with end plates or angle cleats).

*Note: At the time that this design guide was being written, CEN was reorganising and updating Eurocode 3 and it is expected that sometime between 2002 and 2004 Eurocode 3: Annexes J and K will become part of Eurocode 3: Part 1.8: chapters 6 and 7 respectively.*

Connections between hollow sections are dealt with in Eurocode 3: Annex K. This deals primarily with the ultimate strength of axially loaded connections, however, formulae are also given for some types of moment connections between circular or rectangular hollow sections. No information is given for the stiffness.

In Annex J, the types of connections are distinguished as indicated in figure 4.5

| Method of global analysis | Types of connections | | |
|---|---|---|---|
| Elastic | Nominally pinned | Rigid | Semi-rigid |
| Rigid-Plastic | Nominally pinned | Full-strength | Partial-strength |
| Elastic-Plastic | Nominally pinned | Rigid and full-strength | Semi-rigid and partial-strength<br>Semi-rigid and full-strength<br>Rigid and full-strength |
| Type of joint model | Simple | Continuous | Semi-continuous |

Figure 4.5 – Types of connections according to Eurocode 3: Annex J

For an elastic global analysis, the connections are classified according to their stiffness, for a rigid plastic analysis the connections are classified according to their strength and for an elastic-plastic analysis the connections are classified according to both stiffness and strength.

For elastic and elastic-plastic analyses the rotational stiffness of a semi-rigid joint is needed. A simplified method is to use the initial rotational stiffness $S_{j,ini}$ up to $^2/_3$ $M_j^*$ and $S_{j,ini}/\eta$, for larger values as shown in figure 4.6. The value for $\eta$ is between 2 and 3.5, depending on the type of joint. An even more simplified method is to use the stiffness value for $M_j^*$ for all values of $M_j$.

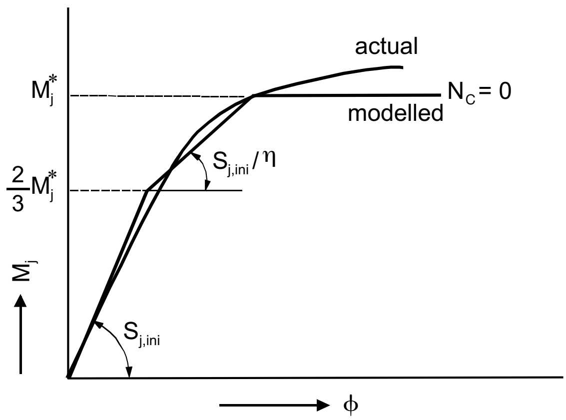



Figure 4.6 – M-ϕ modelling according to Eurocode 3: Annex J

### 4.2.1 Classification of connections

The classification by stiffness is given in figure 4.7. All connections in the zone marked with "semi-rigid" should be classified as semi-rigid. The two other zones may optionally be treated as semi-rigid, if convenient.

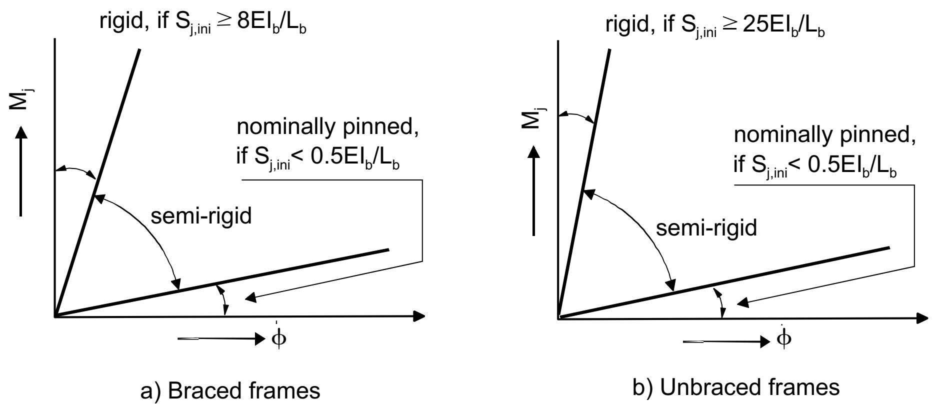



Figure 4.7 – Boundaries for stiffness classification of beam-to-column connections (EC3, Annex J)

The classification by strength is as follows:

Full strength – if the moment design capacity of the connection is such that the plastic moment capacities are first reached in the connecting beam(s) or column(s).

Pinned – if the design moment capacity of the connection does not exceed 25% of the design moment capacity required for a full strength connection.

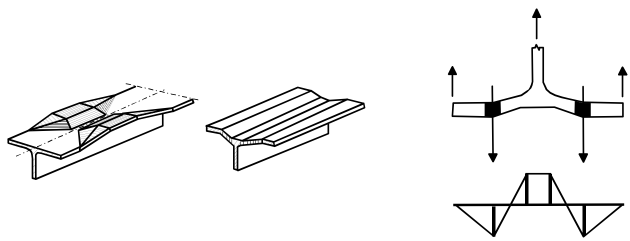
Partial strength – if the connection moment capacities are between the above limits or alternatively for all connection moment design capacities less than the plastic moment capacity of the connecting beam.

### 4.2.2 Moment capacity

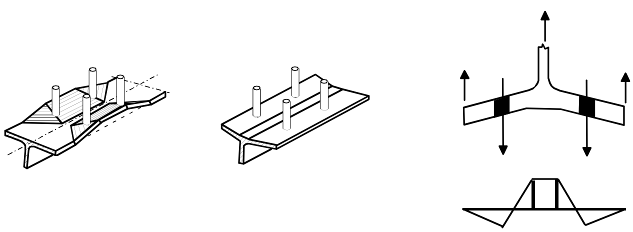
The moment capacity of the connection is based on the strength of all components, which may fail, see figure 4.8. For example, the failure behaviour of an I-beam-to-I-column connection (capacity and stiffness) may be translated to that of an equivalent T-stub length. The combination of all these stiffnesses and capacities result in the behaviour of the connection. Thus for each component in the connection a reference is given to determine the capacity, stiffness and where available the rotation capacity.

Grotmann (1997) analysed the behaviour of some welded and bolted connections between I- or H-section beams and rectangular hollow section columns on the basis of the component method. In principle, he used a methodology comparable to figure 4.8 and similar to that used by Togo (1967) for tubular connections known as the ring model. For RHS columns it is not a ring, but a frame with a certain effective length (see figure 4.9).

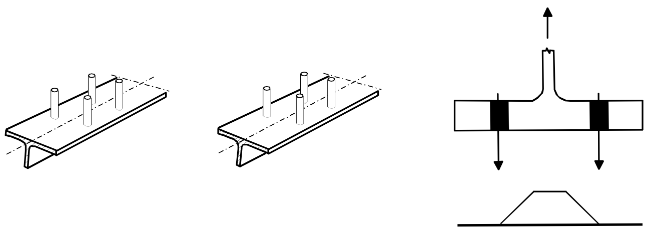
The equivalent effective length can be determined based on a yield line mechanism for the flange to RHS column connection resulting in a similar capacity. In particular cases, he obtained a good agreement with the actual moment rotation curves. However, in other cases large deviations occurred and further evaluation is necessary before this method can be used for the design of hollow section column connections.



Mode 1: Complete yielding of the flange



Mode 2: Bolt failure with yielding of the flange



Mode 3: Bolt failure

Figure 4.8 – Failure modes of actual components and equivalent T-stub flanges for bolted beam-to-column connections (I-beams)

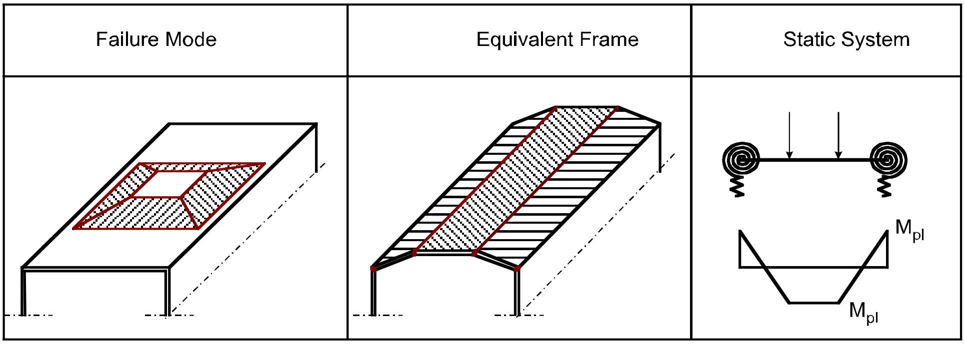



Figure 4.9 – Simplified frame for component behaviour

### 4.2.3 Rotational stiffness

The rotational stiffness of a connection is determined from the flexibilities of its basic components. An advanced model (Jaspart 1997) is shown in figure 4.10. However, in Eurocode 3: Annex J only linear springs for each component of connections between open sections are given.

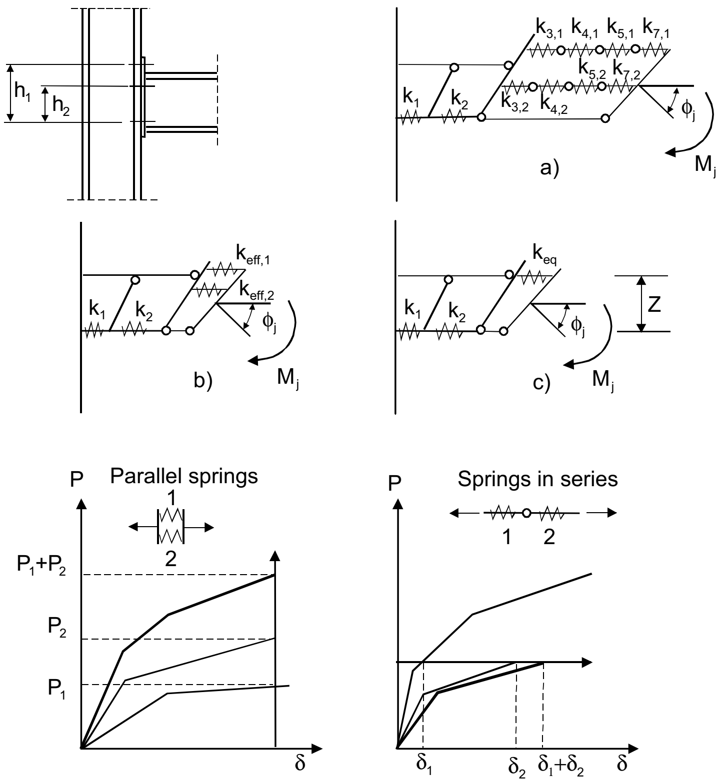



Figure 4.10 – Examples of spring models used for a bolted beam-to-column connection with an end plate (Jaspart 1997)

In figure 4.10 the springs signify the behaviour of the following components:

$k_1$: column web (compression)
$k_2$: beam flange (compression)
$k_3$: column web (tension)
$k_4$: column flange (tension)
$k_5$: bolt (tension)
$k_7$: flange plate (bending)

The springs $k_1$ to $k_2$ and $k_3$ to $k_7$ work in series whereas the results of $k_{3,1}$ to $k_{7,1}$ with $k_{3,2}$ to $k_{7,2}$ work in parallel.

For springs in series the deformations are added for the same force whereas for parallel springs the forces are added for the same deformation, as indicated in figure 4.10.

### 4.2.4 Rotational capacity

With regard to the available rotation capacity, some indications are given in Eurocode 3: Annex J, however, research is still underway to determine the required deformation capacity for various systems and to determine the available rotation capacity for various joint configurations (Boender et al. 1996).

### 4.2.5 Conceptual design

In the conceptual design of steel frames the actual dimensions of the connections are not known, but assumptions have to be made for the stiffness and strength of the connections. Steenhuis et al. (1994, 1996) and Jaspart (1997) give guidance for stiffness values for various types of semi-rigid connections between open sections. Later on, the actual stiffness values, $S_{j,act}$, have to be checked and should not deviate by more than the following limits (Steenhuis et al. 1994):

for braced frames:

$$\text{for } S_{j,\,app} < \frac{8EI_b}{L_b} \text{ then } \frac{8S_{j,\,app}\,EI_b}{10EI_b + S_{j,\,app}\,L_b} \leq S_{j,\,act} \leq \frac{10S_{j,\,app}\,EI_b}{8EI_b - S_{j,\,app}\,L_b} \qquad \text{......................... 4.9}$$

for unbraced frames:

$$\text{for } S_{j,\,app} < \frac{25EI_b}{L_b} \text{ then } \frac{24S_{j,\,app}\,EI_b}{30EI_b + S_{j,\,app}\,L_b} \leq S_{j,\,act} \leq \frac{30S_{j,\,app}\,EI_b}{24EI_b - S_{j,\,app}\,L_b} \qquad \text{......................... 4.10}$$

If these limitations are satisfied the load capacity of the frame between that with the applied value ($S_{j,app}$) in the calculations and that with the actual stiffness ($S_{j,act}$) will differ by less than 5%.

# 5 Simple shear connections

## 5.1 Introduction

The ends of members with simple shear connections are assumed to be rotationally unrestrained or free to rotate under load. However, simple shear connections do actually possess some rotational restraint. This is discussed further in chapter 4, which gives the rigid, semi-rigid, and pinned joint classifications based on initial joint rotational stiffness according to Eurocode 3 (CEN 1992). This small amount of moment resistance is usually neglected and the joint is idealised to be completely flexible. Hence, simple shear connections are sized only for the end reaction or shear force from the supported beam. However, simple shear connections must still provide flexibility to accommodate the required end rotations of the supported beam. To accomplish this, inelastic action at the specified (unfactored) load levels in the joint is permitted. Thus, for most simple framing systems the connection moment-rotation response (as shown in figure 4.7) remains linear only in the initial stages of loading.

In some countries the building codes have a structural integrity/robustness requirement that all shear connections be capable of carrying, in addition to the vertical shear load parallel to the column, a further nominal horizontal load acting normal to the column. This is to ensure that if accidental horizontal forces are present in a building, then the connections will remain intact and the building will not collapse. Assuming that the nominal horizontal load on the connection occurs under the same load combination that produces the maximum beam end reaction (shear) on the connection, then the resultant force on the connection will be inclined to the axis of the column. This is similar to the case of an inclined brace member connected to a column, which is covered in chapter 10.

When members are designed with simple shear connections, provision must be made to stabilise the frame for gravity loads and also to resist lateral loads. Many of the familiar simple (shear) connections that are used to connect I-section beams to I-section columns can be used with hollow section columns. These include single and double angles (cleats), unstiffened and stiffened seats, single shear plates (also termed “shear tabs” or “fin plates”) and tee connections (Packer and Henderson 1997, AISC 1997, SCI 1991). One additional connection type that is unique to hollow section connections is the through-plate. One should note that this alternative is seldom required for structural reasons and it incurs a significant cost penalty when a single shear plate connection would otherwise suffice. Variations in attachments are more limited with hollow section columns since the connecting element will typically be shop-welded to the hollow section column and bolted to the supported beam. Except for seated connections, the bolting will be to the web of an I-section (or other open profile) beam. Beam coping is generally not required except for bottom-flange copes (removal of the bottom flanges) with double angle connections, because of practical erection considerations (the beam is usually lowered vertically down with its web between the angles.)

Simple beam-to-column connections could also be made to RHS columns by direct bolting to the column wall. With such connections a beam, typically with a shop welded flush end-plate, would be site bolted to a column using “blind bolts” or regular bolts in flowdrilled holes. These fastening methods are described in chapter 3. One advantage of bolting directly to the RHS column is that there are no protruding attachments to the column, thereby requiring less care during transportation and erection. Testing of simple shear beam-to-RHS column connections using single-sided bolting systems has not identified any special failure modes, so these shear connections can be designed using normal practice (Yeomans 1996, Korol et al. 1993, Sherman 1995, France et al. 1999).

## 5.2 Limit states for simple shear connections

There are a number of limit states associated with the bolts, connecting elements (plates, angles, tees), welds and beam webs that are applicable to the design of all shear connections, whether using hollow or open section columns, and the applicable national or regional structural steelwork specifications should be followed for such design criteria. In addition to these limit states, the following potential failure modes should also be checked for shear connections to hollow section columns (AISC 1997):

(i) shear yield strength of the tube wall adjacent to a weld (for all connection types);
(ii) punching shear through the tube wall (for single shear plate connections only);
(iii) plastification of the tube wall, using a yield line mechanism (for stiffened seat connections to RHS columns only).

Elaboration of the above three failure modes is provided in the following discussion on various connection types. Mode (iii) above, representing a flexural failure of the hollow section face, is not a limit state (with the one exception as noted) because the end rotation of a beam supported at both ends is limited and is insufficient to develop a yield line mechanism in the column connecting face (AISC 1997). However, Sputo and Ellifritt (1991) performed tests on stiffened seat connections to the webs of I-section columns and found that a yield line mechanism *may* be an applicable limit state. Since this situation (connecting to a plate element that is supported for a long length on two opposite edges) is similar to that for a RHS column face, the yield line mechanism is considered a possible limit state for stiffened seat connections (AISC 1997). The inclusion of this failure mode acknowledges that all “simple shear connections” will still have *some* end rotation of the connected beam.

## 5.3 Single shear plate connections (shear tabs, fin plates)

When selecting the type of connection, one should bear in mind that RHS columns may likely have a smaller width than the equivalent I-section column flange or web, which thereby restricts the width of a connecting angle leg or flange of a tee. Moreover, the factored shear load to be transmitted at a connection is often low so a single shear plate connection, shown in figure 5.1, is frequently a logical and economic choice. One of the earliest experimental studies on simple shear connections to RHS columns was done by White and Fang (1966), but thereafter the topic received little attention for over 20 years. Sherman and Ales (1991) and Sherman (1995) have investigated a large number of simple framing connections between I-section beams and RHS columns, in which the load imposed on the column was predominantly shear. In all, nine different types of simple framing connections to RHS columns were considered structurally and with a relative cost review. The latter showed that the single shear plate and single angle connections were the cheapest. Double angle and fillet-welded tee connections were more expensive, while through-plate and flare-bevel-welded tee connections were among the most expensive (Sherman 1995).

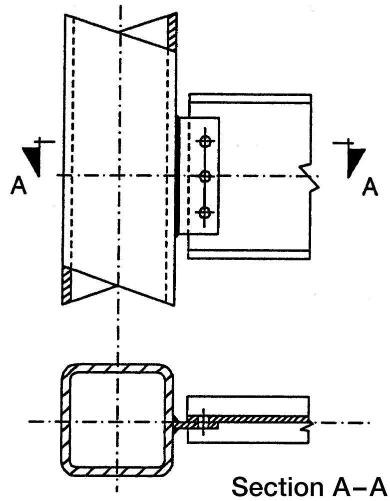



Figure 5.1 – Single shear plate connection

Single shear plate connection tests were performed with bolts both snug tight and fully pretensioned. The connections with snug tight bolts had the same ultimate capacities and eccentricities as those with pretensioned bolts. (The eccentricity is the distance from the column face to the point of contraflexure in the beam, or the distance from the column face to the line of action of the beam shear reaction.) However, at working loads pretensioned bolts produced larger eccentricities (to the contraflexure point/inflection point where negative moment changed to positive moment) and hence larger end moments in the columns. It was found that the local distortion that does occur in the RHS wall (for connections on one or both sides of the RHS) has negligible influence on the column resistance provided the RHS is not thin-walled or slender. The definition of "slender" used herein is a width-to-thickness ratio for the *flat* of the RHS connecting face exceeding $1.4\sqrt{(E/f_{c,y})}$ (AISC 1997). An extrapolation of this provision was also made by AISC (1997) for CHS columns, wherein single shear plate connections were permitted for non-slender CHS under axial load, which was defined by $d_c/t_c \leq 0.114E/f_{c,y}$. Thus, providing the column wall is not slender (according to the above limits), which is normally the case for most practical columns, there is no advantage to using through-plates (Sherman 1995). A possible failure mode for the single shear plate connection is warping of the shear plate due to twisting of the beam. It is therefore recommended that long unbraced beams attached by shear plate connections be provided with lateral support in the vicinity of the connection. Alternatively, avoid shear plate connections in such situations.

Over a wide range of connections tested by Sherman (1995, 1996), only one limit state was identified for the RHS column. This was a punching shear failure related to end rotation of the beam when a thick shear plate was joined to a relatively thin-walled RHS. Two connections failed when the shear plate pulled out from the RHS wall at the top of the plate around the perimeter of the welds. A simple criterion to avoid this failure mode is to ensure that the tension resistance of the plate under axial load (per unit plate length) is less than the shear resistance of the RHS wall along two planes (per unit plate length). Thus (Sherman 1995, AISC 1997),

$$\phi_1\, f_{,p,y}\, t_p \cdot \text{(unit length)} < 2\, \phi_2\, (0.60 f_{c,u})\, t_c \cdot \text{(unit length)} \quad \text{.........} 5.1$$

In the above inequality the left hand side, the tensile strength of the plate, is multiplied (for limit states design) by a resistance factor of $\phi_1 = 0.9$ for yielding. The right hand side of the inequality, the shear strength of the RHS wall, (for which the ultimate shear stress is taken to be 0.6 of the ultimate tensile stress), is multiplied by a resistance factor of $\phi_2 = 0.75$ for punching shear failure (AISC 1997).

Hence $t_p < (f_{c,u}/f_{p,y})\, t_c$ ..........................................................................................5.2

### 5.3.1 Connection to RHS column design example

The following example demonstrates all the typical limit states that need to be checked for a simple I-section beam shear plate connection to a RHS column, along with the unique criterion given by equation 5.2. To do this, it is necessary to conform to a particular limit states structural steel specification and the Canadian standard CAN/CSA-S16.1-94 (CSA 1994) is used in this instance.

*Connect a W410 x 39 Grade 350W beam via a single shear plate to a HSS 203 x 203 x 8.0 Grade 350W Class C column, to develop the capacity of the beam in shear.* (An I-section beam approximately 410 mm deep and weighing 39 kg/metre, with a yield stress of 350 N/mm$^2$, is to be joined to a cold-formed square RHS measuring 203 mm x 203 mm x 8 mm, also with a yield stress $f_{c,y}$ = 350 N/mm$^2$ and minimum ultimate stress $f_{c,u}$ = 450 N/mm$^2$.)

*Shear capacity of beam:*

$V^*$ = 484 kN = required shear capacity of connection CSA Specification

*Shear plate thickness:*

Slenderness of the <u>flat</u> RHS face $= (b_c - 4t_c)/t_c$

$= (203 - 4(7.95))/7.95$

$= 21.5 < 1.4\sqrt{(E/f_{c,y})} = 33.5$

Hence the RHS is not "slender" and equation 5.2 is applicable. Use Grade 300W plate with $f_{p,y}$ = 300 N/mm$^2$ and $f_{p,u}$ = 450 N/mm$^2$.

$t_p < (f_{c,u}/f_{p,y})\, t_c = (450/300)7.95 = 11.93$ mm eqn. 5.2

So choose 10 mm thick plate.

*Bolts required:*

Exclude the bolt threads from the shear plane, the bolts are in single shear, so try 4 M22 ASTM A325 bolts in punched holes.

Total bolt shear resistance, $V_b^* = 4 \cdot (127) = 508$ kN > 484 kN.

With bolts in punched holes, the effective hole diameter = bolt diameter + 4 mm = 26 mm (24 mm punch for 22 mm diameter holes + 2 mm allowance for damage to the edge of the hole caused by punching).

*Bearing resistance:*

Both beam web (thickness = 6.4 mm) and shear plate ($t_p$ = 10 mm) have steels with an ultimate stress of 450 $N/mm^2$, so bearing will be critical on the thinner material (beam web).

$B^* = 3\ \phi_3\ t_{b,w}\ d_b\ n\ f_{b,w,u}$ CSA Specification

$= 3(0.67)(6.4)(22)(4)(0.450) = 509$ kN > 484 kN.

In the above a resistance factor of $\phi_3 = 0.67$ has been used for failure associated with a connector (equivalent to a partial safety factor of 1.5).

*Plate length:*

The clear distance between the beam root fillets for the W410 x 39 section = 348 mm CISC Handbook

So choose a plate length, $L_p$ = 340 mm.

*Shear yield strength of tube wall adjacent to welds:*

$V^* = 2\ \phi_1\ L_p\ t_c\ (0.6 f_{c,y})$ AISC Manual

$= 2(0.9)(340)(7.95)(0.6)(0.350) = 1{,}022$ kN > 484 kN.

[The nominal RHS wall thickness is 7.95 mm.]

*Net section fractures of shear plate:*

The four bolts will be arranged in one bolt line, similar to the connection shown in figure 5.2. All possible failure paths should, in general, be checked. After laying out the bolts as shown in figure 5.2, two possible failure paths as illustrated in that figure will be checked.

For figure 5.2(a):

$V^* = 0.85\ \phi_1\ A_e\ f_{p,u}$ CSA Specification

$= 0.85(0.9)(340 - 4(26))(10)(0.6)(0.450)$

$= 487$ kN > 484 kN.

For figure 5.2(b):

$V^* = 0.85\ \phi_1\ A_e\ f_{p,u}$ CSA Specification

$= 0.85(0.9)[3(70-26)(0.6) + (65-13)(0.6) + (65-13)](10)(0.450)$

$= 559$ kN > 484 kN.

This is clearly less critical than the failure path in figure 5.2(a) because the length of the failure line is still the same but one part is now in tension rather than shear.

*Net section fractures of beam web:*

Non-critical unless the beam is coped.

*Gross section yielding of shear plate:*

$V^* = \phi_1 A_g f_{p,y}$ CSA Specification

$= 0.9(340)(10)(0.300) = 918$ kN > 484 kN.

*Fillet welds:*

A multipurpose electrode is chosen with an ultimate strength of 480 N/mm$^2$. By welding along the full length of the plate, on both sides, a weld shear resistance of 0.762 kN/mm is provided by a 5 mm weld (CISC Handbook).

Hence, $V^* = 2(340)(0.762) = 518$ kN > 484 kN.

So choose a fillet weld (leg) size of 5 mm. Generally, this weld would be carried all around the plate. This design procedure has neglected the bending moment on the weld caused by the eccentricity of the line of action of the shear force from the RHS face, as this bending moment is small.

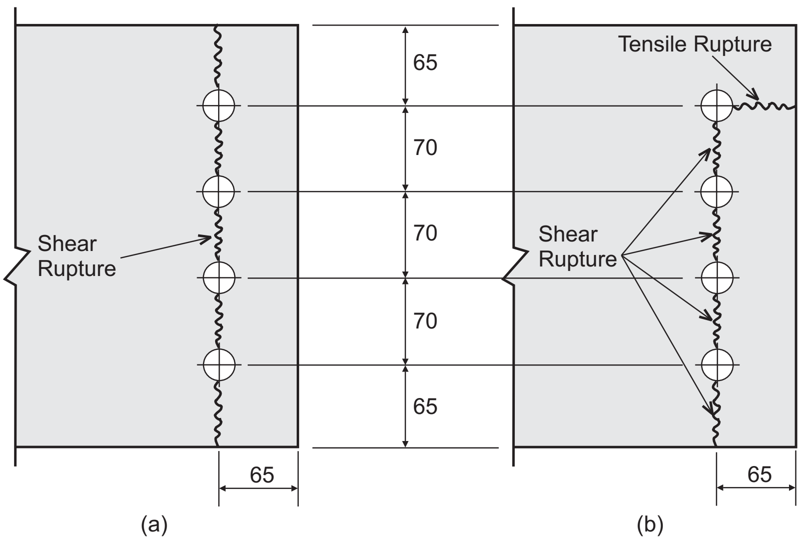



Figure 5.2 – Two possible failure paths for net section fracture in shear plate

### 5.3.2 Connections to CHS columns

This is a popular form of connection (see figures 5.3 and 5.4) because connecting elements do not need to be rounded or saddle-cut. Instead, the vertical shear plate can just be fillet welded all around to the CHS column face. As noted in section 5.3, this type of connection would be permitted for CHS columns that are not "slender"; i.e. $d_c/t_c \leq 0.114E/f_{c,y}$. Aside from this provision, the design procedure would be the same as described in section 5.3.1 for a connection to a RHS column.

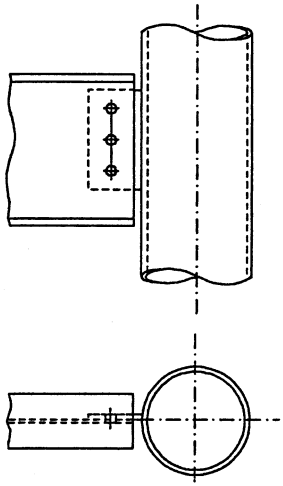


Figure 5.3 – Single shear plate connection to CHS column

Figure 5.4 – Shear plate connection to CHS column

In figures 5.1 and 5.3 the connections are detailed such that the single shear plane of the bolted connection aligns with the centre line of the column. Although this is a common practice, an alternative might be to align the centre line of the beam with the centre line of the column. It is believed that the capacity of the connection will be practically identical with either detailing arrangement.

### 5.3.3 Single shear plate connections to RHS column corner

A variation on the connection shown in figure 5.1 can be made if the plate is connected to the corner of the RHS column, as shown in figure 5.5. The plate is then connected to a much stiffer part of the column cross-section, which thereby avoids any consideration of the RHS wall slenderness as described in section 5.3.

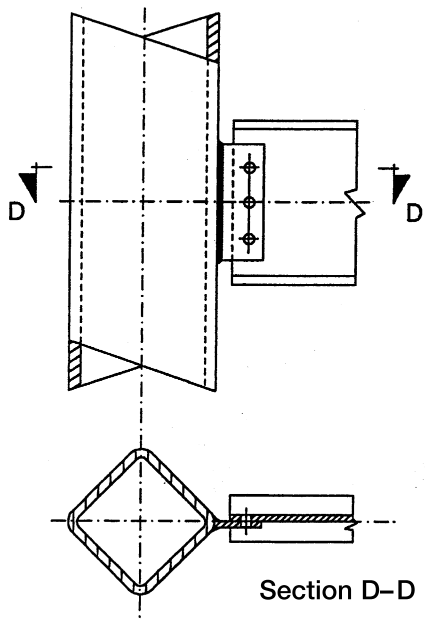



Figure 5.5 – Single shear plate connection to corner of rectangular column

Such connections have been tested by White and Fang (1966) and no special failure limit states have been noted. However, it should be emphasized that if cold-formed RHS columns are used caution should be exercised if heavy welding is planned, as the tube material will have a lower ductility in the corners.

## 5.4 "Through-plate" connections

With the through-plate connection shown in figure 5.6, two opposite faces of the column (either RHS or CHS) are slotted so that the single plate can be passed completely through the hollow section column. The plate is then welded to both faces of the RHS or CHS column.

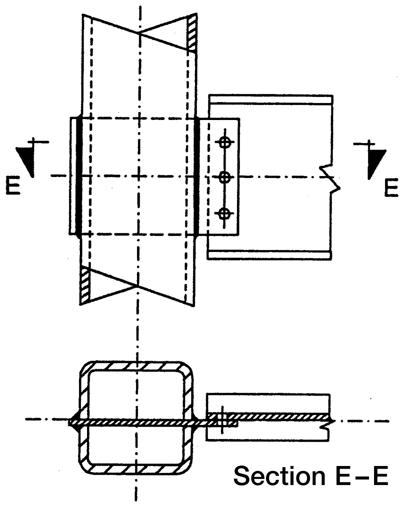



Figure 5.6 – Through-plate connection

The plate does act as reinforcement to the tube face, so this type of connection is preferable to the single shear plate connection if a single plate is still preferred and the column is a "slender" section (see section 5.3). However, the through-plate connection is considerably more expensive than the single shear plate connection, so the latter should be used if it suffices.

When a connection is made on both sides of the column, by using a long or extended through-plate, the portion of the plate inside the hollow section is subject to a uniform bending moment. For long connections this part of the plate may be liable to buckle in a lateral-torsional mode prior to yielding, unless the depth of the column is small (AISC 1997).

## 5.5 End plate connections

A flexible end plate connection, generally with a plate thickness of only 8 or 10 mm, can be partial depth and welded only to the beam web, to achieve a simple or pinned joint. Tests on such connections to RHS columns, using flowdrill connectors, by France et al. (1999, 1999a) have shown that these connections meet the EC3 criterion for pinned joints (see figure 4.7). A common practice has been to use a full depth end plate and to weld this both to the beam web and flanges, making what is commonly called a flush end plate connection, but tests have confirmed that this joint type is semi-rigid by the EC3 criterion. Semi-rigid joints are discussed in chapter 6. France et al. (1999, 1999a) found that the end plate depth, end plate width, end plate thickness, bolt locations and column wall thickness all affected the joint stiffness and strength, as may be expected, but no special connection limit states were observed in their tests beyond those for conventional bolted shear connections. Several connections were tested for the influence of column compression load, with all the RHS columns being "non-slender" according to the limit given in section 5.3. For RHS in this category, column axial stresses of up to 50% of yield had little influence on the behaviour as a simple shear connection. Sherman's (1995) tests on connections with web end plates, bolted to RHS columns with flowdrilled connectors, also confirm these recommendations.

A disadvantage of the end plate connection is that it will require site-bolting to the column using a single-sided (or "blind") bolting system (see Chapter 3). This type of connection – like all the following connection types presented – is also only suitable for RHS columns, not CHS columns.

### 5.6 Tee connections

With this connection, shown in figure 5.7, the flange of the tee is shop-welded to the RHS column and the web of the tee is site-bolted to the beam web. Sherman (1995) has performed tests on these connections with the tee flange narrower than the RHS, with vertical fillet welds, and with the tee flange wider than the RHS, with flare-bevel groove welds to the tube corners. Both details performed well but fillet welding to the flat of the RHS is a more economical alternative.

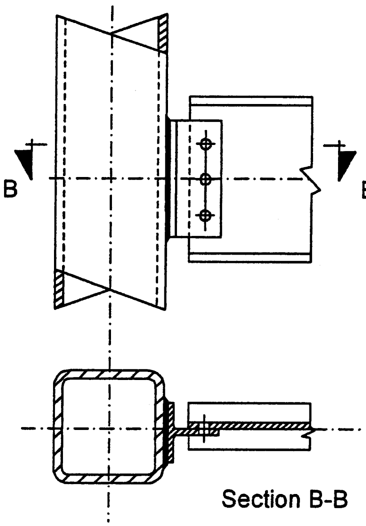



Figure 5.7 – Tee connection

White and Fang (1966) originally proposed that the width to thickness ratio of the tee flange be ≥ 10 in order to provide desired flexibility. Subsequent research by Astaneh and Nader (1990) on tee connections to heavy I-section columns concluded that a tee flange width to thickness ratio ≥ 13 provides sufficient flexibility for the joints to be considered as simple (or pin-jointed). This has since been verified by shear tests on tee connections to RHS columns by Dawe and Mehendale (1995). There is little difference in capacity, whether the tee is centred or offset (to allow the beam to be on the column centreline).

AISC (1997) recommends that, in order to ensure rotational ductility, the tee web (or stem) has a thickness $\leq d_b/2 + 2$ mm. This same criterion could also be applied to a single shear plate or through-plate. As noted in section 5.2, the only limit state unique to the RHS wall to be checked is the shear yield strength of the tube wall adjacent to the vertical welds (assuming the tee flange is welded to the flat of the RHS).

## 5.7 Single and double angle connections

A single angle connection (or angle cleat), see figure 5.8, is made with an angle on one side of the beam web with the angle shop-welded to the RHS column. An L-shaped weld is recommended to provide adequate joint flexibility, with welding along the angle toe and across the bottom of the angle, plus just a small weld return at the top of the angle (see figure 5.9).

Figure 5.8 – Single angle connections to RHS column

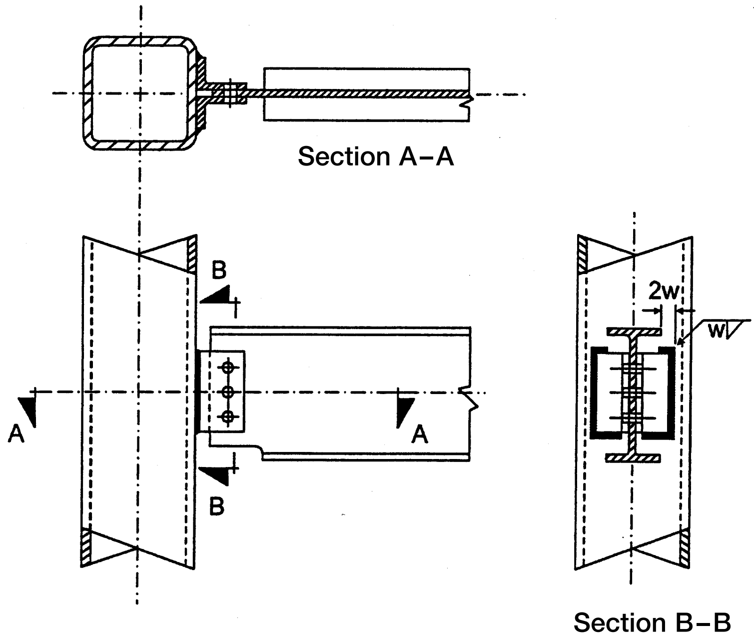



Figure 5.9 – Double angle connection

Welding across the entire top of the angle should be avoided as it would inhibit flexibility (AISC 1997). A 100 mm x 75 mm angle is often selected, with the 75 mm leg welded to the RHS. A minimum angle thickness of 10 mm (for M20 and M22 bolts) or 12 mm (for M24 bolts) is also recommended by AISC (1997). If fillet-welding the angle toe to the flat of the RHS is desired, and the centre of the beam web is to be kept in line with the centre of the RHS, then columns with a connecting face dimension of 200 mm or greater will typically be needed. Alternatively, single angles can be welded to narrow RHS with a flare-bevel groove weld. Assuming the former (fillet) welding procedure is used, the only limit state unique to the RHS wall to be checked is the shear yield strength of the tube wall adjacent to the vertical weld.

A double angle connection (or double angle cleat), as shown in figure 5.9, is one of the most traditional simple shear connections. Pairs of angles are shop-welded along the angle toes, with a small weld return at the top of the angle (see figure 5.9), then field-bolted to the beam web. This connection is sensitive to shop fabrication tolerances, and the two angles may need to be pried apart to allow entry of the beam web on site. It is prudent to cope the bottom of the beam (see figure 5.9) so that erectors can place the beam by lowering it between the angles from above. If the beam is coped, block shear rupture failure of the beam web (a tearout of the beam web, with the failure path passing through the bolt holes) should be checked. Double angle connections provide the strength of bolts in double shear combined with good flexibility and, being symmetrical, the connection avoids any lateral torsion. Fabricators can prepare standard detail angles from stock, rather than prepare special components such as tees, and many steel design handbooks will give standard "pre-engineered connection designs" for this connection type. Sherman (1995) has verified the adequacy of double angle connections to RHS columns and, assuming the angle toes are welded to the flat of the RHS, the only limit state unique to the RHS wall to be checked is the shear yield strength of the tube wall adjacent to the vertical welds (AISC 1997).

## 5.8 Unstiffened seat connections

An unstiffened seated connection is made with a seat angle and a top angle, as illustrated in figure 5.10.

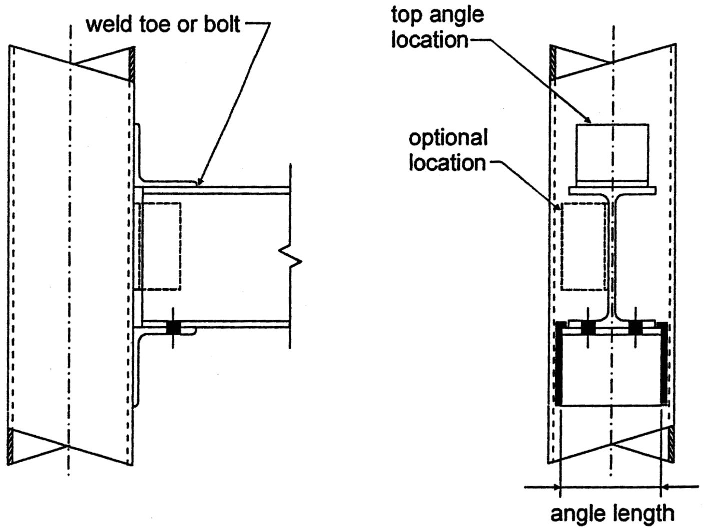



Figure 5.10 – Unstiffened seat connection

Seated connections are common for connections with light loads and for applications such as open web steel joints. While the seat is assumed to carry the entire end reaction of the supported beam, the top angle (typically 100 mm x 100 mm x 100 mm long) must be placed as shown, or in the alternative side location, for satisfactory performance and stability. To provide adequate flexibility for the connection, only the toe of the top angle is welded to the RHS. The thickness of the top angle ought to be 6 mm or greater to accommodate the minimum size fillet weld to the RHS or beam flange. Even if there is no calculated horizontal shear force transfer between the beam flanges and the seat angles, two M20 Grade 8.8 (or ASTM A325) bolts are recommended for the bottom seat angle. Two bolts may also be used to connect the top angle to the beam flange, or a fillet weld may be used across the toe of the top angle. Again, the only limit state unique to the RHS wall to be checked is the shear yield strength of the tube wall adjacent to the two vertical welds to the lower seat angle (AISC 1997).

## 5.9 Stiffened seat connections

A stiffened seated connection is made in the same manner as an unstiffened seated connection except the seat angle is replaced by a tee (either a structural tee or comprised of two plates), wherein the web (or stem) of the tee is vertical and the flange of the tee (on which the beam sits) is horizontal (see figure 5.11).

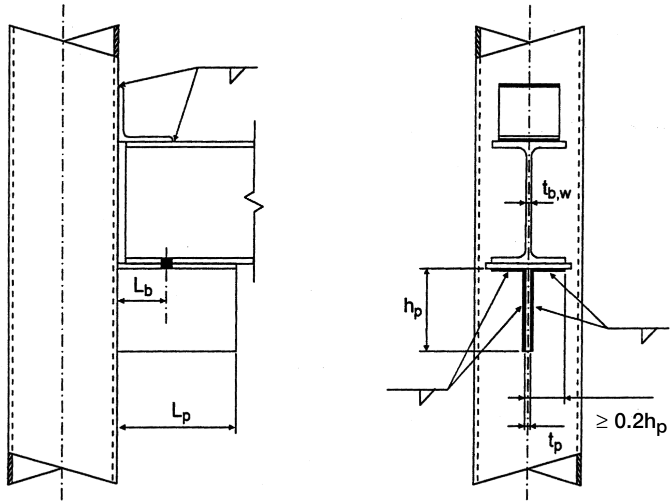



Figure 5.11 – Stiffened seat connection

The seat is again assumed to carry the entire end reaction of the supported beam and the comments given above for the top angle of the unstiffened seated connection are again applicable here. The supported beam must be bolted to the seat plate (tee flange) with two bolts of at least M19 Grade 8.8 (ASTM A325) capacity, to account for prying action caused by the rotation of the joint at ultimate load. Welding the beam to the seat plate is not recommended. Also, the distance ($L_b$) from the RHS column face to the centreline of the bolts should be not greater than the larger of {half the length of the seat plate ($L_p$) measured normal from the RHS column face; and 67 mm}, for practical size beams (AISC 1997).

The thickness of the horizontal seat plate (or tee flange) should be at least 10 mm. Welds connecting the two plates should have a strength not less than the horizontal welds to the support under the seat. It is also a conservative recommendation that the thickness of the tee web (or stem) $t_p$ (see figure 5.11) be (AISC 1997):

$$t_p \geq (f_{b,y}/f_{p,y})\, t_{b,w} \qquad \text{and also } \geq cw \quad \text{.......} 5.3$$

where w is the weld (leg) size and c = 1.5 for $f_{p,y}$ of 350 N/mm², with the welds being assumed to be made of electrode having an ultimate strength of 480 N/mm². Alternatively, if the tee web (or stem) material has $f_{p,y}$ of 250 N/mm², but the same (overmatching) electrode is still used, then c can be taken as 2.

As mentioned in section 5.2, there are two limit states for the RHS face to be checked:

(i) shear yield strength of the tube wall adjacent to the two vertical welds along the tee web (or stem). This failure mode has been cited many times and sample calculations are given in section 5.3.1.
(ii) plastification of the tube wall, using a rotational yield line mechanism. A limit states design resistance for the RHS connecting face under in-plane moment loading is given in section 6.1.2 of CIDECT Design Guide No. 3 (Packer et al. 1992), for an RHS-to-RHS tee joint. That yield line failure mode is deemed to only be applicable for joint width ratios (the ratio in this case of the seat flange width to the RHS column width) less than

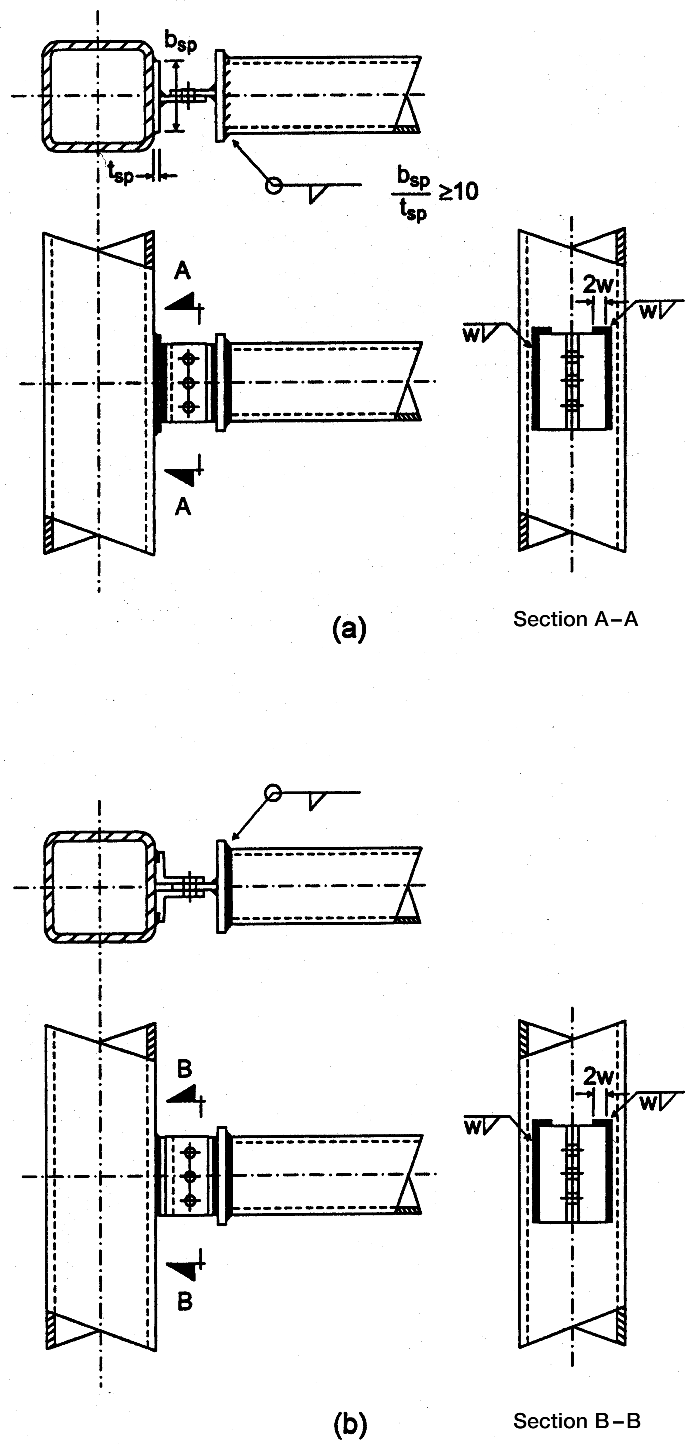
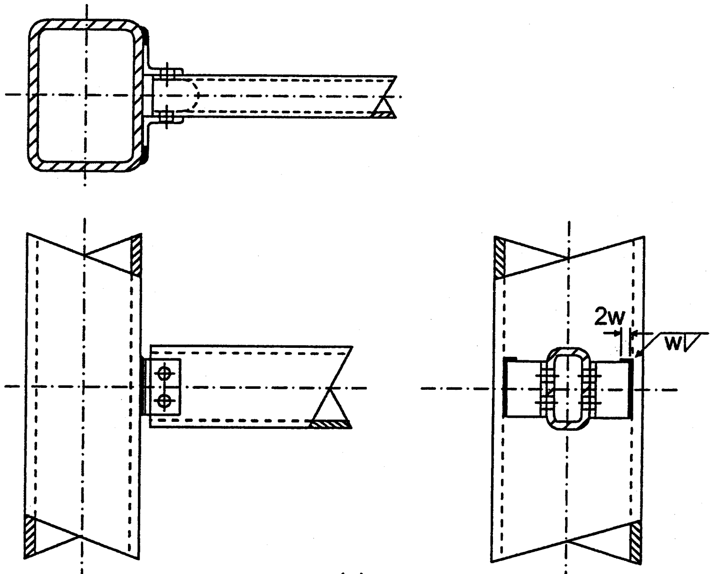
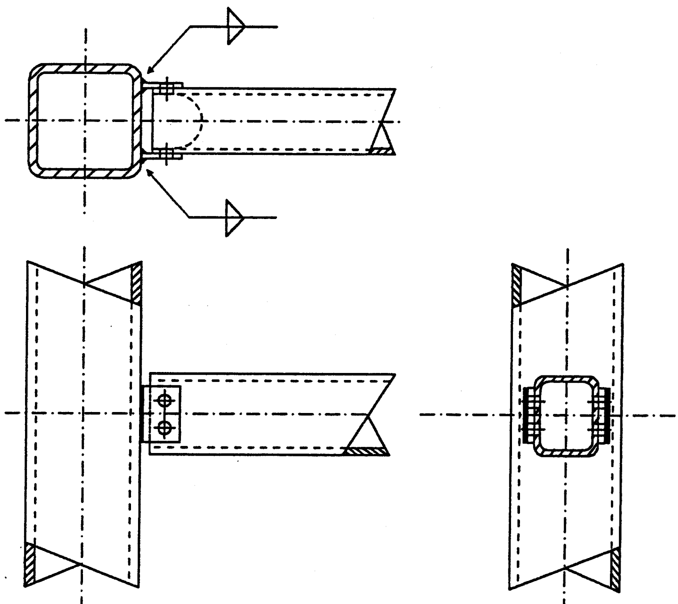



Figure 5.12 – Simple shear connections with hollow section beams

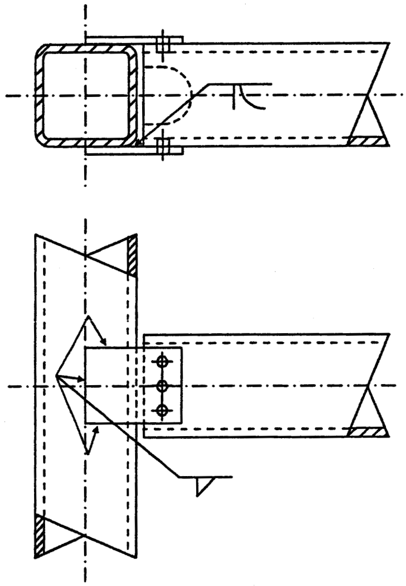



(c)

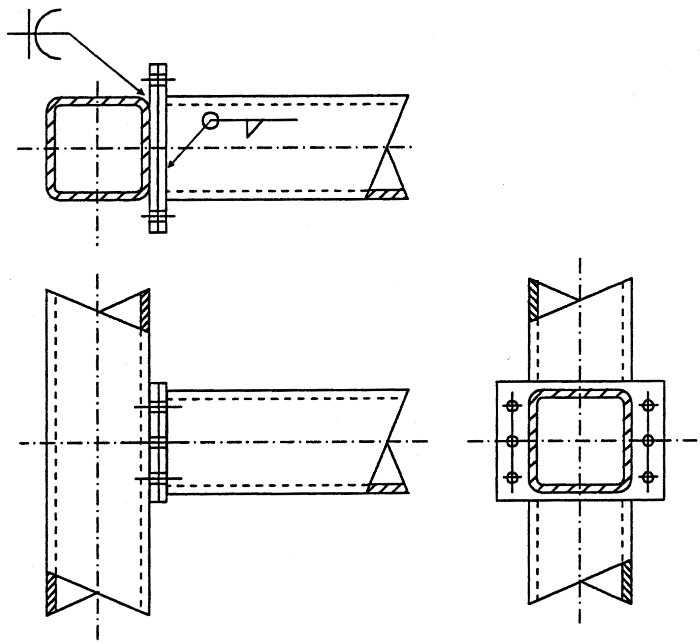


(d)

Figure 5.12 – Simple shear connections with hollow section beams



(e)



(f)

Figure 5.12 – Simple shear connections with hollow section beams

or equal to 0.85. In applying that yield line solution, the depth of the stiffened seat ($h_p$) replaces the term for the branch member depth ($h_1$), the width of the seat flange (which is recommended to be at least $0.4h_p$) replaces the term for the branch member width ($b_1$) and the bending moment applied to the RHS column face is the beam shear force reaction multiplied by the effective eccentricity (e) of this load from the column face. AISC (1997) takes this effective eccentricity, for this connection type, as $0.8L_p$ (see figure 5.11). In section 6.1.2 of CIDECT Design Guide No. 3 it can be seen that this RHS column face moment resistance is also reduced by the effect of the axial compression load in the column.

## 5.10 Hollow section beams to hollow section columns

I-section beams are the usual choice in pin-jointed (simply connected) frames, and these have been presumed in the preceding sections. However, there are instances where hollow sections are used for both the columns and beams. Detail material such as tees, angles and plates is again generally used and site-bolted connections are typical. Some examples are given in figure 5.12.

Figure 5.12(a) shows a double tee connection, with the tees either built up from plates or cut from rolled sections. Central alignment of the beam and column can be maintained by offsetting the tees. The column tee should be welded only along the vertical edges, and should include a short weld return around the top corners. Design guidance for proportioning the column tee is given in section 5.6. Figure 5.12(b) shows a pair of angles that provide double shear loading on the bolts. Welding of these angles to the column is the same as for the tee in figure 5.12(a).

Figure 5.12(c) illustrates a relatively narrow beam framing into a wide RHS column. Angles, welded near the column corners, are used on either side of the beam that has the bottom flange cut back to provide access for bolting. Eccentric loading in the plane of the column face upon the angle welds may limit the capacity of this arrangement. Hence, this unconventional connection type should only be used for lightly-loaded beams. Figure 5.12(d) shows a better match of beam and column widths where two shear plates are used. This would be a relatively stiff connection with the plates welded near the column corners. Again, a cutout to the beam bottom flange is used, to facilitate conventional bolting. A slight variation of this connection is shown in figure 5.12(e), where the members have equal width. Some of these connection arrangements have potential for difficult fitting at the site due to welding-induced distortions. It may be necessary to spread pairs of plates slightly with jacks after the welds cool, because welding contraction will tend to deflect and pull the plates together.

Figure 5.12(f) portrays a popular end plate connection, with bolting done easily beyond the width of the members. This connection accommodates both RHS and CHS beams and allows the hollow section beam to be sealed. A variation of this connection concept is shown in figure 5.13.

## 5.11 Use of through-bolts to hollow section columns

Rather than using “blind bolts” (or a “single-sided bolting system”) when bolting an element such as a beam end plate to a RHS column, it is possible to use long bolts or threaded rods which pass all the way through the RHS column member. Thus, both the head and the nut of the bolt (or threaded rod) are on the outside of the RHS. This is acceptable in a shear connection if the bolts are only snug tight (i.e. not fully pretensioned). Single shear plate connections have been tested by Sherman (1995) with bolts snug tight and fully pretensioned, and both have performed adequately. The connections with snug tight bolts had the same ultimate capacities and eccentricities as those with pretensioned bolts. However, at working loads, pretensioned bolts produced larger eccentricities (to the contraflexure point where negative moment changed to positive moment).

In many connections the bolts will be fully pretensioned, especially if the bolts are liable to go into tension. With full pretensioning, through bolts should only be used if the bolt inside the RHS walls passes through a spacer tube or pipe. This spacer tube should have a length equal to the inside depth of the RHS so that when the bolt is tensioned the spacer tube is placed in compression, hence preventing the flexible faces of the RHS from being pulled inwards and thereby being deformed during the erection process. Section 3.5.6 of CIDECT Design Guide No. 6 (Wardenier et al. 1995) gives some fabrication advice for such joints.

### 5.12 Influence of concrete slabs on behaviour of connections

Composite floor systems are now commonplace in “steel frames”. A concrete slab around the RHS column increases the rotational stiffness of the beam-to-column connection. Thus, a simple shear connection is actually transformed into a composite semi-rigid connection. More information on semi-rigid connections is provided in chapters 6 and 9.

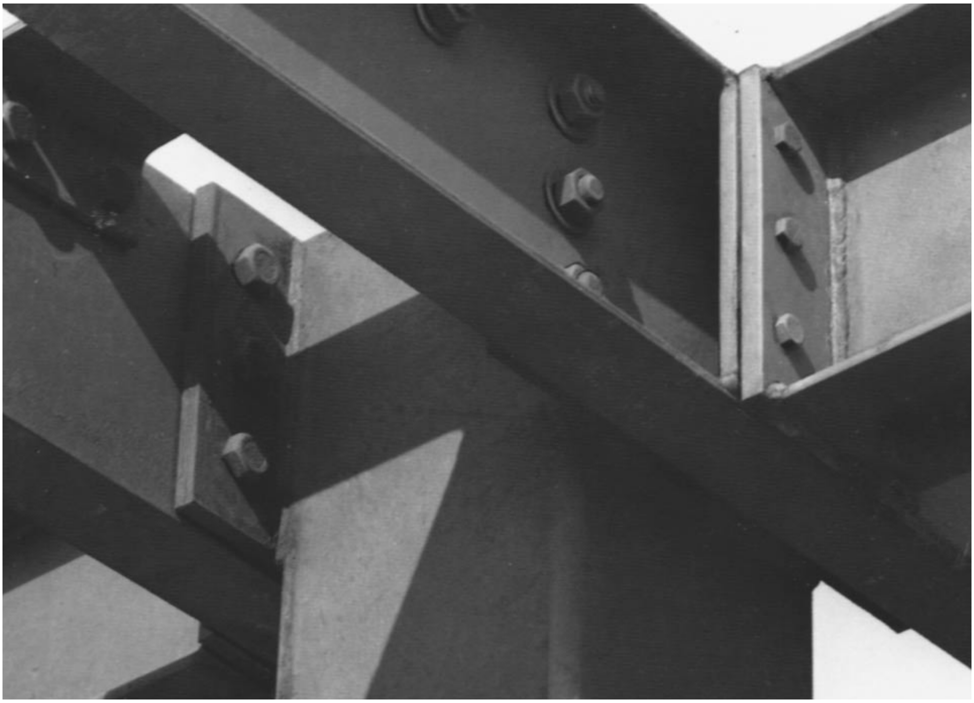


Figure 5.13 – Double channel beams bolted to plates on RHS column faces

# 6 Semi-rigid connections

## 6.1 Types of semi-rigid connections with hollow section members

Moment connections in tubular structures can be designated as indicated in figure 6.1.

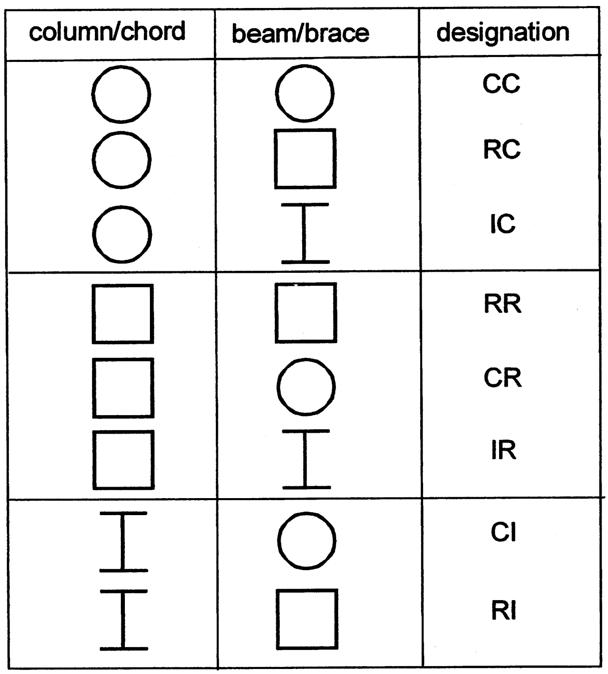



Figure 6.1 – Designation of moment connections with hollow section members

All investigations up to now concern mainly the determination of the moment capacity. Only formulae for unstiffened CHS-to-CHS connections exist for the (initial) joint stiffness, e.g. Efthymiou (1985).

For unstiffened CHS-to-CHS, RHS-to-CHS and I-beam-to-CHS connections formulae for the moment capacity are given in Eurocode 3: Annex K (CEN, 1992), IIW-XV-E (1989) and CIDECT Design Guide No. 1 (Wardenier et al. 1991) mainly based on Kurobane (1981). A detailed study including multiplanar connections is given in Van der Vegte (1995).

Formulae for the moment capacity of CHS-to-CHS connections are also given in API (1997), AWS (2002) and in the draft ISO standard (ISO, 1997), however these standards use a different database and give different recommendations.

Recently a reanalysis has been carried out by Ariyoshi and Makino (2000) on plate-to-CHS connections. This information can also be used for RHS to CHS and I-beam-to-CHS connections.

De Winkel (1998) gives information for the strength of uniplanar and multiplanar I-beam-to-CHS connections, but the stiffness can only be derived from the moment rotation diagrams.

For unstiffened RHS-to-RHS and I-beam-to-RHS connections, design formulae are given in Eurocode 3: Annex K (CEN, 1992), IIW-XV-E (1989) and the CIDECT Design Guide (Packer et al., 1992) mainly based on Wardenier (1982). Yu (1997) has recently carried out a very detailed study on uniplanar and multiplanar RHS-to-RHS connections with various loading combinations.

Lu (1997) gives information for the strength of uniplanar and multiplanar I-beam-to-RHS connections, but, again, the stiffness can only be derived from the moment rotation diagrams. For unstiffened I-beam-to-CHS and I-beam-to-RHS, the moment capacity is based on evidence given in Wardenier (1982) and Packer et al. (1992).

Bolted connections are now becoming more popular due to the use of blind bolting systems. Test results and design recommendations are given by Yeomans (1996, 1996a, 1998) and summarised by Packer and Henderson (1997). Tanaka et al. (1996) describe an interesting new development where the columns have a locally increased wall thickness, which is produced by using local induction heating and axial pressure.

For frames, the I-beam-to-CHS and I-beam-to-RHS connections are most frequently used, since I- or H-sections are primarily used for beams and circular or rectangular hollow sections have advantages for columns. Section 6.2 will concentrate on moment connections between hollow sections (CHS-to-CHS and RHS-to-RHS) and section 6.3 will concentrate on unstiffened I-beam-to-CHS or RHS connections.

## 6.2 Welded hollow section beam and column connections

Beam to hollow section column connections behave in a similar way to the Vierendeel or moment connections dealt with in the CIDECT Design Guides Nos. 1 and 3. However, for completeness they will be dealt with here again. The usual combinations are members of the same type, i.e. CC or RR connections. Other combinations like CR or RC are very rare for moment connections.

### 6.2.1 CHS beam and column members

The strength of moment connections between CHS beams and columns is based on the chord plastification and the chord punching shear criterion. The design strength criteria are given in figure 6.2. Although out-of-plane moments do not occur in plane frames, the strength criteria for out of plane moments are also given for completeness and for the analysis of three-dimensional frames.

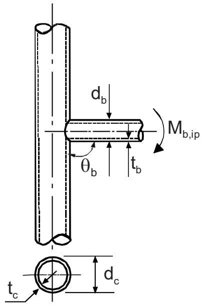
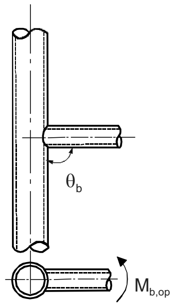
<table>
<tr><th>Type of connection</th><th colspan="3">Factored connection resistance</th></tr>
<tr><td>T,Y,X<br>
</td><td colspan="3">Chord plastification<br>$M^*_{b,ip} = 4.85 \cdot f_{c,y} \cdot t_c^2 \cdot \gamma^{0.5} \cdot \beta \cdot d_b \cdot \frac{f(n')}{\sin\theta_b}$</td></tr>
<tr><td>T,Y,X<br>
</td><td colspan="3">Chord plastification<br>$M^*_{b,op} = f_{c,y} \cdot t_c^2 \cdot \frac{2.7}{1-0.81\cdot\beta} \cdot \frac{f(n')}{\sin\theta_b} \cdot d_b$</td></tr>
<tr><th>General</th><td colspan="3">Punching shear check</td></tr>
<tr><td>Punching shear check for<br>$d_b \le d_c - 2 \cdot t_c$</td><td colspan="3">$M^*_{b,ip} = \frac{f_{c,y}}{\sqrt{3}} \cdot t_c \cdot d_b^2 \cdot \frac{1+3\cdot\sin\theta_b}{4\cdot\sin^2\theta_b}$<br>$M^*_{b,op} = \frac{f_{c,y}}{\sqrt{3}} \cdot t_c \cdot d_b^2 \cdot \frac{3+\sin\theta_b}{4\cdot\sin^2\theta_b}$</td></tr>
<tr><th colspan="4">Functions</th></tr>
<tr><td></td><td colspan="3">$f(n') = 1 + 0.3 \cdot n' - 0.3(n')^2$ for $n' \le 1.0$<br>$f(n') = 1.0$ for $n' > 1.0$<br>$n' = f_{c,p} / f_{c,y}$</td></tr>
<tr><th colspan="4">Range of validity</th></tr>
<tr><td>$0.2 < \frac{d_b}{d_c} \le 1.0$</td><td>Class 2 and<br>$\frac{d_b}{2 \cdot t_b} \le 25$</td><td>$30^\circ \le \theta_b \le 90^\circ$<br>$0.55 < \frac{e}{d_c} \le 0.25$</td><td>$\gamma \le 25$<br>$\gamma \le 20$ (X – joints)</td></tr>
</table>

Figure 6.2 – Design recommendations for CC connections loaded by primary bending moments

The function f(n') represents the influence of the axial and/or bending stress in the column on the resistance according to the chord plastification criterion. As shown in CIDECT Design Guide No. 1 (Wardenier et al. 1991) the design strength for in-plane bending moments is considerably larger than that for out-of-plane moments. For a good efficiency it is recommended to choose the diameter to thickness ratio $d_C / t_C$ of the column preferably below 25 and the yield stress-thickness ratio between column and beam $f_{c,y}\, t_c > 2 \cdot f_{b,y} \cdot t_b$

For stiffness reasons the diameter ratio β between beam and column should be large, preferably close to 1.0.

Combinations of axial loads and bending moments have to be checked for interaction according to:

$$\frac{N_b}{N_b^*} + \left[\frac{M_{b,\,ip}}{M_{b,\,ip}^*}\right]^2 + \frac{M_{b,\,op}}{M_{b,\,op}^*} \leq 1.0 \quad \text{......6.1}$$

In which $N_b$ , $M_{b,ip}$ and $M_{b,op}$ are the loads acting, and $N_b^*$, $M_{b,ip}^*$and $M_{b,op}^*$are the design capacities.

The rotational stiffness C (moment per radian) for Vierendeel connections with $0.3 \leq \beta \leq 0.8$ is given by Efthymiou (1985).

For T- and Y-connections under in-plane moment loading the connection stiffness is:

$$C_{b,ip} = 1.3\, E\, (d_C/2)^3\, \beta^{(2.25+\gamma/125)}\, \gamma^{-1.44} / \sin^{(\beta+0.4)} (\theta) \quad \text{......6.2}$$

For T- and Y-connections under out-of-plane moment loading the connection stiffness is:

$$C_{b,op} = 2.3\, E\, (d_C/2)^3\, \beta^{(2.12)}\, \gamma^{(0.7(0.55-\beta)^2-2.2)} / \sin^{(\beta+1.3)} (\theta) \quad \text{......6.3}$$

These equations can be used within the following validity range:

$0.3 \leq \beta \leq 0.8$ $\quad 10 \leq \gamma \leq 30$ $\quad 0.3 \leq \tau \leq 0.8$ $\quad 35° \leq \theta \leq 90°$

It should be noted that the stiffness can be affected considerably by the presence of axial and/or bending stresses in the column, however insufficient test evidence is available to quantify this influence in more detail.

Van der Vegte (1995) investigated, among other aspects, the geometrical and the loading effect in CHS multiplanar connections. In this study all kinds of loading situations were considered on the in-plane and out-of-plane members. However, there are so many combinations of loading and the interactions are too complicated for routine design. Therefore, these interaction formulae have not been included in this design guide, but information can be obtained from the given reference.

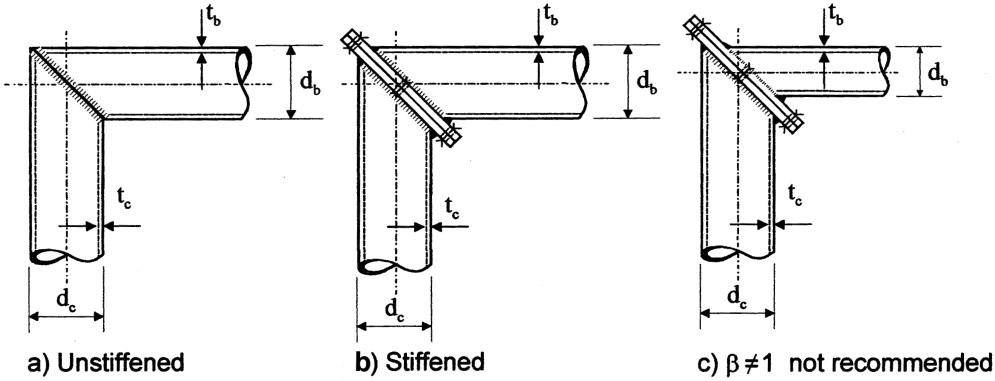



Figure 6.3 – CHS knee connections

Some special types of connections are given in figure 6.3 for frame corners. These knee connections have especially been investigated at the University of Karlsruhe (Karcher and Puthli, 2001 and Choo et al., 2001). They recommend designing these connections based on the following requirements for both members:

$$\frac{N}{N_{pl}} + \frac{M}{M_{pl}} \leq \alpha \qquad \text{......6.4}$$

The term $\alpha$ is a stress reduction factor, which can be taken 1.0 for mitre connections with stiffening plates. For the mitre connections without stiffening plates it is a function of the cross sectional dimensions and is given in figure 6.4 and equation 6.5. The S grade in figure 6.4 corresponds to the nominal yield stress $f_y$.

$$\alpha = (0.05\ d/t + 0.77)^{-1.2}\ (235/f_y)^{0.5} \qquad \text{......6.5}$$

Based on previous work (Mang et al., 1991), it is recommended that for connections without stiffening plates the shear force V and the axial force N should not exceed:

$V/V_{pl} \leq 0.5$ and $N/N_{pl} \leq 0.2$

For mitre knee connections with angles $\theta > 90°$ the same recommendations as for $\theta = 90°$ can be adopted (Karcher and Puthli, 2001).

Although the unstiffened connections have been investigated for $10 \leq d/t \leq 100$, it is recommended that for structural applications d/t is restricted to class 1 sections according to EC 3 (CEN, 1992).

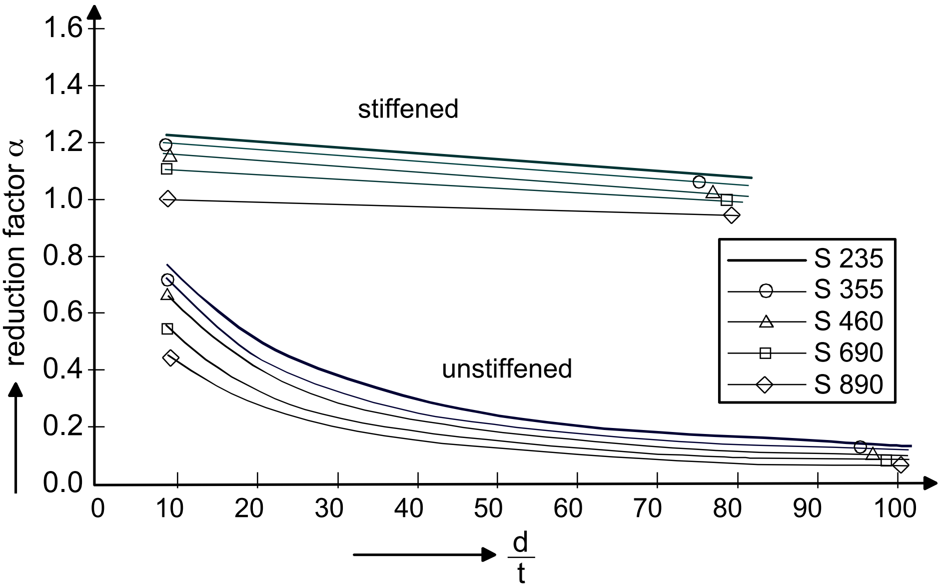



Figure 6.4 – Stress reduction factor $\alpha$, for unstiffened mitred CHS knee connections

The connections with a stiffening plate can be considered to be rigid, whereas the stiffness behaviour of the unstiffened connections depends on the d/t ratio.

The unstiffened connections can only be assumed to be rigid for very low d/t ratios. No formulae for the joint stiffness are available.

For those structural applications where a reasonable strength, stiffness and rotational capacity are required it is recommended that a stiffened connection using class 1 sections is used. For other structural applications it is recommended to use the unstiffened connections only if the sections satisfy at least the plastic design requirements.

The stiffening plate thickness should satisfy $t_p \geq 1.5\ t_b$ and not be taken smaller than 10 mm. An additional requirement is that $d/t \leq 50/(235/f_y)^{0.5}$.

### 6.2.2 RHS beam and column members

The strength of welded moment connections of RHS beams and columns without stiffeners is based on various failure modes, i.e.:

- column face yielding (plastification);
- cracking of the column face (chord punching shear);
- cracking in the beam (effective width);
- yielding or crippling of the column side walls;
- column shear.

These failure modes and the strength criteria are dealt with in detail in Wardenier (1982) and in CIDECT Design Guide No. 3 (Packer et al. 1992) and are here summarised in figure 6.5 for a connection of square hollow sections with $\theta$ = 90°. The strength criteria for out-of-plane moments are also given for completeness and for three-dimensional frames. However, it should be noted that the formulae for out-of-plane loading should only be used if distortion of the chord cross section is prevented, e.g. by stiffeners located close to the connection. These design recommendations have also been adopted for Eurocode 3, Annex K (CEN 1992).

The moment capacity of connections with low to moderate $\beta$ values (0.85) can be determined with a yield line model. The function f(n) is a function to allow for the reduction in moment capacity due to the presence of compression stresses in the column face. For values $\beta > 0.85$, depending on the geometry parameters, several failure modes may be critical. As shown in figure 6.5, the beam effective width criterion and the column side wall failure criterion have to be checked. For a better understanding these criteria are illustrated in figure 6.6.

Punching shear was not observed in the tests and not given as a separate check in figure 6.5 but it is recommended to design the beams with a thickness $t_b < 0.6t_c$ or avoid connections with $b_b \approx b_c - 2t_c$ where punching shear may occur. More detailed information is given in CIDECT Design Guide No. 3 (Packer et al., 1992).

From the expressions in figure 6.5 it can be seen that full width ($\beta = 1.0$) unstiffened RHS Vierendeel connections are capable of developing the full moment capacity of the beam, providing $b_c/t_c$ is sufficiently low. For $h_c = b_c = h_b = b_b$ and $b_c/t_c < 16$ the chord side wall crippling is given by Wardenier (1982):

$$\frac{M^*_{b,ip}}{M_{pl}} \approx \frac{8}{b_c/t_c} \cdot \frac{f_{c,y} \cdot t_c}{f_{b,y} \cdot t_b} \qquad \text{.......... 6.6}$$

Thus, for beam to column connections of square sections with a $\beta \approx 1$, a column width to thickness ratio $b_c/t_c = 16$ and a column to beam thickness ratio $t_c/t_b = 2$ the moment capacity will be equal to the beam plastic moment capacity. This agrees with findings from Korol et al. (1977).

The previous expressions for the moment capacity are based on moment loading only, however axial loads in the beams may also exist. The interaction between axial loads and bending moments depends on the failure criterion. A conservative approximation is to use a linear relationship:

$$\frac{N_b}{N^*_b} + \frac{M_{b,ip}}{M^*_{b,ip}} + \frac{M_{b,op}}{M^*_{b,op}} \leq 1.0 \qquad \text{.......... 6.7}$$

Yu (1997), in a similar way to van der Vegte (1995) for CHS connections, investigated the geometrical and the loading effect in RHS multiplanar connections. In her study all kinds of loading situations were considered on the in-plane and out-of-plane members. However, there are so many combinations of loading and the interactions are too complicated for routine design. Therefore, these interaction formulae have not been included in this design guide, but information can be obtained from the given reference. The work of Yu confirmed that the CIDECT formulae for moment loaded RHS to RHS connections in figure 6.5 give a lower bound for the FE results based on the load or moment capacity at a local deformation of 3% of the column width $b_c$.

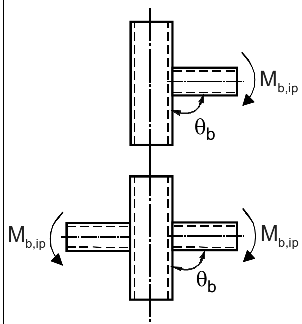
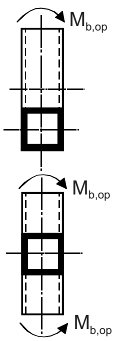
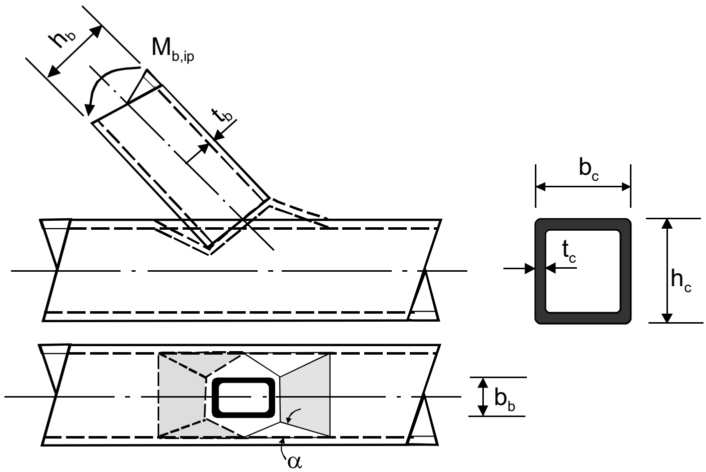
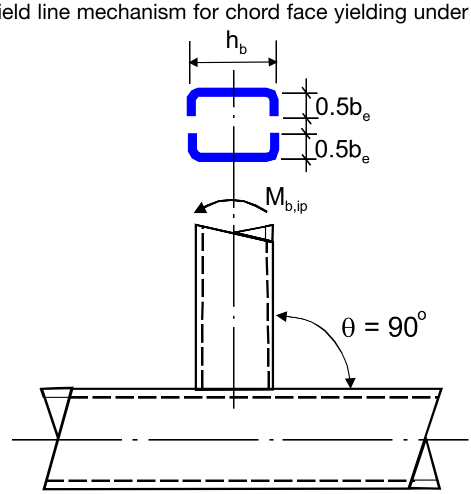
| Type of connection | Factored connection resistance |
|---|---|
| T and X connection under in-plane bending moments | $\beta \le 0.85$ basis: chord face yielding |
|  | $M^*_{b,ip} = f_{c,y} \cdot t_c^2 \cdot h_b \cdot \left\{ \frac{1}{2 \cdot h_b / b_c} + \frac{2}{\sqrt{1-\beta}} + \frac{h_b / b_c}{(1-\beta)} \right\} \cdot f(n)$ |
| | $0.85 < \beta \le 1.0$ basis: effective width |
| | $M^*_{b,ip} = f_{b,y} \cdot \left\{ W_{b,pl} - \left( 1 - \frac{b_e}{b_b} \right) \cdot b_b \cdot t_b \cdot (h_b - t_b) \right\}$ |
| | $0.85 < \beta \le 1.0$ basis: chord side wall failure |
| | $M^*_{b,ip} = 0.5 \cdot f_k \cdot t_c \cdot (h_b + 5 \cdot t_c)^2$ |
| T and X connection under out-of-plane bending moments | $\beta \le 0.85$ basis: chord face yielding |
|  | $M^*_{b,op} = f_{c,y} \cdot t_c^2 \cdot \left\{ \frac{h_b \cdot (1+\beta)}{2 \cdot (1-\beta)} + \sqrt{\frac{2 \cdot b_c \cdot b_b \cdot (1+\beta)}{(1-\beta)}} \right\} \cdot f(n)$ |
| | $0.85 < \beta \le 1.0$ basis: effective width |
| | $M^*_{b,op} = f_{b,y} \cdot \left\{ W_{b,pl} - 0.5 \cdot t_b \cdot (b_b - b_e)^2 \right\}$ |
| | $0.85 < \beta \le 1.0$ basis: chord side wall failure |
| | $M^*_{b,op} = f_k \cdot t_c \cdot (h_b + 5 \cdot t_c) \cdot (b_c - t_c)$ |
| **Function** | |
| f(n) = 1.0 for $n \ge 0$(tension)<br>f(n) =1.3 + $\frac{0.4}{\beta} \cdot n$ for n < 0<br>but $\le 1.0$<br>$n = \frac{N_c}{A_c \cdot f_{c,y}} + \frac{M_c}{W_{c,el} \cdot f_{c,y}}$ | $b_e = \frac{10}{b_c / t_c} \cdot \frac{f_{c,y} \cdot t_c}{f_{b,y} \cdot t_b} \cdot b_b \le b_b$<br>$f_k = f_{c,y}$ for T connections<br>$f_k = 0.8 \cdot f_{c,y}$ for X connections |
| **Range of validity** | |
| Braces: class 2 sections | $\frac{b_c}{t_c}$ and $\frac{h_c}{t_c} \le 35$<br>$\theta_b = 90^\circ$ |

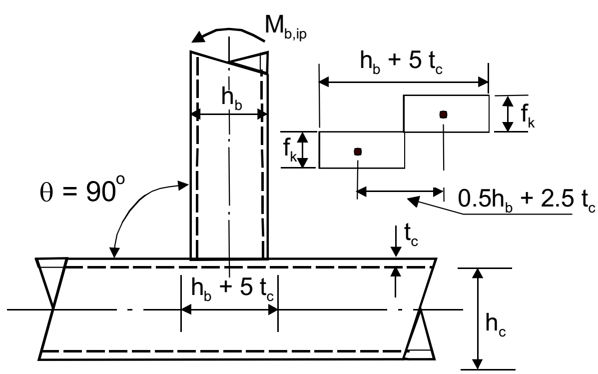
Figure 6.5 – Design recommendations for RHS-to-RHS connections loaded by primary bending moments




a. Yield line mechanism for chord face yielding under in-plane bending




b. Brace effective width criterion for T, Y and X joints




c. Chord side wall bearing or buckling failure model under in-plane bending

Figure 6.6 - Failure criteria for RHS-to-RHS moment connections loaded by in-plane bending

The design capacities for axial loading $N_b$* can be obtained from the CIDECT Design Guide No. 3 (Packer et al., 1992) and are not reproduced here again.

The connections between rectangular hollow sections with ratios $\beta < 1.0$ are not stiff enough to be used as moment connections. However, they can be stiffened by plates or haunches.

Figure 6.7 shows some knee connections for Vierendeel girders or for frame corners. These knee connections have been investigated at the University of Karlsruhe (Mang et al., 1991) and at the University of Sydney (Wilkinson and Hancock,1998). Based on the research evidence it is recommended in CIDECT Design Guide No. 3 to design these connections based on the following requirements for both members:

$$\frac{N}{N_{pl}} + \frac{M}{M_{pl}} \le \alpha \qquad \text{.......} 6.8$$

with $V/V_{pl} \le 0.5$ and $N/N_{pl} \le 0.2$ .......6.9

Here N, M and V refer to the acting axial force, the acting bending moment and the acting shear force in a connecting member at the connection, whereas $N_{pl}$, $M_{pl}$ and $V_{pl}$ are the capacities of the connecting member with

$$V_{pl} = 2h\ t\ f_y/\sqrt{3} \qquad \text{.......} 6.10$$

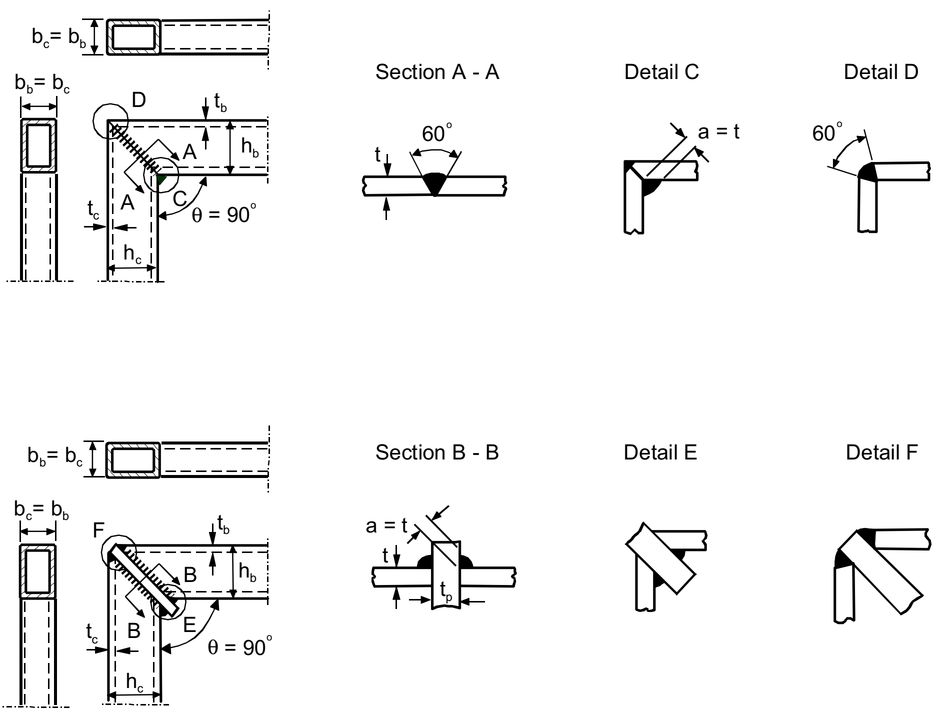



Figure 6.7 – RHS knee connections

Similar to the approach for CHS-to-CHS knee connections the term $\alpha$ is a stress reduction factor, which can be taken as 1.0 for mitre connections with stiffening plates. For the mitre connections without stiffening plates it is a function of the cross sectional dimensions and is shown in figures 6.8 and 6.9. If mitre knee connections are used with an angle $\theta > 90°$ between the members use conservatively the same design checks as for $\theta = 90°$.

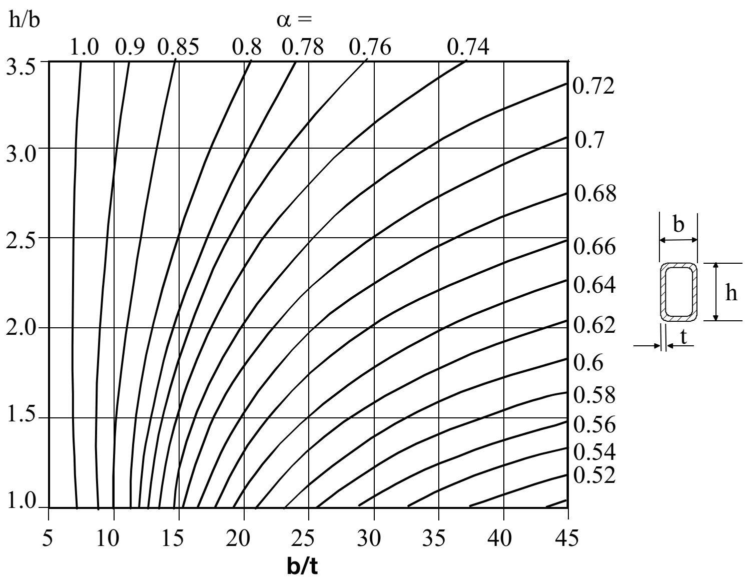



Figure 6.8 – Stress reduction factor $\alpha$, for 90° unstiffened mitred RHS knee connections subjected to bending about the major axis

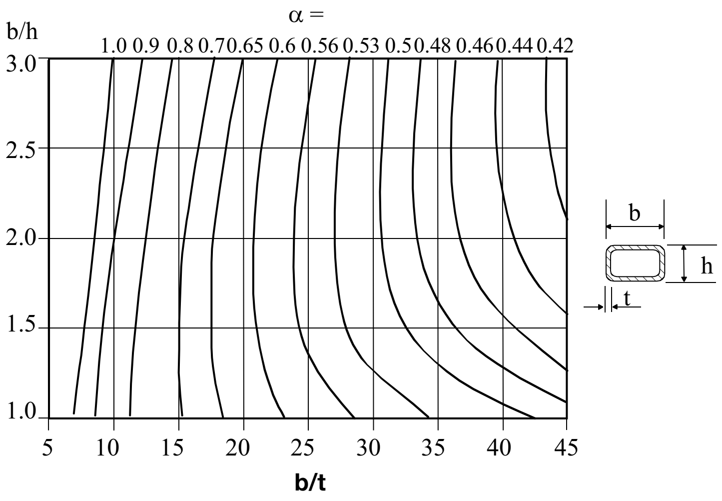



Figure 6.9 – Stress reduction factor $\alpha$, for 90° unstiffened mitred RHS knee connections subjected to bending about the minor axis

Since the rotation capacity of the unstiffened connections might be rather low, it is also recommended here to use a stiffened connection for those structural applications where a reasonable rotational capacity is required. For other structural applications it is recommended to use the unstiffened connections only if the sections satisfy at least the plastic design requirements.

The stiffening plate thickness should satisfy $t_p \geq 1.5t$ and not be taken smaller than 10 mm.

Additional requirements are that the welds should be at least equal to the connected wall thickness and that the factor $\alpha$ used in design should be:

$\alpha < 0.84$ for $f_y = 235$ N/mm$^2$

$\alpha < 0.71$ for $f_y = 355$ N/mm$^2$

The connections with a stiffening plate can be considered to be rigid whereas the stiffness behaviour of the unstiffened connections especially depends on the b/t and h/t ratio. Only for very low b/t ratios can the connection be assumed to be rigid. No formulae for the joint stiffness are available.

An alternative form of connection reinforcement is a haunch of the same width as the connected RHS members on the inside of the knee. However, insufficient test evidence is available to quantify the properties, especially the rotational capacity, of this connection type.

### 6.2.3 CHS and RHS beam and column members

Connections with a CHS beam and a RHS column are very rare and not efficient in transferring moments. Connections with a RHS beam and a CHS column are not frequently used because of the end preparation required, however, with the current end cutting machines, the end preparation is not a problem anymore. For moment loading no test results are available, but based on the research on plate and I-beam-to-CHS column connections recommendations are given in CIDECT Design Guide No. 1 which have been adopted for Eurocode 3 and will also be adopted here.

The design formulae are given together with the I-beam-to-CHS column connections in figure 6.11.

## 6.3 Welded I-beam-to-hollow section column connections

The first investigations on unstiffened connections between plates or I-beam and CHS hollow sections have been carried out in Japan. The work of Akiyama, Kamba, Kanatani, Kurobane, Makino, Sasagawa, Suzuki, Tabuchi, Taguchi, Tanaka and Wakabayashi, mainly published in Japanese papers, has been first summarised and analysed by Kurobane (1981). Later reanalyses have been given by Wardenier (1982), Makino (1984), Kamba and Taclendo (1998) and recently by Ariyoshi and Makino (2000).

Unstiffened moment connections between plates or I-beams and RHS hollow sections have initially been investigated by Kanatani et al. (1980).

Ting et al. (1991) and Shanmugan et al. (1993) investigated numerically the effect of various types of external stiffeners for I-beam-to-RHS columns.

The rigid diaphragm stiffened connections have been extensively studied in Japan and summarised by Kurobane (1981) and Kamba et al. (1995, 1998). Most of the other research is related to simple shear connections (using shear tabs, plates or cleats) or rigid moment connections using straps or diaphragm plates or other reinforcing plates, see chapters 5 and 8.

In the nineties, an extensive programme has been carried out by Lu (1997) and de Winkel (1998) to investigate the behaviour of unstiffened uniplanar and multiplanar connections between I- or H-section beams and circular or square hollow section columns (see figure 6.10).

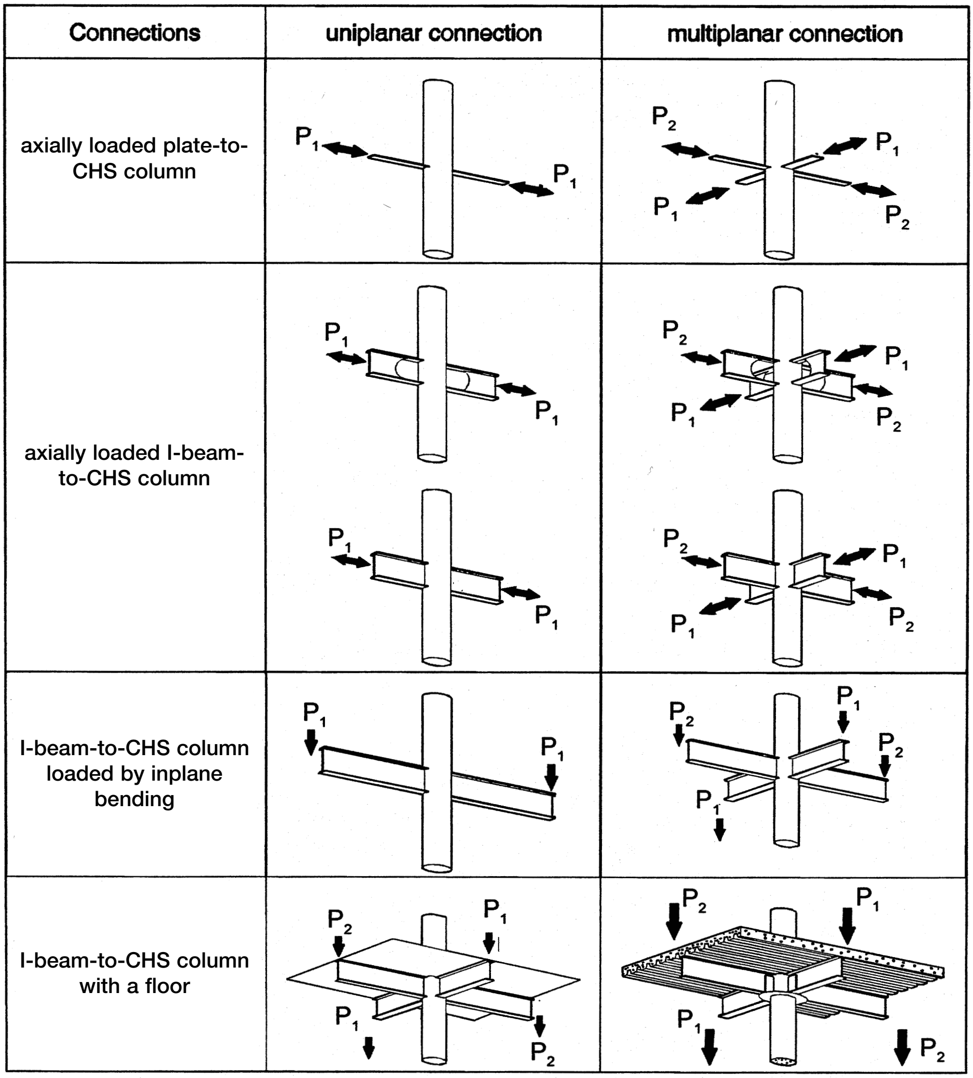



Figure 6.10 - I-beam-to-CHS or RHS column connections investigated by de Winkel (1998) and by Lu (1997); only the I-beam-to-CHS column connections are shown here.

Within this programme, the following aspects have been investigated both for circular hollow section columns and for square hollow section columns.

1. Behaviour of plate to tubular column connections
   Multiplanar geometrical effect
   Multiplanar load effect
2. Interaction of two plates at different distances
   Effect of beam web
3. Behaviour of I- or H-beam to tubular column connections loaded by in-plane bending moments
   Multiplanar geometrical effect
   Multiplanar load effect
4. Effect of a steel plate floor (offshore)
5. Effect of a composite steel-concrete floor (buildings)
6. Influence of concrete filling of the column for the various load conditions
7. Influence of column loading or the moment capacity

The programme was set up in such a way that information could be obtained for particular parts, i.e. flange and web and the various parameter influences investigated. The intention was to simulate the behaviour of the more complicated connections by combination of the separate effects. For example, the moment rotation diagram for a beam to column connection with a composite floor can be built up from the load-deformation behaviour of the connection of the bottom flanges, the web, the bolts, the studs between beam and concrete and the reinforcement of the concrete. However, such a component method could not be presented in a simple and sufficiently accurate way.

With numerical simulations many load deformation and moment-rotation curves have been established. The work concentrated on the strength formulation but information for the stiffness can be obtained from the many load (moment)-deformation (rotation) diagrams.

All models used in the parameter study were carried out for columns with a diameter or width of 300 mm and varying thickness and beam dimensions. All welds were modelled as butt (groove) welds which results in somewhat lower results than specimens with fillet welds.

The ultimate load capacity was defined as the peak in the load-deformation curve or moment-rotation curve, or, if reached earlier, the load or moment at which a local deflection of 3% $b_c$ or 3% $d_c$ occured in the column wall (Lu et al. 1994, Lu 1997).

### 6.3.1 I-beam-to-CHS column connections

As mentioned in section 6.3, recent analyses for gusset plate-to-CHS connections have been given by Kamba and Taclendo (1998) and by Ariyoshi and Makino (2000). However, these formulae for the yield and ultimate strength need further evaluation and modification to design strengths and further simplification. This may be done for the next revision of the IIW-XV-E and CIDECT design recommendations. In principle the same applies to the work of de Winkel (1998) on unstiffened I-beam-to-CHS column moment connections.

In this design guide the recommendations are consistent with the recommendations in CIDECT Design Guide No. 1 which have been based on Kurobane (1981), Wardenier (1982) and other reanalyses for the IIW-XV-E committee and Eurocode 3 and later confirmed by

Makino et al. (1991). These formulae have also been adopted for Eurocode 3. Where required, the recommendations here have been extended or modified based on the mentioned recent investigations.

The recommended formulae for the design strength of a plate, an I-beam and a RHS-to CHS column connection are based on the ring model approach for chord plastification with a statistical curve fitting. They are given in figure 6.11.

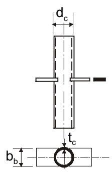
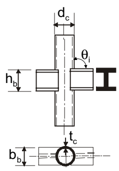
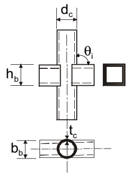
| **Design strength for XP- and TP-joints** | | | | | | |
|---|---|---|---|---|---|---|
| **Type of connection** | **Axial loading** $N^* = f(\beta) \cdot f(\eta) \cdot f(n') \cdot f_{c,y} \cdot t_c^2$ | | | | **Bending in plane** | **Bending out-of-plane** |
| | $f(\beta)$ | $f(\eta)$ | $f(n')$ | $f_{c,y} \cdot t_c^2$ | | |
| XP-1/TP-1  | $\frac{5.0}{1-0.81 \cdot \beta}$ | 1 | $f(n')$ | $f_{c,y} \cdot t_c^2$ | ------ | $M^*_{b,op} = 0.5 \cdot b_b \cdot N_{(XP-1)}$ |
| XP-4/TP-4  | $\frac{5.0}{1-0.81 \cdot \beta}$ | $1+0.25 \cdot \eta$<br>$\eta \le 4$ | $f(n')$ | $f_{c,y} \cdot t_c^2$ | $M^*_{b,ip} = h_b \cdot N^*_{(XP-1)}$ | $M^*_{b,op} = 0.5 \cdot b_b \cdot N_{(XP-4)}$ |
| XP-5/TP-5  | $\frac{5.0}{1-0.81 \cdot \beta}$ | $1+0.25 \cdot \eta$<br>$\eta \le 2$ | $f(n')$ | $f_{c,y} \cdot t_c^2$ | $M^*_{b,ip} = h_b \cdot N^*_{(XP-5)}$<br>$\eta \le 2$ | $M^*_{b,op} = 0.5 \cdot b_b \cdot N_{(XP-5)}$ |
| **General** | **Punching shear check** | | | | | |
| Punching shear check for<br>$b_b \le d_c - 2 \cdot t_c$ | $f_b \cdot t_{b,f} \le 1.16 \cdot f_{c,y} \cdot t_c$<br><br>$f_b \cdot t_{b,f} \le 0.58 \cdot f_{c,y} \cdot t_c$ | for XP-1 / TP-1 (general)<br>for XP-4 / TP-4 (bending in plane)<br>for other cases | | | | |
| **Function** | | | | | | |
| | $f(n') = 1 + 0.3 \cdot n' - 0.3 \cdot (n')^2$<br>$f(n') = 1.0$<br>$n' = \frac{f_{c,p}}{f_{c,y}}$ | for $n' \le 1.0$<br>for $n' > 1.0$ | | | | |
| **Range of validity** | | | | | | |
| $\theta_i = 90°$ | $\frac{d_c}{t_c} \le 40$ | beams: class 2 | | | | |

Figure 6.11 – Design strength formulae for uniplanar RC and IC connections.

The formulae for the moment capacity of the I-beam to column connection are based on the strength of the plate connections. In principle the connections with one beam at one side of the column (indicated as TP) behave somewhat different from those with a beam at both sides of the column (indicated as XP) and in the mentioned literature different formulae are given. From the statistical evaluation the function f(β) for the plate-to-CHS connections is as follows:

for XP-1 connections with two plates:

$$f(\beta) = 5/(1 - 0.81\beta) \quad \text{.......} 6.11$$

for TP-1 connections with one plate:

$$f(\beta) = 4 + 20\beta^2 \quad \text{.......} 6.12$$

However, due to the statistical curve fitting procedures the resulting formulae are not correct for small β values, i.e. the XP connection becomes stronger than the TP connection which is physically incorrect. Therefore, here it is recommended to use the formulae of figure 6.11.

From the work of Ariyoshi and Makino (2000) indications can be given for the axial stiffness of flange plate connections (TP and XP connections). The formulae for the initial stiffness of the flange plate connections have been simplified here and are given by equations 6.13 and 6.14:

for XP-1 connections with two plates:

$$K_{XP\text{-}1} = 6.8\, E\, t_c\, \beta\, (2\gamma)^{-1.3} \quad \text{.......} 6.13$$

for TP-1 connections with one plate:

$$K_{TP\text{-}1} = 1.9\, E\, t_c\, \beta^{1.3}\, (2\gamma)^{-0.7} \quad \text{.......} 6.14$$

The original and the simplified equations are presented in figures 6.12 and 6.13

For the moment rotation behaviour the following equations apply:

$$M_{b,ip} = N_p\,(h_b - t_{b,f}) = C_{b,ip}\,\phi = C_{b,ip}\,\frac{2\,\delta}{(h_b - t_{b,f})} = C_{b,ip}\,\frac{2\,N}{K(h_b - t_{b,f})} \quad \text{.......} 6.15$$

or $$C_{b,ip} = 0.5\,K(h_b - t_{b,f})^2 \quad \text{.......} 6.16$$

Thus multiplying the axial flange plate connection stiffness K by 0.5 $(h_b - t_{b,f})^2$ gives an approximation for the initial rotation $C_{b,ip}$ for IC moment connections, however the effect of the beam web is neglected.

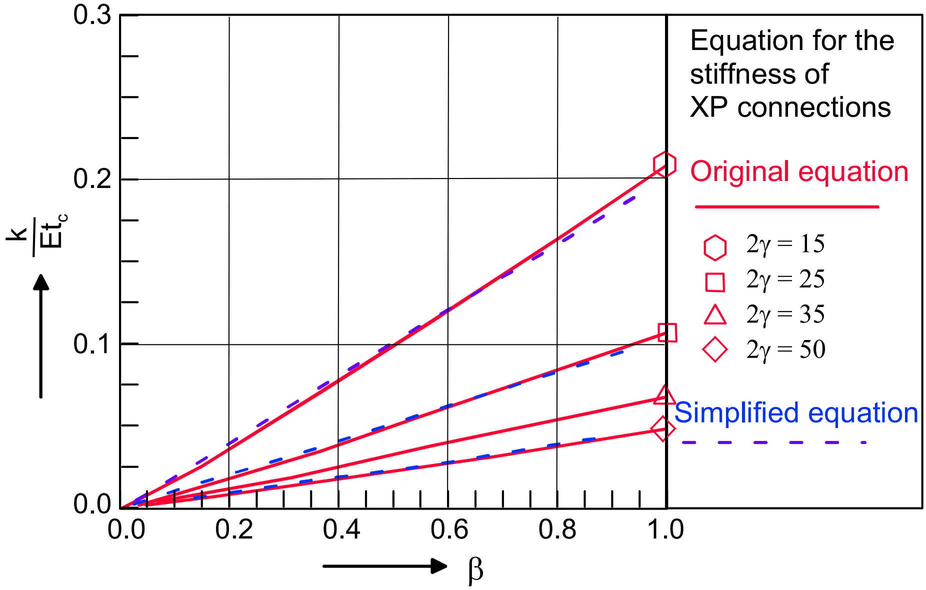



Figure 6.12 – Stiffness of XP-1 connections

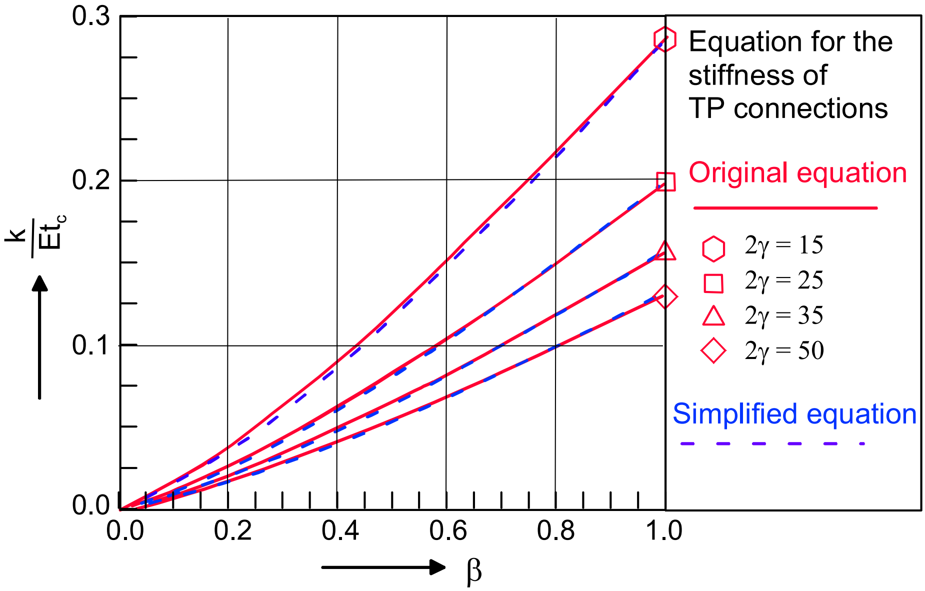



Figure 6.13 – Stiffness of TP-1 connections

From the work of de Winkel (1998) the following conclusions can be drawn:

– Comparing the multiplanar connections with the uniplanar connections shows that the geometrical stiffening effect becomes only significant for $\beta$ values close to 0.7; for $\beta \leq$ 0.5 it is negligible. However, for $\beta$ values close to 0.7 the rotation capacity may also decrease since cracking in the column between the beam flanges may be the critical failure mode.

– Positive load or moment ratios J (i.e. the ratio of the load or moment out-of-plane divided by that in-plane) generally show some beneficial effect, whereas negative load ratios J show a considerable decrease of the capacity.

This effect is here somewhat simplified compared to the original very complicated formula and given by:

$$M_j/M_{j=0} = 1 + J\,(\beta - 0.4\,\beta^2 - 0.1) \quad \text{for } J < 0 \qquad \text{..........6.17a}$$

$$M_j/M_{j=0} = 1 \text{ for } J \geq 0 \qquad \text{..........6.17b}$$

The simplified equations generally are conservative except for low $\beta$ values in combination with $J > 0$. Figures 6.14 to 6.16 give a graphical presentation of the multi-planar effect and the effect of the simplification.

– The influence of the load ratio is independent of the beam depth.

– Axially loaded I-beam-to-CHS column connections have, for $\eta < 2$, a capacity which is less than twice that of the plate-to-CHS connection.

– If the web is not present at the intersection with the column, the connection strength is reduced by only 2–12%.

– Column prestressing decreases the connection capacity considerably. The prestressing function based on the maximum column stress, in a simplified form by excluding the $\beta$ effect, is given by:

$$f(n) = 1 - 0.25n^2\,(2\gamma)^{0.3} \qquad \text{..........6.18}$$

The original and the simplified equation is, for $\beta = 0.45$, given in figure 6.17.

– The use of a steel plate floor, as used for offshore decks, does not enhance the connection capacity.

– Concrete filling of the CHS columns increases the stiffness and capacity considerably but limits the deformation capacity of the connections.

– The connection strength and stiffness can be significantly increased by using a composite floor. However, if the concrete reinforcement is governing for the connection capacity, the rotation capacity is small if cold formed (low ductility) reinforcement is used.

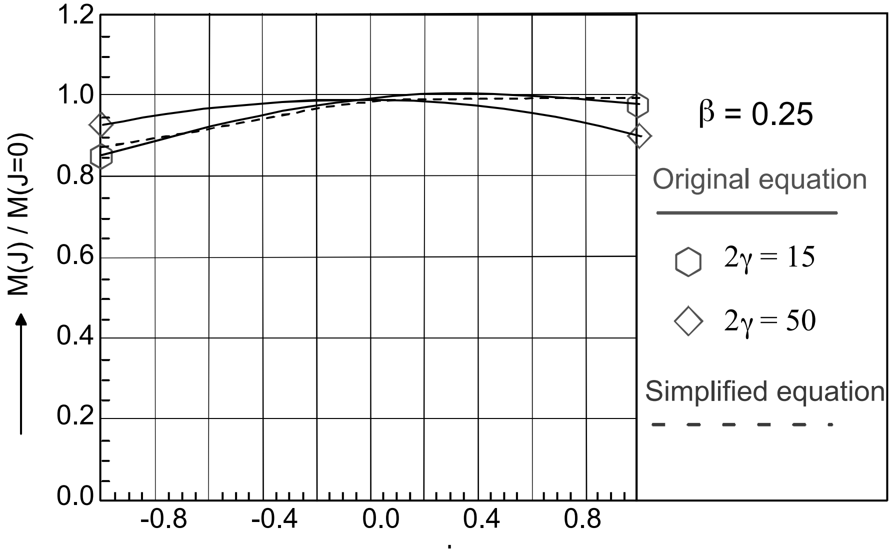



Figure 6.14 – Multiplanar effect of I-beam-to-CHS column connections loaded by in-plane-bending moments (β= 0.25)

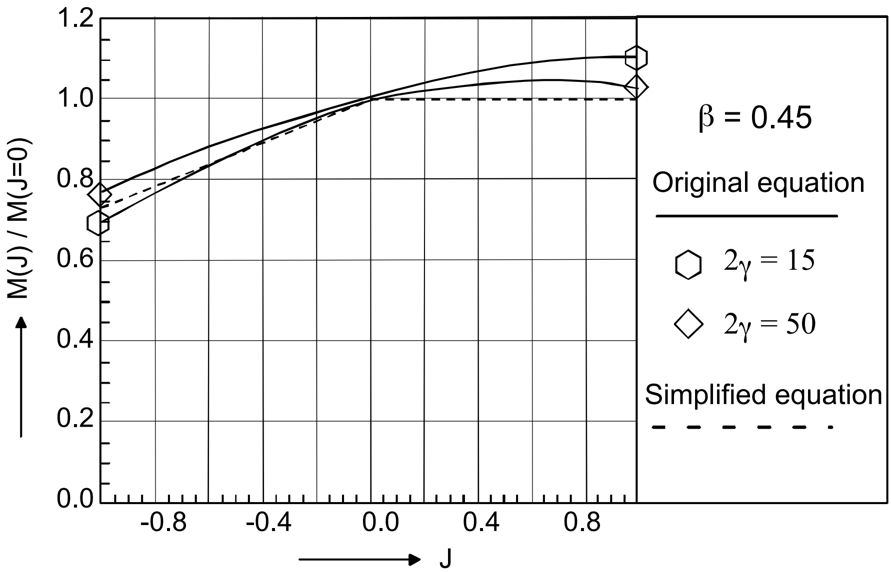



Figure 6.15 – Multiplanar effect of I-beam-to-CHS column connections loaded by in-plane-bending moments (β = 0.45)

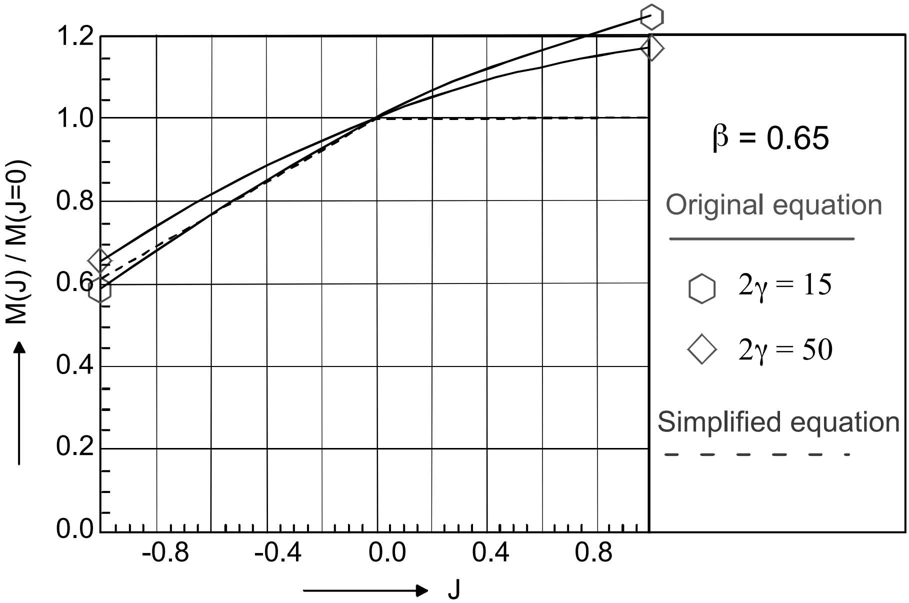



Figure 6.16 – Multiplanar effect of I-beam-to-CHS column connections loaded by in-plane-bending moments (β = 0.65)

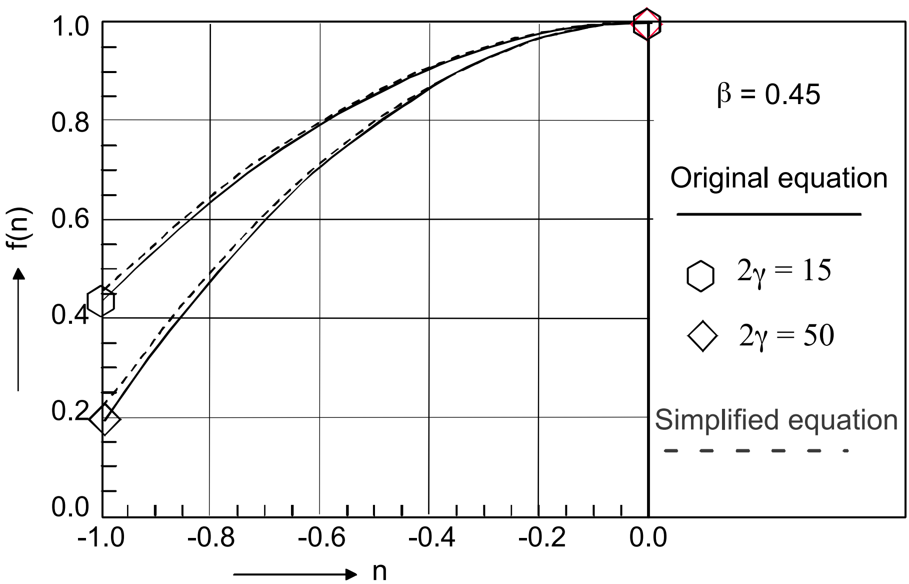



Figure 6.17 – Influence of column stress on I-beam-to-CHS column connections loaded by in-plane-bending moments (β = 0.45)

### 6.3.2 I-beam-to-RHS column connections

Initial investigations on I-beam-to-RHS connections have been carried out by Kanatani et al. (1980) and further flange plate to RHS connections have been investigated by Wardenier (1982) and Davies and Packer (1982). For I-beam-to-RHS column connections, Lu (1997) has recently carried out similar investigations as de Winkel (1998) did for I-beam to-CHS column connections.

For consistency the formulae given here are in principle consistent with those given in the CIDECT Design Guide No. 3 (Packer et al., 1992) and in Eurocode 3 Annex K (CEN 1992). However, they have been checked with recent research and extended where required.

Comparison of the design strength formulae for the various failure criteria of flange plate-to-RHS column connections (Wardenier 1982, Packer et al. 1992) shows that for $t_p \leq t_c$ the plate effective width criterion (equation 6.20) generally is critical compared to punching shear, column face plastification and column side wall failure (see figure 6.18).

$$N_p^* = f_{p,y}\, t_p\, b_e \qquad \text{.......... 6.19}$$

with

$$b_e = \frac{10}{b_c/t_c} \cdot \frac{f_{c,y} \cdot t_c}{f_{p,y} \cdot t_p} \cdot b_p \leq b_p \qquad \text{.......... 6.20}$$

The moment capacity for I-beam-to-RHS column connections follows then by multiplying the flange plate connection strength $N_p^* = N_{b,f}^*$ for axial loading with the depth $(h_b - t_{b,f})$.

$$M_b^* = N_{b,f}^*(h_b - t_{b,f}) \qquad \text{.......... 6.21}$$

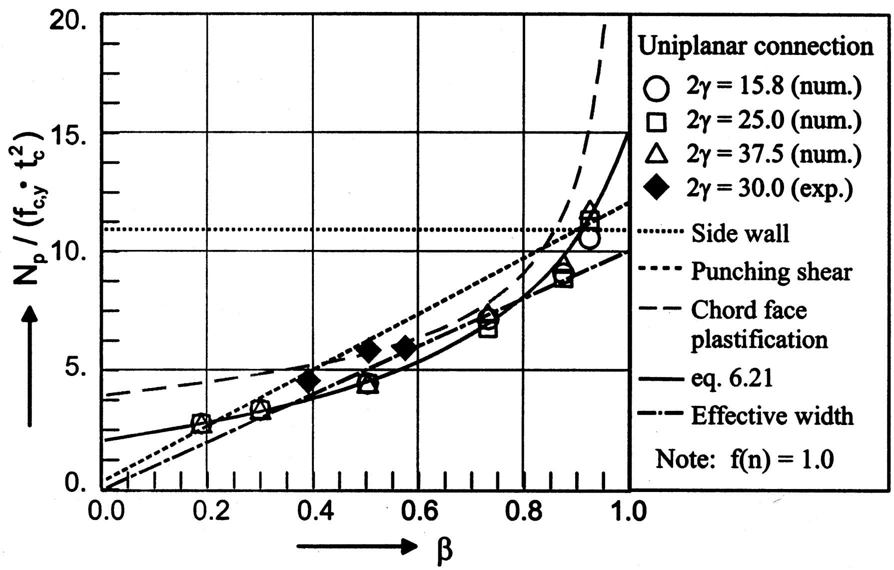



Figure 6.18 – Comparison of the FE results based on the 3% of the column width $b_c$ deformation criterion and the column face yield line criterion, the punching shear criterion, the plate effective width and column side wall criterion (Lu, 1997)

However, for highly loaded columns the design strength for column face plastification is reduced by a function f(n) which may result in this criterion governing. Further, Lu (1997) showed that the yield line mechanism for column face plastification occurs at deformations which exceed the deformation limit of 3% of the column width $b_c$. As a consequence she proposed based on the deformation limit of 3% of the column width $b_c$, the following modified criterion for column face plastification of I-beam-to-RHS column connections:

$$M_b^* = (0.5 + 0.7\beta) \frac{4 f_{c,y} t_c^2}{(1 - 0.9\beta)^{0.5}} (h_b - t_{b,f}) f(n) \quad \text{.......} 6.22$$

with (not simplified) and based on the maximum column stress:

$$f(n) = 1 + 1.48 \cdot n \cdot (2\gamma)^{-0.33} - 0.46 \cdot n^{1.5} \cdot (2\gamma)^{(0.33 - 0.11\beta^2)} \quad \text{.......} 6.23$$

but $f(n) \leq 1.0$

The function f(n) is given in figure 6.19. Thus, the minimum moment resistance calculated using equations 6.19 to 6.23 governs. The criterion for column face plastification for the plate-to-RHS connection is not given in CIDECT Design Guide No. 3.

In the case of axial loads it should be noted that the axial load capacity of an I-beam-to-RHS column connection will only be two times the axial load capacity of one flange if $\eta > 2\sqrt{(1-\beta)}$, see Lu (1997).

In that case for the column face plastification criterion there is no interaction between one flange and the other (if two separate flanges would be present). Thus for values $\eta < 2$ several strength criteria may have to be considered.

From the work of Lu (1997) the following conclusions can be drawn:

– All multiplanar connections with a load ratio J = 0 and $\beta \leq 0.73$ behave like uniplanar connections. It should be noted that for higher $\beta$ ratios up to 1.0, a positive geometrical effect is expected, in line with the findings of Yu (1997).

– As shown in figure 6.20 for the investigated width ratios $0.15 < \beta < 0.75$, negative load ratios (J < 0) decrease the connection capacity considerably, whereas positive load ratios (J > 0) generally have a small beneficial effect. Simplified, this effect (see fig. 6.20) is given by f(J) = 1 + 0.4J, but $\leq 1.0$. This lower bound can also be used for axially loaded I-beam- to-RHS connections. Based on the work of Yu (1997), it is expected that for $\beta$ ratios close to 1.0 that positive load ratios may also have a negative effect on the load, therefore the validity range is limited to $0.2 \leq \beta \leq 0.8$.

– The use of a steel plate floor does not enhance the connection capacity significantly if based on a load deformation of 3% $b_c$.

– Concrete filling of the RHS columns increases the stiffness and capacity considerably.

– The connection strength and stiffness can be significantly increased by using a composite floor. However, if the concrete reinforcement is governing for the connection capacity, the rotation capacity is small if low ductility reinforcement is used (see chapter 9).

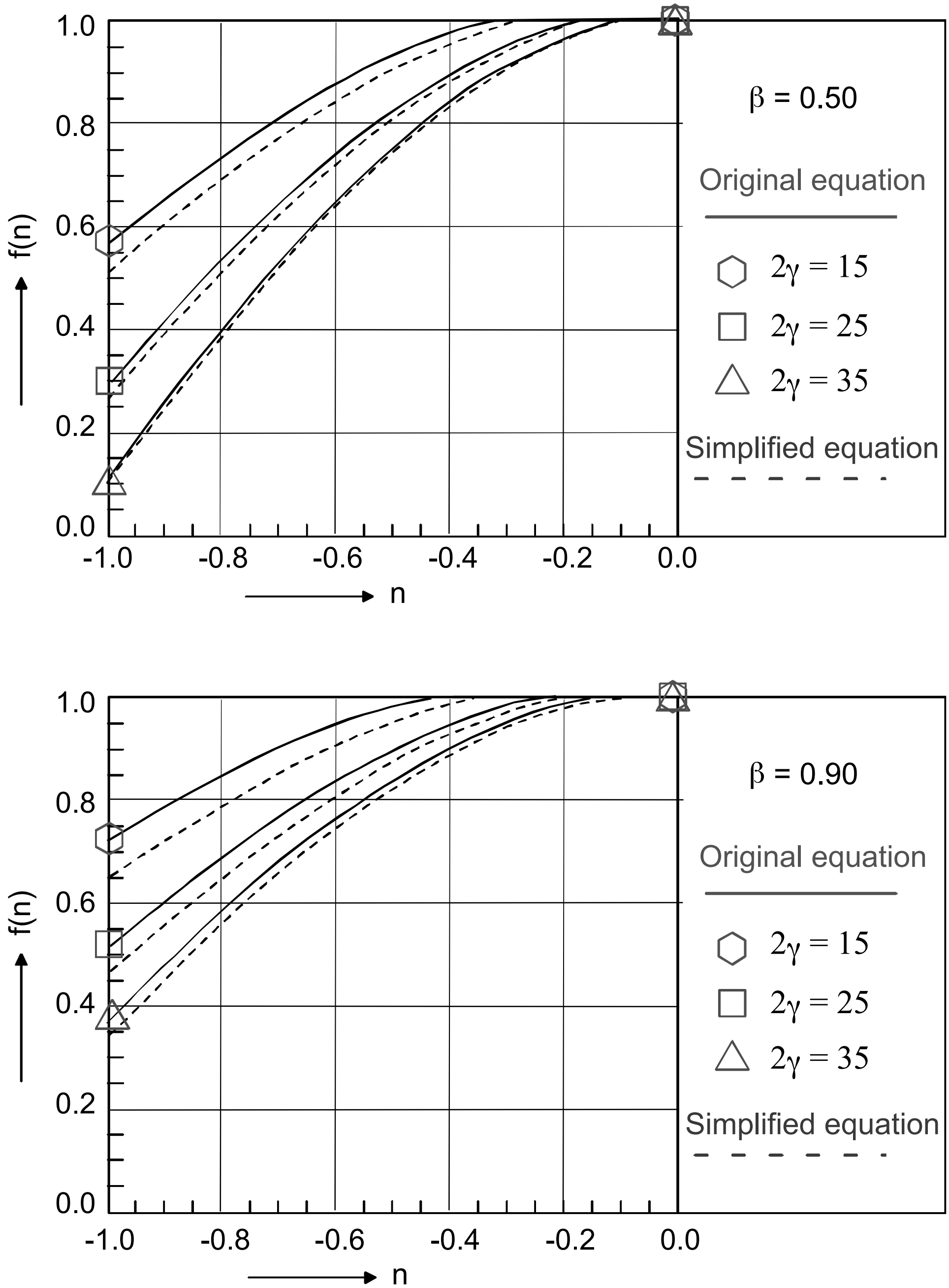



Figure 6.19 – Effect of column loading on the connection moment resistance

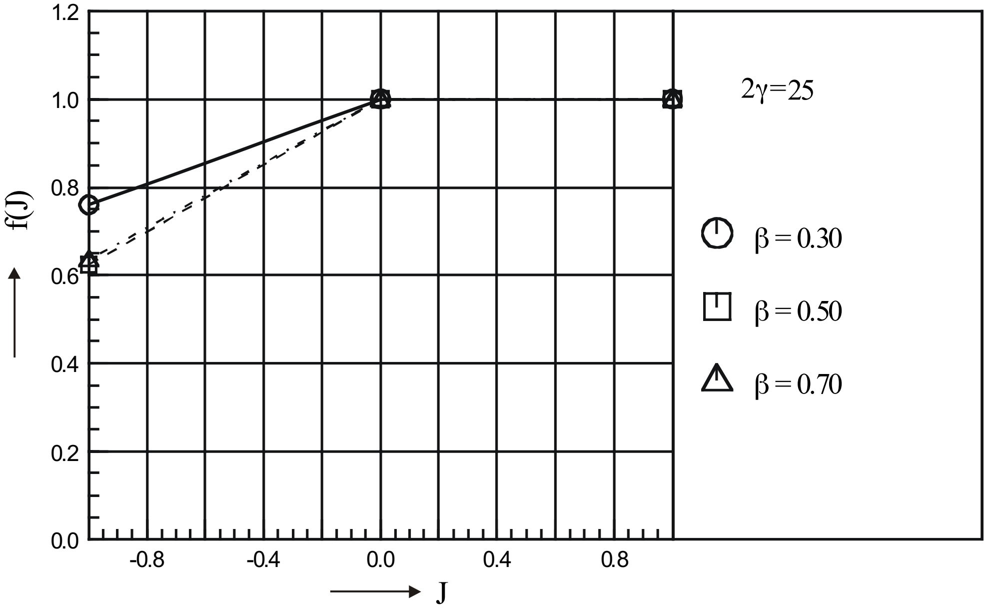



Figure 6.20 – Multiplanar loading effect for I-beam-to-RHS column moment connections

The previous mentioned strength criteria have been summarised in figure 6.21.

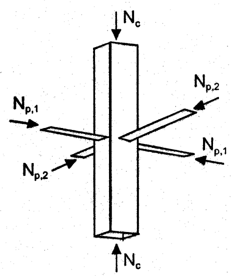
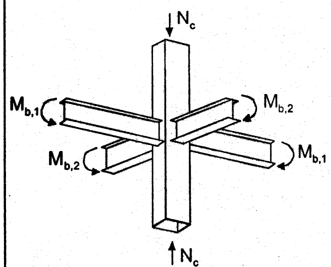
| Type of connection | Factored connection resistance |
|---|---|
| <br>$J = N_{b,2}/N_{b,1} \leq 1.0$ | $N_p^* = (0.5 + 0.7\beta) \cdot \frac{4}{\sqrt{1-0.9\beta}} \cdot f_{c,y} \cdot t_c^2 \cdot f(n) \cdot f(J)$<br>$N_p^* = f_{p,y} \cdot t_p \cdot b_e$ |
| <br>$J = M_{b,2}/M_{b,1} \leq 1.0$ | $M_b^* = (0.5 + 0.7\beta) \cdot \frac{4}{\sqrt{1-0.9\beta}} \cdot f_{c,y} \cdot t_c^2 \cdot (h_b - t_{b,f}) \cdot f(n) \cdot f(J)$<br>$M_b^* \leq f_{b,y} \cdot t_{b,f} \cdot b_e \cdot (h_b - t_b)$ |
| **General** | |
| Punching shear<br>Chord side wall crippling | Do not govern within validity range given |
| **Functions** | |
| $n = \frac{N_c}{A_c \cdot f_{c,y}} + \frac{M_c}{W_{c,el} \cdot f_{c,y}}$<br>$J = \frac{N_{p,2}}{N_{p,1}}$ or $\frac{M_{b,2}}{M_{b,1}}$ | $f(n) = 1 + 1.48n(2\gamma)^{-0.33} - 0.46n^{1.5}(2\gamma)^{(-.33-0.11\cdot\beta^2)}$<br>$\leq 1.0$<br>$f(J) = 1 + 0.4 \cdot J$ (lower bound)<br>$\leq 1.0$<br>$b_e = \frac{10}{b_c/t_c} \cdot \frac{f_{c,y} \cdot t_c}{f_{b,y} \cdot t_{b,f}} \cdot b_{b,f}$ but $\leq b_{b,f}$ |
| **Range of validity** | |
| $0.2 \leq \beta \leq 0.8$, $1.5 \leq 2\gamma \leq 37.5$, $0.3 \leq \eta \leq 2.0$ | |

Figure 6.21 - Design strength formulae for I-beam-to-RHS column connections

### 6.4 Bolted hollow section beam and column connections

Bolted connections between hollow section members can be made using flange plates, gusset plates, angles, cleats or cut-outs of open sections. Most bolted connections are designed as shear connections, tension loaded splices or stiffened moment connections.

#### 6.4.1. CHS beam-to-column connections

The only common types of bolted moment connections between CHS members are shown in figure 6.22.

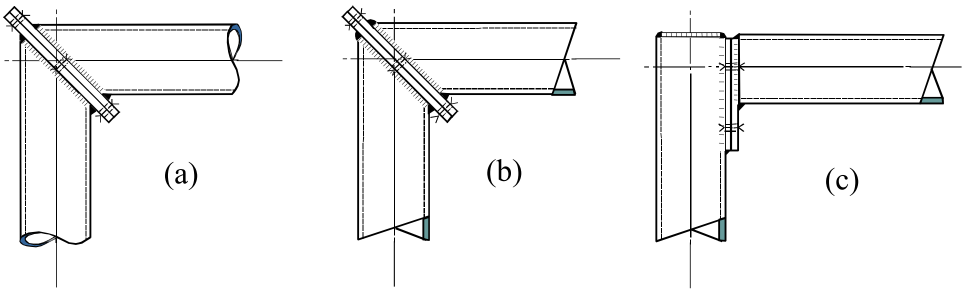



Figure 6.22 – Bolted knee assemblies of CHS or RHS members for portal frames

No detailed evidence is available for the stiffness of these connections in relation to the plate dimensions and the bolt locations. It is therefore recommended to use a plate thickness such that the connection can be assumed to be rigid. The bolts should preferably be designed for the moment capacity of the connected hollow section.

#### 6.4.2 RHS beam-to-column connections

For the bolted knee connections, shown in figure 6.22, the same remarks can be made as for the connections with CHS members. Welding a haunch between the bottom flange of the RHS beam and the flange plate stiffens the knee assembly of figure 6.22 (c). Another bolted assembly, which may be designed to transfer moments, is shown in figure 6.23.

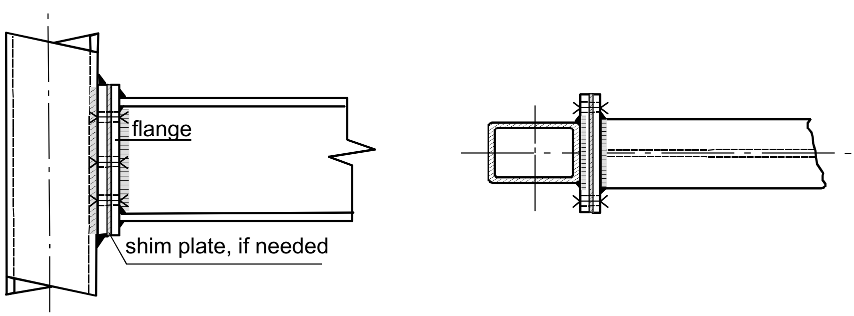



Figure 6.23 – Bolted flange plate connection between RHS members

## 6.5 Bolted I-beam-to-hollow section column connections

Bolted connections between I-beams and hollow section columns can be distinguished in connections with continuous beams as shown in figure 6.24, through plate connections, shown in figure 6.25, and connections with continuous columns, shown in figure 6.26.

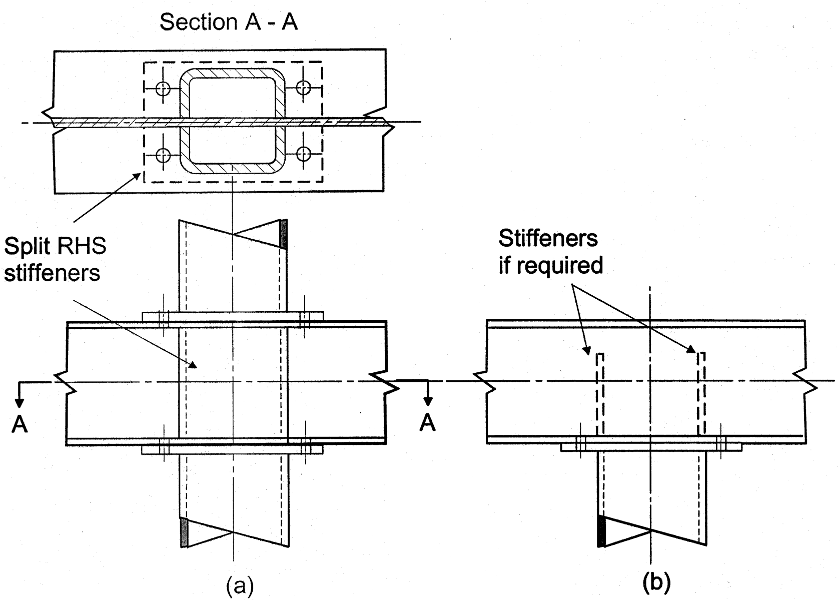



Figure 6.24 – Bolted continuous beam-to-column connections (Packer and Henderson, 1997)

The strength and stiffness of bolted connections with a continuous beam is directly dependent on the thickness of the cap plates, the reinforcement of the beam and the bolts. Since no evidence is available for the stiffness, it is recommended to design these connections as rigid moment connections with relatively thick cap plates.

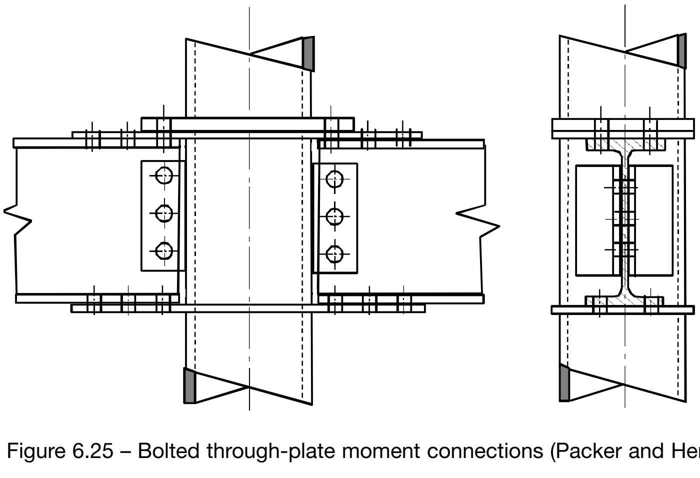


Figure 6.25 – Bolted through-plate moment connections (Packer and Henderson, 1997)

The through-plate connections of figure 6.25 allow a direct load transfer from beam to beam or to a column whereas the shear is transferred by shear tabs or angles welded to the column web. Here the flexibility for the beam connection depends mainly on the bolts loaded in shear (prestressed or not); for the top column connection with an interrupted column it is similar to that of the connections in figure 6.24.

The bolted connection in figure 6.26(a) is in principle a welded beam-to-column connection, as discussed in section 6.3.1, with a bolted splice. This type of connection is very common and easy to handle on site.

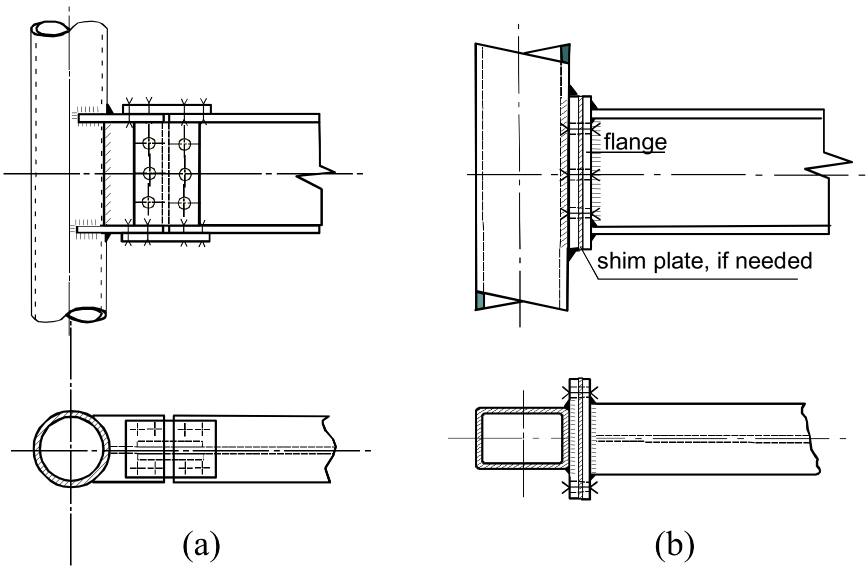



Figure 6.26 - Bolted beam-to-column connection with a continuous column

### 6.5.1 I-beam-to-CHS column connections

Most of the bolted moment connections between I-beams and CHS columns are stiffened with plates and can be designed as rigid connections. These are further dealt with in chapter 8.

### 6.5.2 I-beam-to-RHS column connections

Besides the bolted I-beam-to-RHS column connections with extended plates shown in figures 6.23 and 6.26(b), nowadays it is also possible to connect directly to the face of the RHS column. In this case *single sided* (also called *blind*) *bolting* is used. The systems currently used, i.e. the Flowdrill system, the HolloBolt and the Huck Ultra-Twist Bolt system, are described in chapter 3. Figure 6.27 shows two examples for moment connections: (a) with an extended end plate, and (b) with a flush end plate.

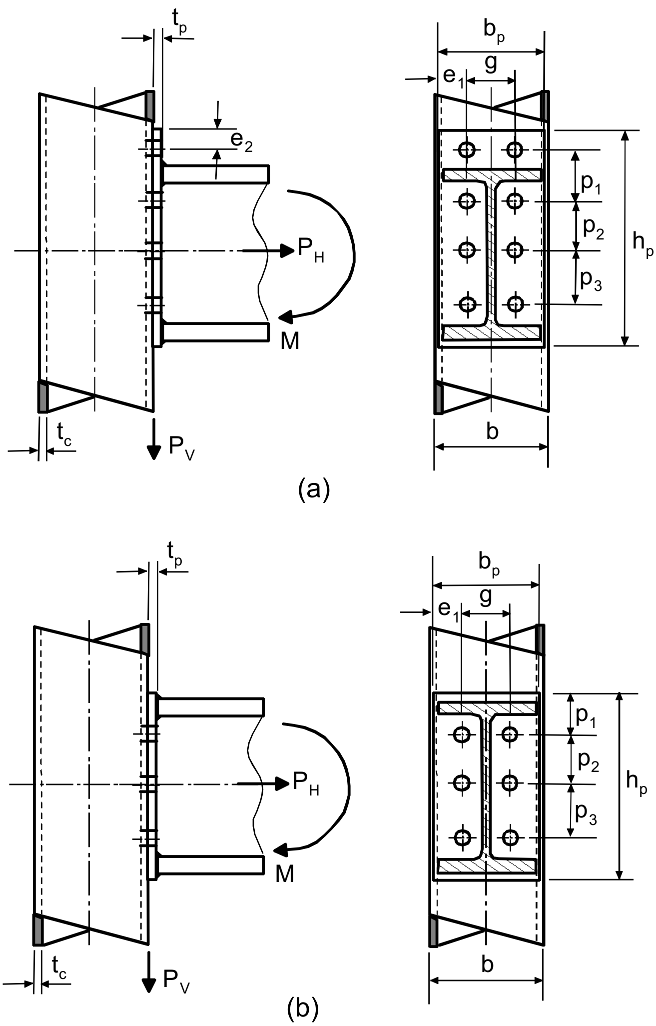



Figure 6.27 - Blind bolted I-beam-to RHS column connections

As discussed in chapter 3, these systems can be treated as normal bolted connections provided the limitations for bolt diameter in relation to the RHS column thickness are taken into account. However, for moment connections not only the deformation of the end plate has to be considered but also the flexible face of the RHS column.

Thus, the following criteria have to be considered (Yeomans 1996, 1996a):

- bolts (tension and shear and bolt bearing for plate and column face);
- stripping of the bolt threads;
- column face - punching shear of the bolt through the column face;
- column face plastification (yield line pattern);
- column side wall crippling;
- plate plastification (yield line pattern).

For the bolt design the following well known criteria can be used:

- bolt shear capacity;
- bolt bearing capacity;
- bolt tensile capacity;
- combination shear and tension.

Stripping of the threads of the bolts has to be checked for the bolts in flowdrilled connections. The bolt thread strip capacity is:

$F_{ts} = 0.6\ f_{c,y}\ \pi\ d_b\ (t_c + 8\ mm)$ ..............................6.24

For the punching shear criterion the diameter to be considered depends on the type of system being used. For example for punching shear of the flowdrilled extrusion from the RHS:

$F_{ps} = 0.6\ f_{c,y}\ \pi\ t_c\ (d_b + t_c)$ ..............................6.25

For a hollowbolt connection the punching shear capacity is just given by:

$F_{ps} = 0.6\ f_{c,y}\ \pi\ t_c\ d_b$ ..............................6.26

As shown in figure 6.28, the column face plastification criterion depends on the plate dimension in relation to the width of the RHS column. For small plate widths and stiff end plates the compression area will be pushed in and the tension area of the connection will be pulled out giving a yield line pattern in the column face (case a). However, for moment loaded connections it is recommended to have the plate width the same as the column width, which increases the stiffness and the moment capacity (case b). In this case a yield line pattern will be formed in the tensile area only if the crippling strength of the column walls is not governing. The column face plastification criterion for a bolt pattern with four bolts in tension given by Yeomans (1994, 1998) is:

$N_{pl} = f_{c,y}\ t_c^2\ [2(h_b - d_b)/b' + 4(1 - c/b')^{0.5}]/(1-c/b')\ f(n)$ ..............................6.27

where $b' = b_c - t_c$ and $c = g - d_b$

$f(n) = 1 + n \leq 1.0$ ..............................6.28

In those cases where the plate thickness is smaller, the plate has to be checked in a similar way as for beam-to-column connections of I-beams, i.e. considering the model shown in figure 6.29. Here the plate plastification (Zoetemeyer 1974, Eurocode 3-Annex J) has to be checked for complete end plate yielding (case c1) and end plate yielding with bolt failure (case c2) similar to column connections of I-beams. However, depending on the stiffness and thus the deformation of the column face the prying force action may be different.

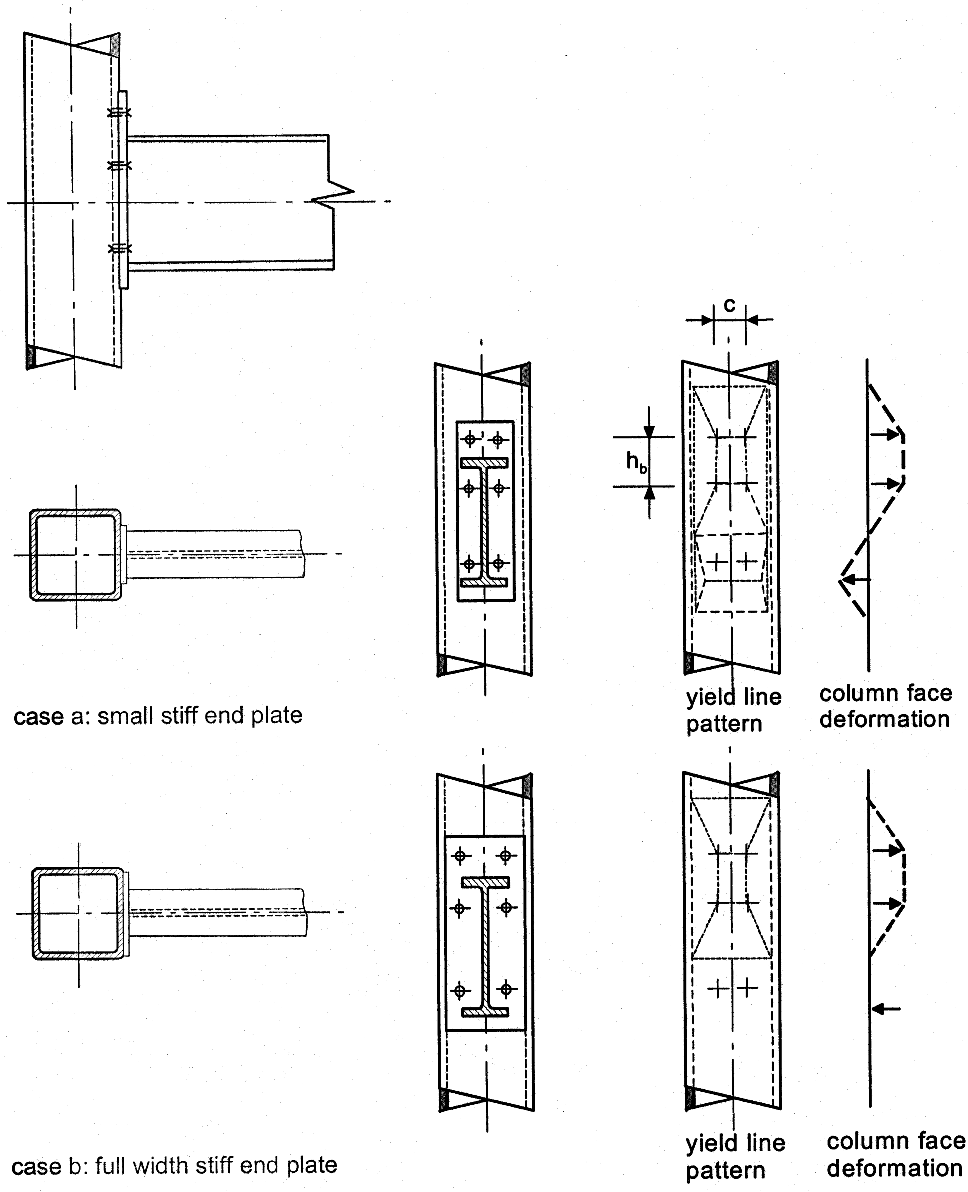



Figure 6.28 – Column face plastification yield line patterns

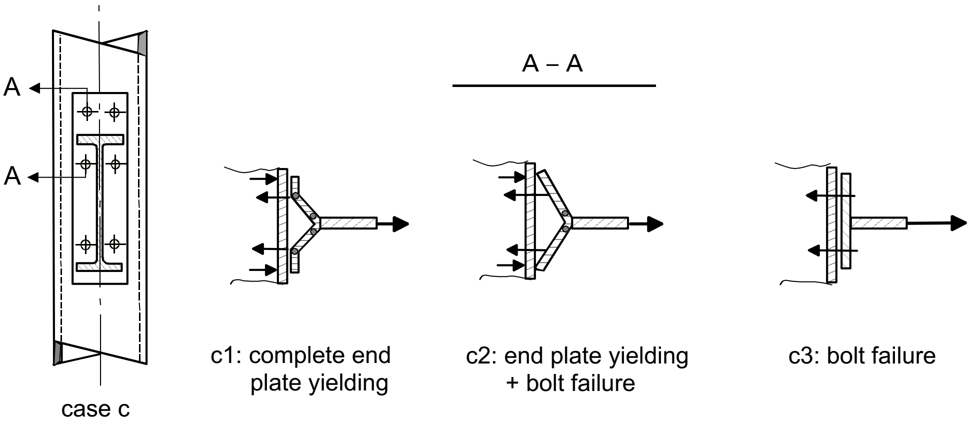



Figure 6.29 - Plate plastification models

The wall bearing or crippling capacity for columns with $b_c/t_c < 35$ can be given by:

$N_{c,w} = 2\ f_{c,y}\ t_c\ b_w$ ..........................................................................................................6.29

The bearing width $b_w$ can be taken as:

$b_{w} = (t_{b,f} + 2t_p + 5t_c)$ ..........................................................................................................6.30

Apart from the bolted part the welds connecting the end plate and the I-beam have to be checked. It should also be noted that for one sided connections the shear capacity of the column has to be checked.

The moment capacity of the connection follows by multiplying the minimum governing axial load capacity by the beam depth ($h_b$ - $t_{b,f}$).

At present no information in formulae or graphs is available about the stiffness, thus a real semi-rigid analysis is not yet possible for these connections.

## 6.6 Examples

In the design of semi-rigid connections the following procedures can be followed:

1. Assume rigid connections and after determination of the member sizes check if the connection stiffness meets the minimum stiffness requirement given in figure 4.7.

   a. If not, the joint parameters and thus the sections should be changed in such a way that the stiffness requirement is met, or

   b. the actual stiffness has to be used in the design calculations and it has to be checked whether the structure still meets the strength and stiffness requirements.

2. Assume pin-ended connections and after determination of the member sizes check if the connection stiffness does not exceed the maximum stiffness requirement given in figure 4.7 for a pin-ended condition.

a. If the requirement is not met, the joint parameters and thus the sections should be changed in such a way that the stiffness requirement is met, or

b. the actual stiffness has to be used in the design calculations and it has to be checked whether the strength and stiffness requirements are met.

3. Design the frame based on a rigid plastic frame analysis and check if the stiffness of the connections and the rotation capacity allow the assumed redistribution of moments.

4. If the designer has knowledge about the connection parameters it is also possible to determine the associated connection stiffness and use it in the design; after determination of the final member sizes it should be checked that the actual connection stiffness does not deviate too much from the assumed stiffness. If so, the design is O.K., otherwise the analyses should be done again with the actual connection stiffness.

Method 1 is especially appropriate for connections with a large stiffness e.g. with a low $2\gamma$ ratio, a large $\beta$ ratio and/or a low $\eta$ ratio.

Method 2 is more appropriate for connections with a low stiffness, e.g. with a high $2\gamma$ ratio, a low $\beta$ ratio and/or a low $\eta$ ratio.

Method 3 is a very simple approach for those cases where the stiffness of the connections is less important, e.g. braced frames.

**Example 1: CHS beams and columns**

Figure 6.30 gives a braced frame of CHS members for which the connections between the circular hollow sections have been assumed to be rigid.

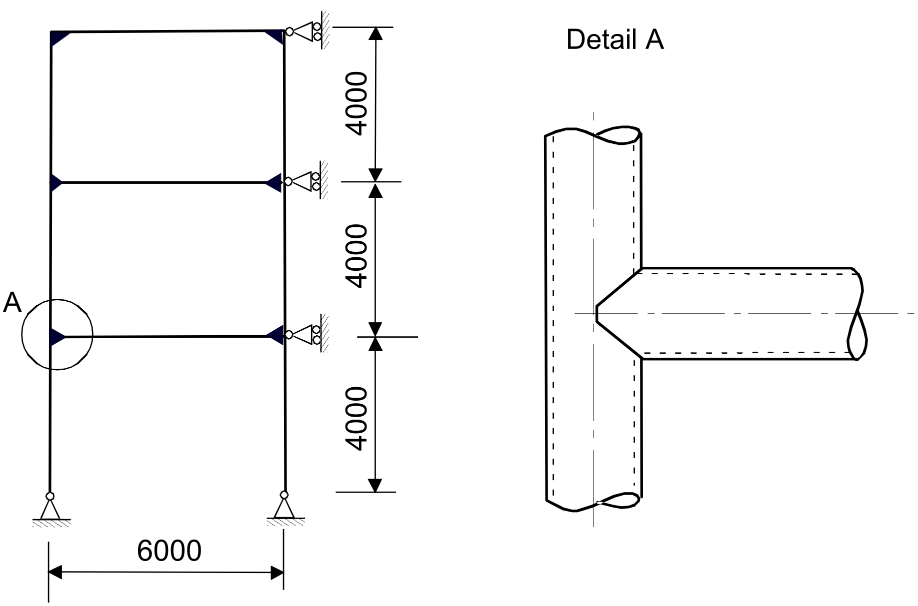



Figure 6.30 – Frame of CHS beams and columns

The steel grade is S355 with a yield stress $f_y = 355$ N/mm$^2$.

Assume that based on the frame analysis with rigid connections the following sections have been selected:

columns 298.5 x 10 : beams 298.5 x 6.3

*(Note: These sections are not available at every tube supplier.)*

***Check if the stiffness is sufficient to assume a rigid connection.***

Eurocode 3 (CEN1992) gives the following requirement for braced frames (see figure 4.7): $S_{j,ini} \geq 8EI_b/L_b$

For the beam 298.5 x 6.3 the following properties apply:

$I_b = 6175 \times 10^4$ mm$^4$, $W_{el} = 414 \times 10^3$ mm$^3$, $E = 2.1 \times 10^5$ N/mm$^2$, $L_b = 6000$ mm

Hence the beam stiffness is:

$EI_b/L_b = (2.1 \times 10^5)(6175 \times 10^4)/6000 = 2161 \times 10^6$ Nmm/rad = 2161 kNm/rad

And the required stiffness for braced frames is: 8 x 2161 = 17288 kNm/rad

The stiffness for connections between CHS members is given by equation 6.2; however, this equation is only valid for $0.3 \leq \beta \leq 0.8$.

If, however, the stiffness would be sufficient assuming $\beta = 0.8$, then it will also be sufficient for $\beta = 1.0$ because the stiffness increases with $\beta$.

$$C_{b,ip} = 1.3E \left[\frac{d_c}{2}\right]^3 \beta^{(2.25+\gamma/125)}\ \gamma^{-1.44}\ \frac{1}{\sin^{(\beta+0.4)}\theta}$$

$= (1.3)(2.1 \times 10^5)(3.32 \times 10^6)(0.589)(0.02) \times 1 = 10.89 \times 10^9$ Nmm/rad
$= 10890$ kNm/rad $< 17288$ kNm/rad

Thus the available stiffness for $\beta = 0.8$ is not sufficient to assume a rigid connection.

For example, it can be checked now if relevant test evidence is available.

Van der Vegte (1995) did numerical calculations for T-connections with $\beta = 1.0$ and loaded by bending in-plane, however the column diameter was 406.4 mm. Thus, the stiffness given by van der Vegte should be corrected for the influence of the column diameter, or the influence of the $\beta$ parameter between 0.8 and 1.0 has to be estimated.

In (van der Vegte, 1995) the stiffness can only be determined from the figures with test results. To avoid all kinds of recalculations to account for the different dimensions it is easier to determine the influence of the parameter $\beta$ between 0.8 and 1.0.

From the results it can be concluded that the stiffness for $\beta = 1.0$ is about 60% higher than for $\beta = 0.8$. Equation 6.2, which is graphically shown in Wardenier et al. (1991) and here in figure 6.31, gives for a conservative linear extrapolation from $\beta = 0.8$ to $\beta = 1.0$ an increase of 50 to 60%. Thus, an increase of 60% seems to be acceptable.

This results in a connection stiffness of: 1.6 x 10890 = 17424 kNm/rad.

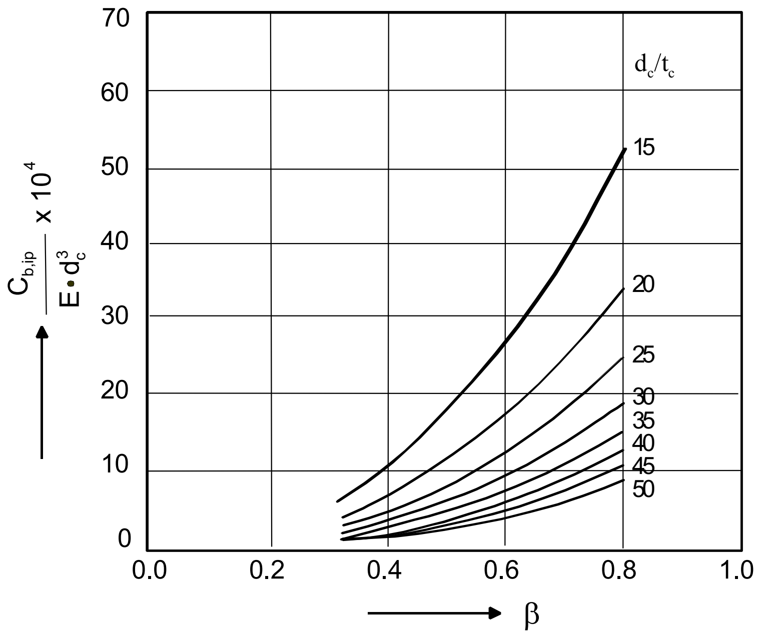



Figure 6.31 - Connection stiffness for T-connections between CHS members

Because 17424 > 17288, the connection may be assumed to be rigid for braced frames.

**Note 1:** Instead of doing all these exercises it could also have been checked with equation 4.9 if the initially calculated value of $C_{ip}$ = 10890 kNm/rad (for $\beta = 0.8$) would not result in a frame capacity which is more than 5% lower than intended. The limits given would result in $5793 \leq 10890 \leq 36782$ which means that the stiffness of 10890 kNm/rad would be acceptable. This also shows that even for large deviations in stiffness the influence on the frame capacity is small.

**Note 2:** If the frame in this example had been an unbraced frame the required stiffness would have been:

$S_{j,ini} \geq 25EI_b/L_b$ = 25 x 2161 = 54025 kNm/rad

Selecting sections with a 20% larger thickness and a 20% smaller diameter has, according to equation 6.2, the following effect on the stiffness:

$$(0.8)^3 \left[\frac{0.8}{1.2}\right]^{-1.44} = 0.92$$

The favourable effect of reducing the $\gamma$ ratio is compensated by the reduction in diameter and the resulting stiffness is nearly the same.

Keeping the member diameters the same and increasing the column thickness has a considerable effect on the connection stiffness. However, this would result in more material costs, thus from an economical point of view it is better here to adopt the previously determined stiffness in the frame analysis.

***Check of the connection capacity.***

The formulae for the connection capacity are given in figure 6.2. It is also possible to use the design graph of CIDECT Design Guide No. 1 (Wardenier et al., 1991) and here given in figure 6.32.

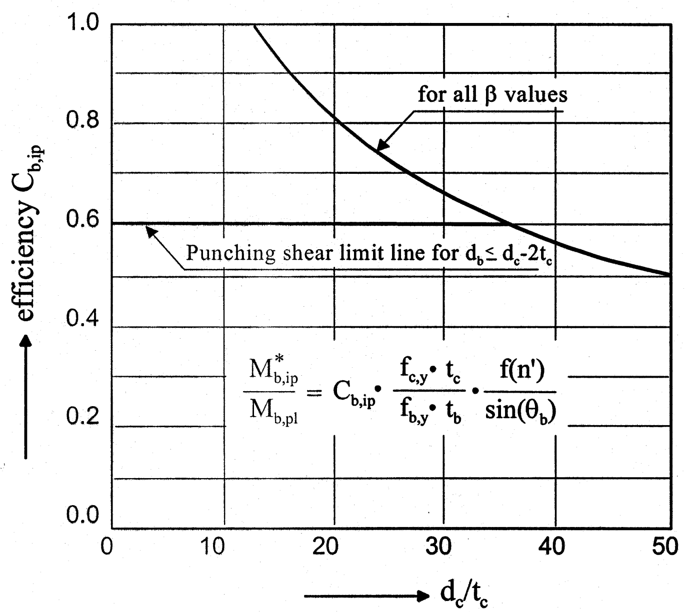



Figure 6.32 – Design graph for CHS connections loaded by in-plane bending moments

From the design graph:

for $\frac{d_c}{t_c} = 29.85$: $C_{b,ip} = 0.6$

for $\frac{f_{c,y} \cdot t_c}{f_{b,y} \cdot t_b} = 1.6$: $M^*_{b,ip} = 0.6 \cdot 1.6 \cdot f(n') \cdot M_{b,pl} = 0.96f(n') \cdot M_{b,pl}$

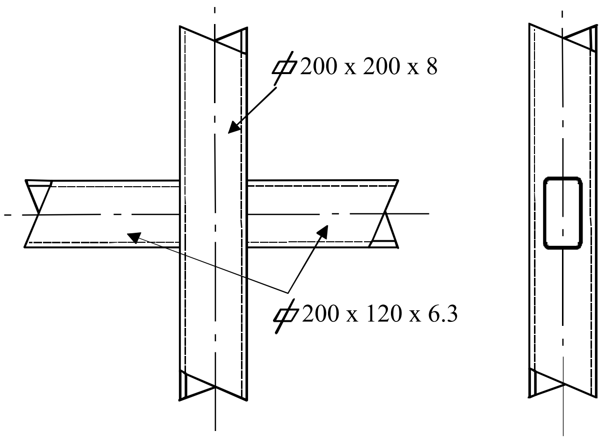
Suppose the compression stress in the column is 0.6 $f_{c,y}$ then n' = - 0.6 and with $f(n') = 1 + 0.3n' - 0.3(n')^2 = 0.71$

$M^*_{b,ip} = 0.68 \cdot M_{b,pl}$

Thus for braced frames this connection is a **rigid partial strength connection** and for unbraced frames it is a **semi-rigid partial strength connection**.

**Example 2: RHS beams and columns**

Figure 6.33 shows an X-connection of RHS beam and column members. For these connections no formulae are available for the determination of the stiffness. However, indications can be obtained from literature, e.g. Yu (1997). Compared to connections of CHS sections here the parameter $\eta = h_b/b_c$ also has to be considered because the sections can be rectangular instead of square.




column 200 x 200 x 8

beams 200 x 120 x 6.3

steel grade $f_y$ = 355 N/mm²

Figure 6.33 – Connection of RHS members

***Check if the connection can be assumed to be pin ended.***

Eurocode 3 (CEN 1992) states that the connection should be assumed to be pin-ended if the following requirement is satisfied (see figure 4.7):

$S_{j,\,ini} < 0.5EI_b/L_b$

For the beam 200 x 120 x 6.3 the following properties apply:

$I_b = 2065 \times 10^4$ mm⁴, $W_{b,el} = 207 \times 10^3$ mm³ , $W_{b,pl} = 253 \times 10^3$ mm³

$E = 2.1 \times 10^5$ N/mm², $L_b = 4000$ mm

Hence $0.5EI_b/L_b = 0.5(2.1 \times 10^5)(2065 \times 10^4)/4000 = 542 \times 10^6$ Nmm/rad

= 542 kNm/rad

For the connection stiffness the following connection parameters are important:

$\beta = 120/200 = 0.6$ ; $2\gamma = 200/8 = 25$ ; $\eta = 200/200 = 1.0$

In Yu (1997) a graphical presentation of test results (see figure 6.34) is given for this type of connection for:

$\beta = 0.6$ ; $2\gamma = 24$ ; $\eta = 2\beta = 1.2$

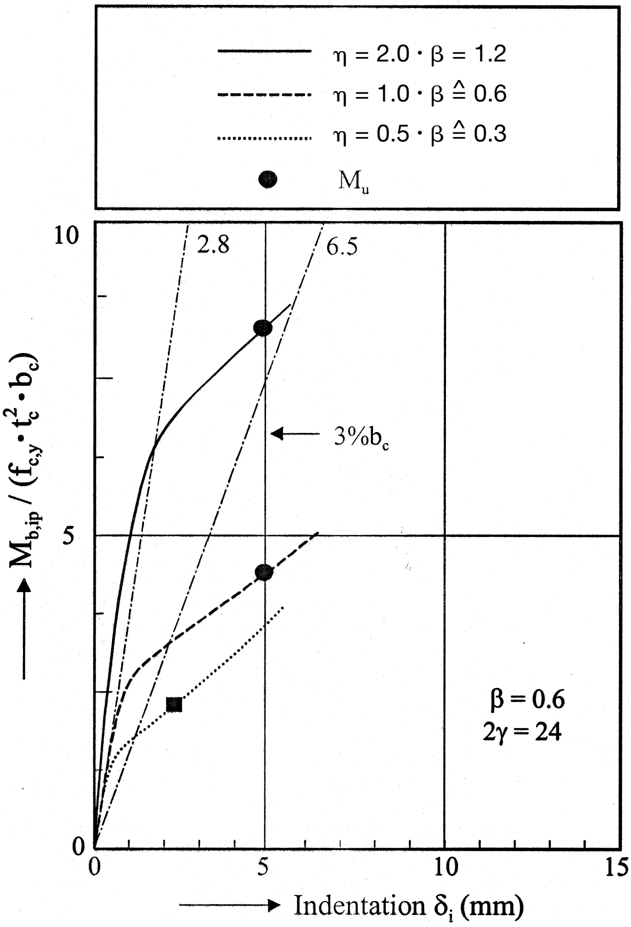



Figure 6.34 – Numerical results of Yu (1997) for X-connections loaded by in-plane bending moments

The parameters are nearly the same as for the connection being considered, only the dimensions in the tests were different, i.e. $b_c$ = 150 mm instead of 200 mm. Thus, the influence of $b_c$ has to be incorporated in the results of Yu and the effect of $\eta$ should be included by interpolation between $\eta = 0.6$ and $\eta = 1.2$.

As shown in the figure the moment rotation curve is strongly bi-linear. The initial stiffness $\phi_i$ is given by:

$$\phi_i = 2\delta_i/h_b$$

For $\eta = 1.67\beta = 1.0$ and for $M_{b,ip} = 10 \cdot f_{c,y} \cdot t_c^2 \cdot b_c$ : $\delta_i = 2.8$ mm thus, $\phi_i = 2 \times 2.8/150 = 0.037$ for

$M_{b,ip} = 10 \times 355 \times (6.25)^2 \times 150 = 20.8 \times 10^6$ Nmm = 20.8 kNm

$C_{b,ip} = M_{b,ip}/\phi_I = 562$ kNm/rad

The local indentation is a function of $b_c^4$; for the rotation the indentation is divided by the beam depth, which is related to $b_c$, thus the rotation is a function of $b_c^3$.

Consequently the initial stiffness is given by:

$$C_{b,ip} = 562\left[\frac{200}{150}\right]^3 = 1332 \text{ kNm/rad} > 542 \text{ kNm/rad}$$

Thus, it can be concluded that according to Eurocode 3 the connections (considering the 3% $b_c$ limit) cannot be assumed to be pin-ended.

Considering the secant stiffness at the moment the capacity of the connection (3% $b_c$) is reached, gives:

$\delta_i = 6.5$ mm instead of 2.8 mm for $\eta = 1.67\beta = 1.0$ and for $M_{b,ip} = 10\, f_{c,y}\, t_c^2\, b_c$

consequently the rotational stiffness drops to:

$$\frac{2.8}{6.5} \times 1332 = 573 \text{ kN/rad} > 542 \text{ kNm/rad}$$

This is marginally higher than the limit, and based on stiffness the connection could be classified as semi-rigid.

> ***Note:*** *However, if the connection has sufficient rotation capacity, there is no problem to assume a pin-ended connection.*

***Check of the connection capacity***

The formulae for the connection capacity are given in figure 6.5.

$$M^*_{b,ip} = 355 \times 8^2 \times 200 \left[\frac{1}{2\,\frac{200}{200}} + \frac{2}{(1-0.6)^{0.5}} + \frac{\frac{200}{200}}{(1-0.6)}\right] f(n) = 28 \times 10^6 f(n) \text{ Nmm}$$

$$= 28 \times f(n) \text{ kNm}$$

The beam capacity is:

$M_{b,pl} = (253 \times 10^3)(355 \times 10^{-6}) = 89.8$ kNm >> $28 \times f(n)$ kNm

*(**Note:** both sections can be classified as class 1 sections)*

Thus, this connection should be classified as a **semi-rigid partial strength** connection.

## Example 3: I-beams and CHS columns

Figure 6.35 gives a frame with I-beams at both sides welded to CHS columns.

Columns: 273 x 6 beams: IPE 360

For the IPE 360 beam the following properties apply:

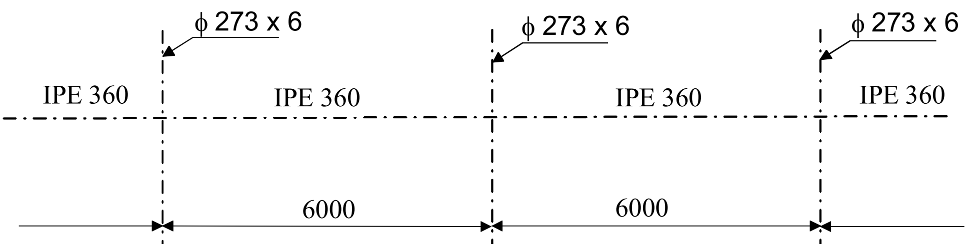
$I_b$ = 16270 x $10^4$ $mm^4$ ; $W_{b,el}$ = 904 x $10^3$ $mm^3$ ; $W_{b,pl}$ = 1020 x $10^3$ $mm^3$

E = 2.1 x $10^5$ $N/mm^2$ ; $L_b$ = 6000 mm

The steel grade of the beams and columns is S235 with a specified minimum yield stress $f_y$ = 235 $N/mm^2$.

The I-beam has originally been designed for a uniform distributed loading assuming pin-ended connections.

***Check whether based on a rigid plastic analysis the loading on the IPE 360 beams can be increased.***



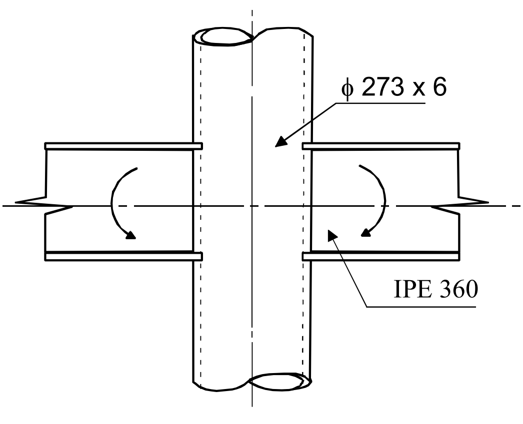




Figure 6.35 – Welded connection between the IPE 360 and the CHS 273 x 6

For the connections with the columns 273 x 6 the connection parameters are:

$\beta$ = 170/273 = 0.62 ; $2\gamma$ = 273/6 = 45.5

***Check of the connection capacity***

The connection moment capacity is given in figure 6.11.

$$M^*_{b,ip} = 360 \frac{5}{1-0.81\beta} (1 + 0.25 \frac{360}{273}) \times 235 \times 6^2 \times f(n') = 40.675 \times 10^6 f(n') \text{ Nmm}$$
$$= 40.7 f(n') \text{ kNm}$$

Suppose that $n' = -0.7$, then according to figure 6.11 : $f(n') = 1 + 0.3 n' - 0.3n'^2 = 0.64$

(N.B.: according to equation 6.18 and assuming that $n \simeq n'$ :
$f(n) = 1 - 0.25n^2 \times 45.5^{0.3} = 1 - 0.79 n^2 = 0.61$)
Thus $M^*_{b,ip} = 40.7 \times 0.61 = 26$ kNm.

Check punching shear.

$f_b t_{b,f} \leq 1.16 f_{c,y} \cdot t_c$ or $f_b \leq 1.16 (6/12.7) \cdot f_{c,y}$
hence:
$f_b \leq 0.55 f_{c,y}$ and $f_b \leq 129$ N/mm$^2$.
$M^*_{b,ip} = W_{b,el} \times f_b = (904 \times 10^3) \times 129 = 116.4 \times 10^6$ Nmm = 116.4kNm > 26 kNm.

Thus punching shear is not governing.

The plastic moment capacity of the IPE 360 is:

$W_{b,pl} \cdot f_{b,y} = (1020 \times 10^3) \times (0.235) = 240 \times 10^3$ kNmm = 240 kNm.

This means that the total capacity of the beam and the connection is:

$M^*_{b,ip} + W_{b,pl} f_{b,y} = 26 + 240 = 266$ kNm,

and the connection capacity gives in this case an increase of only 10.6% in the total capacity provided the requirements regarding stiffness and/or rotation capacity are satisfied.

***Check if the connection stiffness is sufficient to reach the connection moment capacity before the beam reaches at the centre the rotation capacity.***

The connection stiffness is given by equations 6.13 and 6.16:

$C_{b,ip} = 6.8 E t_c \beta (2\gamma)^{-1.3} (h_b-t_b)^2/2$
$C_{b,ip} = 6.8 (2.1 \times 10^5) (6) (0.62) (45.5)^{-1.3} (360 - 12.7)^2/2 = 2240 \times 10^6$ Nmm/rad
= 2240 kNm/rad

The limit for assuming a pin-ended condition is (see figure 4.7):

$S_{j,ini} < 0.5EI_b/L_b$
$0.5EI_b/L_b = 0.5 (2.1 \times 10^5) (16.270\times10^4) /6000 = 2847 \times 10^6$ Nmm/rad
= 2847 kNm/rad
$C_{b,ip} = 2240 < 2847$, thus the original assumption of a pin-ended connection is correct.

With this low connection stiffness the plastic moment capacity at the centre of the beam will be reached first; thus it should be checked whether the connection capacity is reached before the beam reaches the rotation capacity.

Suppose the beam has a rotation capacity R = 3, which means that according to the definition of R the beam can rotate over (R + 1) $\phi_{pl} = 4\phi_{pl}$.

If it is assumed that in the plastic hinge, when the plastic moment is reached, yielding takes place in the outer beam fibres at both sides of the hinge over a distance equal to the beam depth, thus over a total distance of $2h_b$, then the elongation $\Delta L$ in the outer fibres is equal to $2h_b \cdot \epsilon_y$.

The rotation in the plastic hinge of the beam is then:

$$\phi = 4 \frac{\Delta L}{0.5h_b} = 4 \frac{2h_b \cdot \epsilon_y}{0.5h_b} = 16 \frac{f_{b,y}}{E} = 18 \times 10^{-3} \text{ rad}$$

Assuming a rigid plastic situation, thus neglecting the elastic part, the connection will rotate over:

$0.5\phi = 9 \times 10^{-3}$ rad.

For this rotation the connection moment resistance will be:

$0.5\phi \cdot C_{b,ip} = (9 \times 10^{-3})\ 2240 = 20.2$ kNm

This is smaller than the connection moment capacity $M^*_{b,ip} = 26$ kNm at the ultimate limit state and the actual increase in capacity by including the connection capacity is 20.2 kNm resulting in a total capacity of the connection and the beam:

20.2 + 240 = 260.2 kNm

***Note:*** *In this example the column rotation has been neglected.*

### Example 4: Bolted I-beam-to-RHS column connection

A flowdrilled connection is considered for various I-beams and RHS columns.

For these connections the stiffness is not only influenced by the stiffness of the column face but also by the geometry of the end plate. The simplest way is to design the connections to be pin-ended, which can for example be achieved by using partial depth and relatively thin end plates for the beams. Otherwise the stiffness has to be obtained from the tests in literature or numerical calculations have to be carried out. As an example, some tests carried out by France et al. (1999, 1999a) are shown in figure 6.36.

The columns were 200 x 200 with the thickness varying between 6.3 and 12.5 mm. The steel grade used was S275, however the actual column yield stresses varied from 300 to 340 N/mm$^2$.

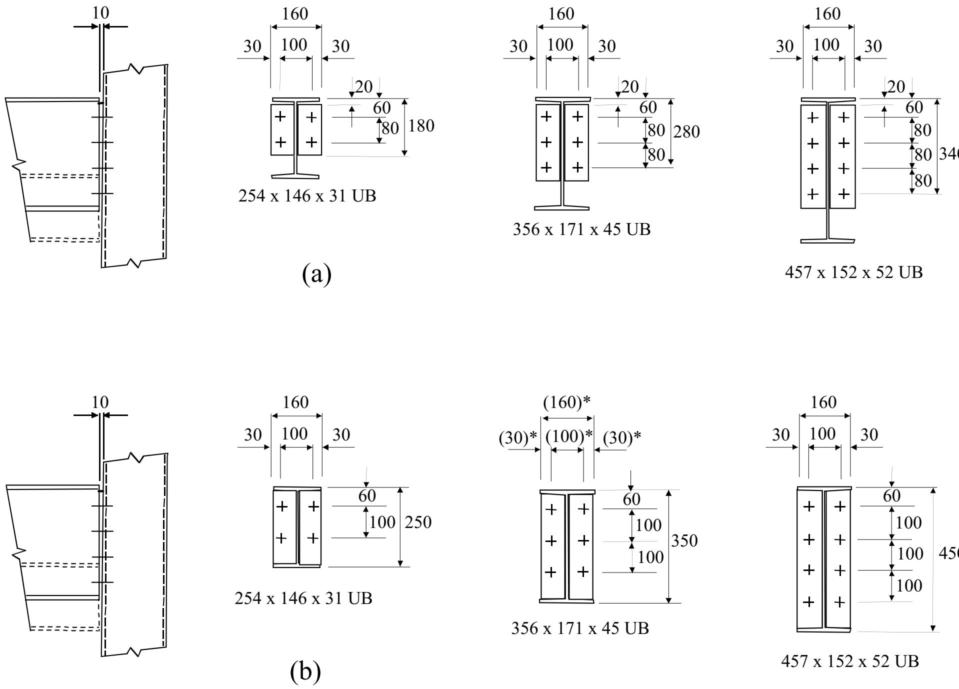



Figure 6.36 – I-beam-to-RHS column connection with partial depth and flush endplates (France et al.1999, 1999a)

Figure 6.37 shows a comparison between the moment rotation curves of a flush and a partial depth endplate for a connection between a 457 x 152 x 52 UB section and a 200 x 200 x 8 RHS column. Also, the pin-end requirement for a span of 7.5 m is indicated. The flush end plate connection is classified as semi-rigid and the partial depth end plate connection as pin-ended.

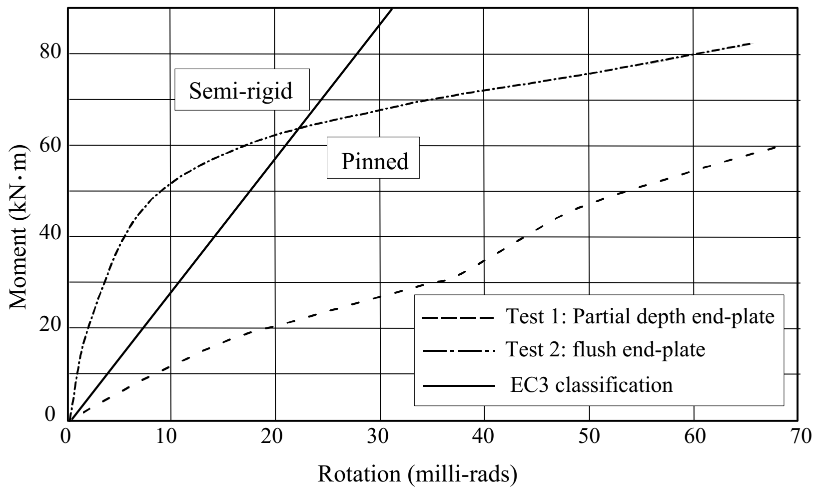



Figure 6.37 – Moment rotation curves of a flush and a partial depth endplate connection

From this investigation it can also be concluded that if the flush endplate thickness is about 1.5 times the thickness of the RHS section it only marginally contributes to the deformation of the connection; i.e. the deformation of the RHS face is most important.

The influence of the RHS thickness is shown in figure 6.38. Thus, in a similar manner to example 2, use can be made of available test data for design.

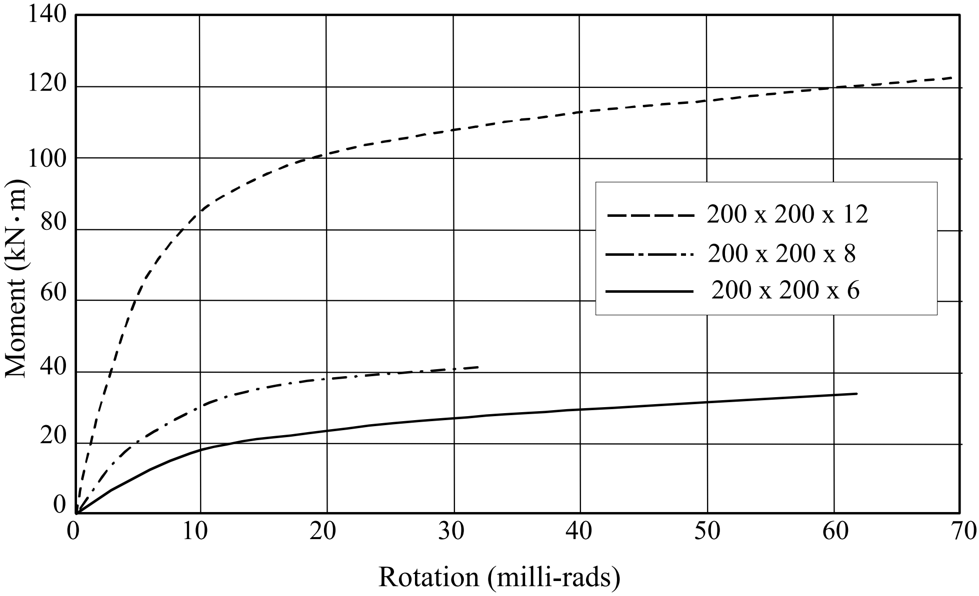



Figure 6.38 – Moment-rotation curves of flush endplate connections between a 356 UB and RHS columns with varying thickness

# 7 Special requirements for seismic loading

The seismic load varies with the energy dissipation (or absorption) capacity of the structure. This fact is explained by simple relationships between the response shear load V (response acceleration multiplied by mass) and response displacement δ when a structure is subjected to an impulsive load at its base as shown in figure 7.1. If the structure behaves elastically, the structure sustains the shear load $V_{el}$ and the displacement $\delta_{el}$. If, however, the structure yields and achieves the ultimate shear capacity at $V_u$, the structure sustains displacement $\delta_u$. The energies dissipated by the two structures, namely the areas under the two load-displacement curves, are roughly the same.

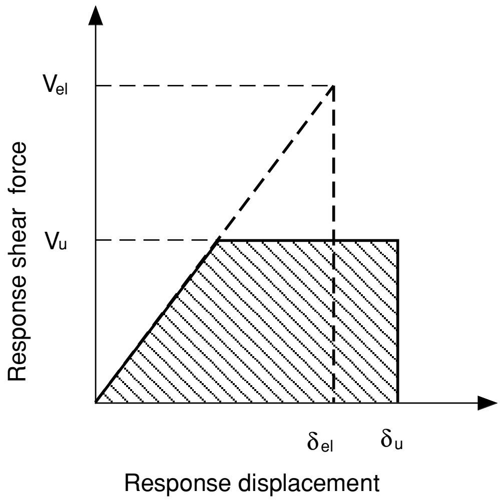



Figure 7.1 – Load displacement relationships of elastic and inelastic structure under impulsive loading

The above fact makes seismic design different from ordinary ultimate limit state design which considers other loads like gravity loads or wind force. If a wind load exceeds the capacity $V_u$, the structure collapses. But this is not the case with seismic load. Instead, seismic design requires that the structure should not collapse even if the maximum displacement reaches $\delta_u$.

Thus, the existing seismic codes specify design earthquake loads as a function of the energy dissipation capacity of structures. Specifically, the behaviour factor q in Eurocode 8 (CEN 1994), the reduction factor R in Uniform Building Code (ICBO1997) and the structural characteristics factor $D_s$ in the Japanese Building Code (BCJ 1997) similarly play the role of reducing the elastic response spectrum to obtain the design response spectrum, taking into account the different dissipation characteristics of the various types of structures. Furthermore, all these codes specify detailing rules for structural elements and frames to ensure that the structure can dissipate a certain amount of energy corresponding to the reduction factor.

Another point that makes seismic design different from ordinary ultimate limit state design is the fact that the impulsive load is applied not once but cyclically, although the number of cycles of major impulses is very small, say 2 or 3 cycles. Nevertheless, portions of the structure are strained well into the strain-hardening region cyclically. This cyclic cold working quickly deteriorates material toughness, which may cause a non-ductile tensile failure of structural elements, frequently starting from critical points in welded connections. Non-ductile failures are undesirable and should be avoided. If the structure is designed to remain nearly elastic, even under rare intense earthquakes, the material deterioration due to cold working is avoided. However, keeping an ordinary building structure nearly elastic to provide for the probability of such a rare occurrence is grossly uneconomical and not usually attempted unless the structure is isolated from ground shaking by using special devices.

The following part of this chapter discusses special requirements for the earthquake-resistant design of beam-to-column connections, additional to those required for ordinary ultimate limit state design. The descriptions generally follow the Eurocode 8 format, although design procedures adopted in Eurocode 8 are similar to those adopted in the other codes like the Uniform Building Code or the Japanese Building Code (the Building Standard Law of Japan and its subsidiary laws and regulations issued by the Ministry of Land, Infrastructure and Transportation). Hereafter, these are referred to as the Japanese Building Code.

### 7.1 Dissipative and non-dissipative structural behaviours

Eurocode 8 recommends the following two design concepts:

a) dissipative structural behaviour;

b) non-dissipative structural behaviour;

In concept a) the capability of parts of the structure (called dissipative zones) to resist earthquake loads beyond their elastic region is taken into account. Members and joints in dissipative zones sustain yielding or local buckling and participate in dissipating input energy during earthquakes by hysteretic behaviour. When assuming design earthquake loads, the behaviour factor q is taken greater than 1.0 in accordance with the energy dissipation capacity of the structure. Values of the behaviour factor are referred to later in section 7.3.

In concept b) a frame analysis is based on an elastic analysis without taking into account non-linear material behaviour. When assuming design earthquake loads, the behaviour factor (namely the reduction factor) q is taken as 1.0. For structures designed using concept b) the resistance of members and connections can be evaluated in accordance with the rules presented in Eurocode 3 (CEN 1992), without having to satisfy the ductility requirements presented in this chapter. The design concept b) may be used for structures in low seismicity zones, slender trussed structures or isolated structures and will not be discussed any more in this chapter.

### 7.2 Materials

The 1994 Northridge and 1995 Kobe earthquakes both took structural engineering professionals by surprise in that many of the welded connections in modern steel building frames sustained brittle fractures. These fractures most frequently occurred in regions around beam bottom flange groove welds. Especially in Northridge, brittle fractures initiated at a very low level of plastic demand and, in some cases, while structures remained elastic. Low toughness of weld metal produced by electrodes designated as AWS E70T-4 and by high deposition rate welding procedures was found to have played an important role in inducing brittle fractures (Fisher 1997). The recent trend is for seismic codes to impose more stringent toughness requirements on steel to be used in dissipative zones.

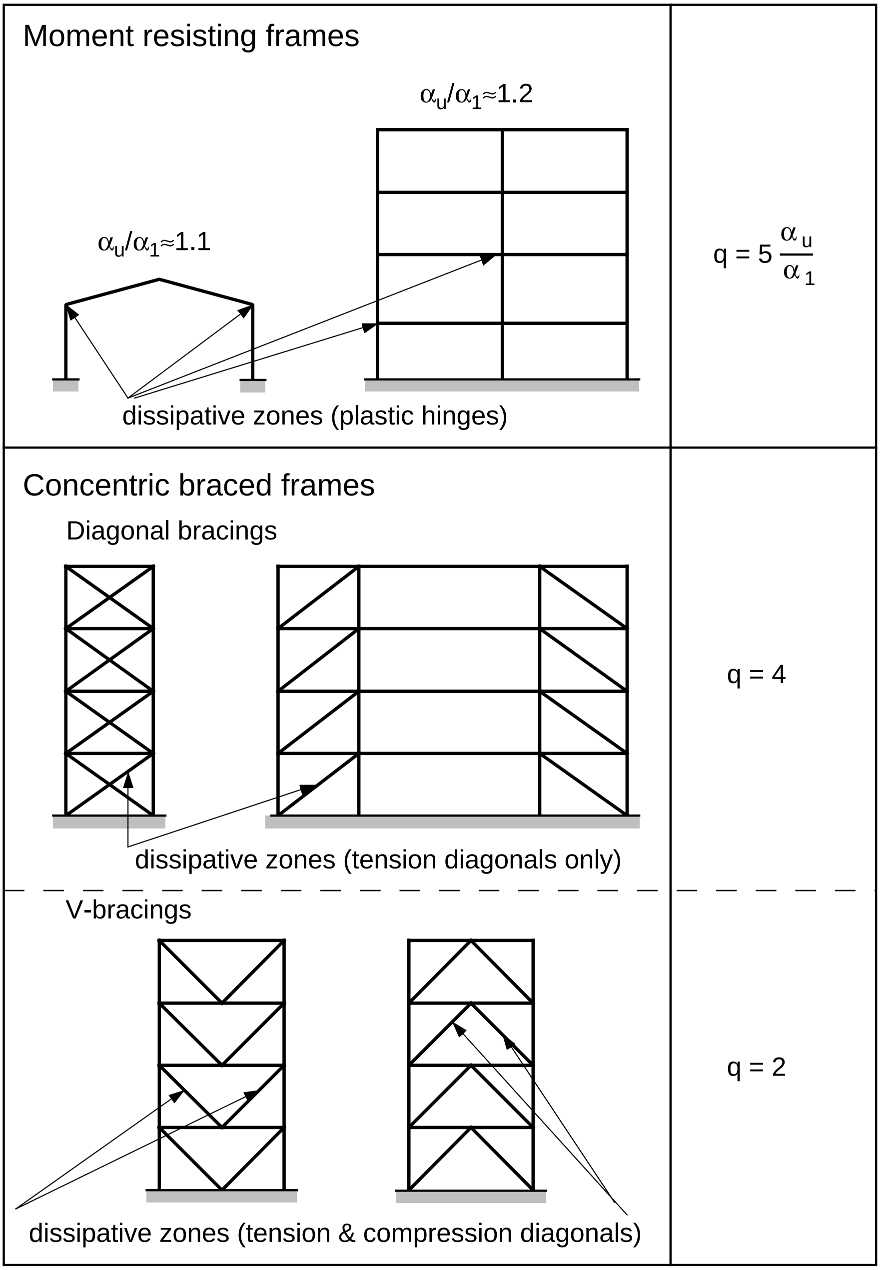
Eurocode 8 specifies that steel in dissipative zones should conform to EN 10025 (CEN 1993). The minimum required Charpy V Notch (CVN) toughness varies from 27 Joules at 20 °C to 40 Joules at – 20 °C depending on the grade of steel. The IIW and CIDECT recommendations for fatigue design of hollow section connections (IIW 1999, Zhao et al. 2000), recently revised, specify ISO 630 steel as well as hot-finished and cold-formed hollow sections designated as EN 10210-1(HF) and EN 10219-1(CF) (CEN 1994a, 1997a), which both require the same minimum CVN toughness as EN 10025, see the CIDECT design guide on fabrication, assembly and erection for more details (Dutta et al. 1998). The FEMA design criteria (2000) are recommending 27 J at 21 °C for base metal and 27 J at – 29 °C and 54 J at 21 °C for weld filler metal as the minimum required CVN values. It is also worth noting the fact that most of the experimental investigations, which formed the basis for these recommendations, used materials that showed toughness properties much superior to the minimum requirement of 27 J at 0 °C currently recommended by Japanese building authorities.

The other important mechanical properties required for dissipative zones are yield strength and yield/tensile strength ratio. These requirements are to ensure that the structure shows the same collapse mechanism during earthquakes as that anticipated at the design stage and will be discussed in later parts of this chapter. Eurocode 8 recommends that the variation range of yield and tensile strengths of steel used in fabrication should be specified.

## 7.3 Structural types and behaviour factors

Steel building frames resist horizontal earthquake loads by moment resisting frames or by braced frames. When moment resisting frames and braced frames are used in combination, those frames are called dual structures in Eurocode 8 and also in Uniform Building Code. Moment resisting frames resist horizontal loads by members acting in an essentially flexural manner. In these structures the dissipative zones are mainly located in plastic hinges near the beam-column connections and energy is dissipated by means of cyclic bending. Braced frames resist horizontal loads by axial forces in the bracings. In these frames the dissipative zones are mainly located in tension and/or compression bracings. Braced frames are in general much stiffer and stronger than moment resisting frames. However, moment resisting frames show much greater deformation capacity than braced frames. Eurocode 8 recommends the values of the behaviour factor depending on the type of structures as shown in figure 7.2, although these values are applicable only when the detailing rules shown in chapters 7 – 8 are met.

Note that tension bracings have greater energy dissipation capacity than compression bracings. Values of the q factor in Eurocode 8 are smaller than the R factors in Uniform Building Code but greater than the $1/D_S$ factors in the Japanese Building Code. These differences, however, have no influence on detailing rules described in this chapter.




Note: $\alpha_u/\alpha_1$ denotes the ratio of the seismic load at which a number of sections, sufficient for development of overall structural instability, reach their plastic moment resistance to the seismic load at which the most strained cross section reaches its plastic resistance. $\alpha_u/\alpha_1$ should be limited to 1.6.

Figure 7.2 – Structural types and behaviour factors according to Eurocode 8

## 7.4 Joints in dissipative zones

Eurocode 8 defines the following criteria for seismic design:

1. Structural parts of dissipative zones should have adequate ductility and resistance until the structure sustains sufficient deformation without failing due to overall instability.

2. Non-dissipative parts of dissipative structures and the connections of the dissipative parts to the rest of the structure should have sufficient overstrength to allow the cyclic yielding of the dissipative parts.

To ensure the sufficient overstrength of connections Eurocode 8 specifies the following detailing rules:

1. The value of the yield strength of the steel actually used in the fabrication should not exceed by more than 10% the value $f_y$ used in the design.

2. Connections of dissipative parts made by means of complete joint-penetration (CJP) groove welds (full-penetration butt welds) are considered to satisfy the overstrength criterion.

3. For fillet-welded or bolted connections the following requirements should be met. These are also applicable to connections at the ends of bracings.

    a) (resistance of the connection according to clause 6 of part 1-1 of Eurocode 3) ≥ 1.2 x (plastic resistance of the connected part)

    b) For bolted shear connections bearing failure should precede bolt shear failure.

It has been shown after the Northridge and Kobe earthquakes that the detailing rule on butt-welded joints mentioned above is not always correct. Further detailing rules to fulfil the sufficient overstrength criterion are discussed in section 7.6.

Eurocode 8 allows connections that are designed to contribute significantly to the energy dissipation capability inherent in the chosen q-factor. The overstrength conditions need not apply for these connections. But these connections have to use experimentally-verified special devices and, therefore, are not suitable to ordinary design office work. The only exception to these difficult connections is the column web panel, which is described in section 7.7.

## 7.5 Strong column-weak beam design

The formation of hinges in columns, as opposed to beams, is undesirable, because this may result in the formation of a storey mechanism (see figure 7.3), in which damage concentrates on a few storeys, and relatively few elements participate in energy dissipation. In addition, such a mechanism may result in local damage to the columns that are critical gravity load bearing elements.

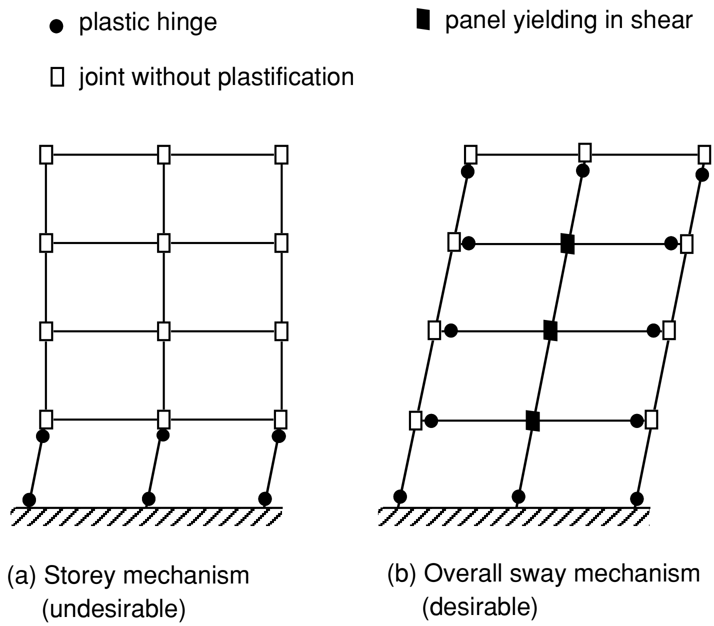



Figure 7.3 – Comparison of desirable and undesirable collapse mechanisms

Eurocode 8 states: “Moment resisting frames shall be designed so that plastic hinges form in the beams and not in the columns. This requirement is waived at the base of the frame, at the top floor of multi-storey buildings and for one storey buildings.” The AISC Seismic Provisions (1997a, 2000) also include relationships that must be satisfied to provide for a nominal condition of strong column-weak beam design, although the AISC equations are not enough to prevent hinging of columns in actual structures. The Interim Guidelines by SAC (1999) as well as FEMA Design Criteria (2000) recommend a more detailed formula to ensure the strong column-weak beam condition. The formula reflects a probable increase in yield strength of beam material and locations of plastic hinges in reinforced beam-to-column connection assemblies.

The Japanese design guide for cold-formed columns (BCJ 1996) recommends that the sum of plastic moment capacities of columns should be 1.5 times greater than the sum of plastic moment capacities of beams, both being calculated using nominal yield strengths, at each connection. The ratio of 1.5 is the result of an engineering judgement based on the examinations of the following factors influencing the strong column-weak beam condition and is found to be about equal to the ratio given by the FEMA Design Criteria mentioned above.

1. When horizontal seismic loads act diagonally to the principal axis of the building, the beams in the two directions participate in carrying bending moments in the columns. Thus, the columns have to be 1.4 times stronger than the beams.

2. Beams are frequently designed as composite elements with concrete slabs.

3. The variability of the yield strength in beam and column materials gives a certain probability of the columns being weaker than the beams.

4. Higher modes of vibration during earthquake responses may force a concentration of bending moment on one side of the columns.

### 7.6 Beam-to-column moment connections (rigid and full-strength connections)

Figure 7.4 shows an example of a beam-to-column assembly along with bending moments in the beam due to horizontal loads.

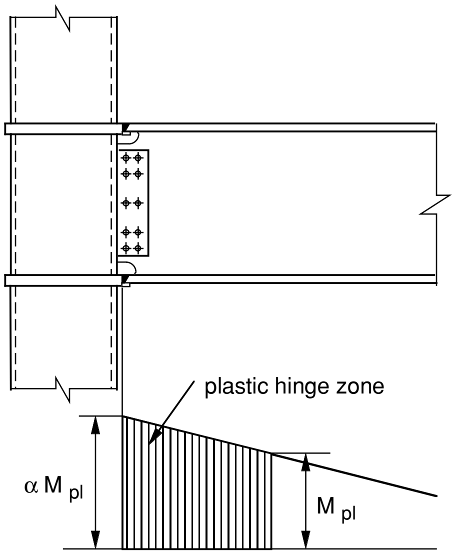



Figure 7.4 – Example of beam-to-column assemblies with conventional details

The bending moment at the beam end, $\alpha M_{pl}$ is controlled by local or combined local, torsional and lateral buckling of the beam, unless tensile failure governs the maximum load. Eurocode 8 specifies that the width-to-thickness ratio of plate elements of dissipative zones of moment resisting frames be in the range of class 1 compact sections and that premature lateral or lateral torsional buckling of beams be prevented by following the clause 5.5.2 of Eurocode 3. Further, Eurocode 8 specifies that the beam-to-column connections should have adequate overstrength to allow the plastic hinges to be formed in the beams. The Eurocode 3 clause 6.9.6.3 recommends to meet the following inequality for rigid full-strength connections:

$$M_{j,\,end}^{*} \geq 1.2M_{pl}^{*} \qquad \text{.......... 7.1}$$

where $M_{j,end}^{*}$ denotes the flexural resistance of the connection at the beam end. Namely, the overstrength factor of $\alpha = 1.2\gamma_{Mweld}/\gamma_{M0} = 1.36$ or greater is recommended so that the plastic hinges at the beam ends have the rotation capacity sufficient for an overall structural mechanism. A typical value for $\alpha$ recommended in the Japanese Building Code is 1.3, although several different values are proposed in AIJ publications varying with the material used and the type of connections. During Northridge and Kobe earthquakes, however, many moment connections sustained tensile failures in beam-to-column joints, which were designed in accordance with detailing rules similar to those of Eurocode 8.

Extensive investigations have been performed both in the US and Japan to find improved details for avoidance of premature tensile failure of beam-to-column joints. One of the important issues, which invalidates conventional details, will be discussed below.

Figure 7.4 shows a typical beam-to-RHS column connection detail. The connection has through continuity plates, also called through diaphragms, at the position of the beam flanges. The beam flanges are field-welded to the through diaphragms using single bevel complete penetration groove welds with backup bars. The beam web is field-bolted to a single plate shear tab that is shop-welded to the column. The column web panel (or connection panel), which is actually a stub-column groove-welded to the through diaphragms at the ends, is not reinforced; the same section as the column section is used for the connection panel.

Cracks frequently started at toes of beam copes and at craters or toes of groove welds around the weld tab regions (the starting and stopping ends of welded butt joints) and extended in a brittle manner across the beam flanges during the Kobe earthquake. One of the reasons for frequent occurrences of fracture in this area is that the lack of flexural capacity in the bolted web connection leads to overstress of the beam flange and the flange groove welds. Namely, the shear tab is welded onto the column flange, which resists a flexural moment from the beam web by out-of-plane bending of the thin-walled column flange. Furthermore, if the web bolts slip, the bolted web connection requires relatively large deformation in order to develop significant flexural capacity (see Engelhardt and Sabol 1996). Therefore, much stiffer flange welds resist most of the bending moment at a connection. A simplified analysis of a connection (Tanaka et al. 1997) shown below clearly demonstrates how the overstress occurs.

Two simplifying assumptions have been made:

1. The maximum moment of $1.2M_{pl}$ or greater is required to develop sufficient rotations at the beam end under cyclic loading;

2. the bolted web connection can transfer 10 % of the moment carried by the flanges. Thus, in order to achieve adequate plastic rotations, the following inequality must be fulfilled.

$$1.1\, A_{b,f}\, (h_b - t_{b,f})\, f_{b,u} \geq 1.2\, W_{pl}\, f_{b,y} \qquad \text{.......7.2}$$

where $A_{b,f}$ signifies the cross-sectional area of the beam flange, while $A_{b,w}$ in the next equation signifies the cross-sectional area of the beam web. $W_{pl}$ signifies the plastic section modulus of the beam. The above relationship can be rewritten as:

$$\frac{f_{b,y}}{f_{b,u}} \leq \frac{1}{1 + 0.275\,\dfrac{A_{b,w}}{A_{b,f}}} \qquad \text{.......7.3}$$

Typical values of $A_{b,w} / A_{b,f}$ for rolled beam sections are about 1.5. This means that, unless the yield/tensile strength ratio is lower than 0.71, the beam flanges might fracture in the beam-to-column joints before developing adequate plastic rotation. Eurocode 3 specifies that the yield/tensile strength ratio of beam material should be lower than 1/1.2 = 0.83. This limiting value is insufficient to prevent fracturing at the beam flange ends.

The connection details shown in figure 7.4 were found inappropriate to attain ductile behaviour. If defects exist in the groove welds, brittle fracture may occur. Improvements in connection details, as will be discussed in the next chapter, are required.

It should be noted herein that the overstrength factor $\alpha$ is an indirect measure to ensure sufficient rotation capacity of a plastic hinge in the member adjacent to the connection. The required $\alpha$ varies with many factors, such as the width-thickness ratio of plate elements of the member as well as the design performance objective of the building frame. Although $\alpha$ is a measure that can be easily used for connection design, it is more unambiguous to specify the inter-storey drift angle to attain a certain performance objective. The FEMA Design Criteria (2000), as a result of nonlinear analytical investigations, recommend the minimum inter-storey drift angles, as shown in table 7.1, up to which connections should withstand the maximum considered earthquake demands without strength degradation or complete failure leading to global collapse of the structure.

| Structural system | Qualifying drift angle capacity-strength degradation, $\Theta_{SD}$ (radians) | Qualifying drift angle capacity-ultimate $\Theta_U$ (radians) |
|---|---|---|
| OMF | 0.02 | 0.03 |
| SMF | 0.04 | 0.06 |

Note: The ordinary moment-resisting frame (OMF) is a moment-resisting frame not meeting special detailing requirements for ductile behaviour. The special moment-resisting frame (SMF) is a moment-resisting frame specially detailed to provide ductile behaviour and comply with requirements given in this Chapter.

$\Theta_{SD}$ takes that value of $\Theta$, at which either failure of the connection occurs or the strength of the connection degrades to less than the nominal plastic capacity, whichever is less. $\Theta_U$ takes that value of $\Theta$, at which connection damage is so severe that continued ability to remain stable under gravity loading is uncertain.

Table 7.1 – Minimum qualifying total inter-storey drift angle capacities for ordinary moment and special moment frame systems according to FEMA 350

The joints at the column ends are usually designed as CJP groove-welded joints with backing bars. However, when cold-formed RHS columns are used, cracks may form at the HAZ at the corners of the RHS section and extend rapidly under inelastic cyclic loading. Material deterioration due to cold working applied during manufacturing processes was found to be mainly responsible for such a premature development of a ductile crack and its change to brittle fracture. Past test results have showed that the rotation capacity of cold-formed RHS columns was reduced considerably by early developments of cracks, although the maximum moments at column ends reached values greater than $M_{pl}$. If the column sections are not interrupted by the diaphragms at beam-column connections, developments of cracks are suppressed. One of the latter examples is a connection with internal diaphragms as described in section 8.5.

Failures at the column ends due to cracks can be prevented by conducting a frame design in which the columns are always stronger than the beams. Further, in many cases the maximum moments at the column ends are limited by yielding of the column web panels, as will be discussed in the next section. If, however, a frame analysis indicates that all the columns in any particular storey have plastic hinges at the top and bottom ends of the columns, the frame is considered to be sustaining a storey mechanism. In this case, the strength of the columns in that particular storey has to be increased. Two approaches for frame design have been proposed. One approach is to decrease the q-factor (increase the $D_S$-factor) for that storey (Akiyama 1994), while the other approach is to reduce the design resistance of the columns in that storey (BCJ 1996). Column hinging only at the base of a frame or only at the top floor of a multi-storey building frame does not constitute a storey mechanism.

## 7.7 Column web panel

The following equation evaluates the shear strength of the column web panel shown in figure 7.5 (AIJ 2001).

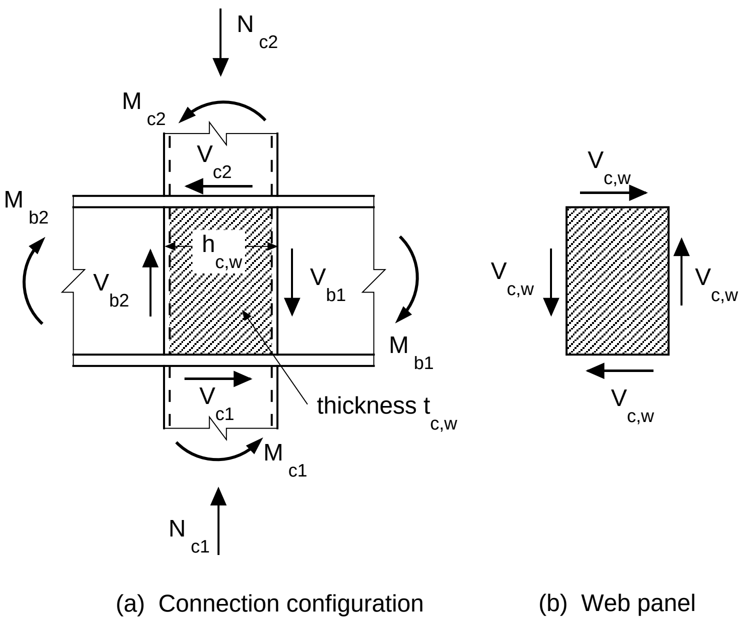



Figure 7.5 – Column web panel framed by flanges and stiffeners

$$V_{c,w}^* = A_{c,w} \sqrt{l - n^2} \; \frac{f_{c,y}}{\sqrt{3}} \qquad \text{7.4}$$

where $A_{c,w}$ signifies the shear area of column web panel and is calculated by

$$A_{c,w} = 2(h_{c,w} - t_{c,w})\, t_{c,w}$$

for both CHS and RHS columns, where $h_{c,w}$ and $t_{c,w}$ denote the depth and thickness of CHS and RHS web panels, respectively. n signifies the average stress in the column web panel divided by the panel yield stress. Equation 7.4 is applicable to both CHS and RHS columns, provided that transverse stiffeners exist at the levels of the beam flanges. The above equation is more suitable for hollow section columns than that of Annex J of Eurocode 3, the latter being appropriate for H-section columns. Connections in moment resisting frames should be designed to satisfy the following inequality:

$$V_{c,w}^* \geq V_{c,w} \qquad \text{.......... 7.5}$$

where the design shear force $V_{c,w}$ can be calculated by:

$$V_{c,w} = \frac{M_{b1} + M_{b2}}{h_b - t_{b,f}} - \frac{V_{c1} + V_{c2}}{2} \qquad \text{.......... 7.6}$$

In the above equation the design loads acting on the column web panel are represented by $M_{b1}$, $M_{b2}$, $V_{c1}$ and $V_{c2}$ denoting, respectively, the flexural moments from the beams on the right-hand and left-hand sides and the shear loads from the columns on the under and upper sides (see figure 7.5).

The elastic rotation of the column web panel $\phi_{el,c,w}$ due to shear can be evaluated by the following equation.

$$\phi_{el,c,w} = \frac{V_{c,w}}{GA_{c,w}} \qquad \text{.......... 7.7}$$

where $GA_{c,w}$ is the initial shear stiffness of the column web panel and has the same form for both CHS and RHS columns.

The column web panel sustains a shear strain $\gamma$ of about 0.5 to 0.6 percent when the shear load reaches the value given by equation 7.4 according to past test results. The panel, however, is able to carry a further increase in load owing to strain hardening, showing a stable load vs. deformation curve. Eurocode 3 Annex J states:

> "A beam-to-column joint in which the design moment resistance $M_{j,cf}^*$ is governed by the design resistance of the column web panel in shear may be assumed to have adequate rotation capacity for plastic global analysis."

Thus, moderate yielding of the column web panel has a beneficial effect on enhancing the performance of moment-resisting frames because it can participate in dissipating input energy. Excessive yielding of the column web panel, however, should be avoided because large shear strain in the panel induces local bending (kinking) of the column or beam flanges in the region adjacent to the groove welds, which may result in a premature development of tensile failures in these regions.

If the connection panel has the same cross section as that of the columns on both sides of the connection, the column web panel is usually weaker than the columns. A rather unusual design, in which the beam depth is greater than 1.5 times the column depth, is an exception to this. Yielding of the column web panel, therefore, helps to prevent hinging of the column. Furthermore, yielding of the column web panel serves the same function as plastic hinges at the beam ends on both sides of the connection.

An example of collapse mechanisms accompanying yielding of the column web panel is shown in figure 7.3(b). Two sided connections are easier to yield in the column web panels, while one sided connections are easier to have a plastic hinge at the beam end. The collapse mechanism shown in figure 7.3(b) has no hinging in the columns except at the base and the top floor of the frame. Thus, a storey mechanism is avoided and more parts of the frame can participate in dissipating energy.

# 8 Rigid (full strength) connections

Semi-rigid beam-to-column connections may be an advantageous option for seismic design, because a certain amount of energy may be dissipated within the connections and also because unnecessary overstress in local areas of connections leading to brittle fracture may be avoided. However, the development of reliable semi-rigid connections under an inelastic cyclic loading condition requires considerable investigation and is not readily applicable to ordinary design office work. Thus, the majority of beam-to-column connections for moment resisting frames use rigid full-strength connections for seismic design, except for the column web panel, which is allowed to yield in shear. The beam end plate connections described in section 8.7 are not fully rigid and one of a few exceptions to rigid connections.

In order to develop full moment capacity, transverse column stiffeners are usually required to transfer axial loads in the beam flanges. The stiffener can be either a through diaphragm, internal diaphragm or external diaphragm. The through diaphragm is the most popular option in Japan because the axial load in the beam flange is directly transferred to the column web in the simplest manner. The other factor for the frequent use of through diaphragms is that most Japanese fabricators have established production resources, particularly welding robots, most suited to producing this type of connection.

This chapter starts with the design of those rigid full-strength connections using through diaphragms, which are specifically applicable to seismic loading conditions.

### 8.1 Connections with through diaphragms for shop welding application

Stub beam-to-column joints are shop-welded at each connection as shown in figure 8.1. The beam spanning the distance between the two stub beams is field bolted. The through diaphragms are usually designed to be thicker than the beam flanges by 3 to 6 mm.

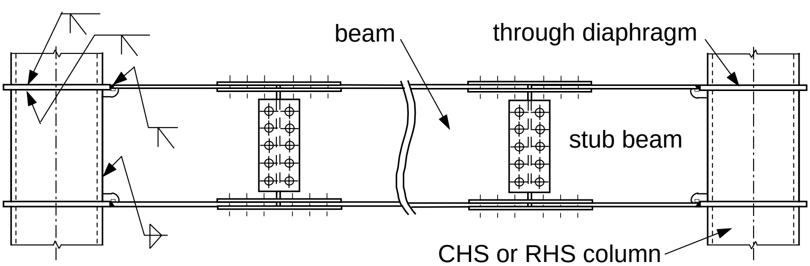



Figure 8.1 – Beam-to-column connections with through diaphragms for shop-welding

Although the CJP groove-welded joints between the beam flanges and through diaphragms look to be relatively simple and conventional, both the Kobe earthquake and post-earthquake investigations demonstrated that brittle fracture could occur in these beam-column joints unless improvements were made in joint details and in welding procedures. One of the reasons for brittle fracture is overstress in the beam flange and in the beam flange groove welds due to the lack of flexural capacity of welded web joints, as discussed in section 7.6. However, there exist several other factors that could trigger tensile failure including brittle fracture at the joints.

A large-scale post-earthquake investigation of beam-to-RHS column connections with through diaphragms was conducted using many replicate specimens (AIJ Kinki 1997). An assessment of results of this investigation revealed that the importance of four factors influencing the ductility of beam-column assemblies could be rated in quantitative terms, and that the ductility of conventional and improved connections could be predicted (Kurobane 1998). This assessment is summarised in Annex A.

Table 8.1(a) shows the predicted mean and factored (mean less one standard deviation) values of the ductility of beam-to-column assemblies and demonstrates how the ductility increases with the improvement of connection details.

(a) Shop-welded connections

| Improvement made in | | | Cumulative plastic deformation factor | |
|---|---|---|---|---|
| Beam cope | Weld pass | Weld tab | Mean | Mean minus one SD |
| yes and no | no | no | 31 (0.040) | 15 (0.029) |
| yes and no | yes | no | 46 (0.047) | 30 (0.039) |
| no | no | yes | 51 (0.049) | 35 (0.041) |
| no | yes | yes | 51 (0.049) | 35 (0.041) |
| yes | no | yes | 54 (0.051) | 38 (0.043) |
| yes | yes | yes | 68 (0.056) | 52 (0.050) |

(b) Field-welded connections

| Improvement made | | | | Cumulative plastic deformation | |
|---|---|---|---|---|---|
| Beam cope | Weld pass | Weld tab | weld defect | Mean | Mean minus one SD |
| yes and no | no | no | non exist | 29 (0.038) | 6 (0.019) |
| yes and no | no | yes | exist | 34 (0.041) | 12 (0.025) |
| no | yes | no | non exist | 42 (0.045) | 19 (0.032) |
| no | yes | yes | exist | 46 (0.047) | 24 (0.035) |
| yes | yes | yes | exist | 47 (0.048) | 25 (0.036) |
| yes | no | yes | non exist | 49 (0.049) | 27 (0.037) |
| yes | yes | yes | non exist | 62 (0.054) | 40 (0.044) |
| Note: The values in the parentheses show inter-storey drift angles in radians. | | | | | |

Table 8.1 – Rotation capacities of conventional and improved beam to RHS column assemblies with through diaphragms

The ductility is represented by the cumulative plastic deformation factor as defined in Annex A. Improved and conventional cope profiles are shown in figure 8.2. The improvement condition indicated as “yes and no” in table 8.1 shows that an improvement in cope profile gives no influence on the ductility, because fracture starts at a weld tab region.

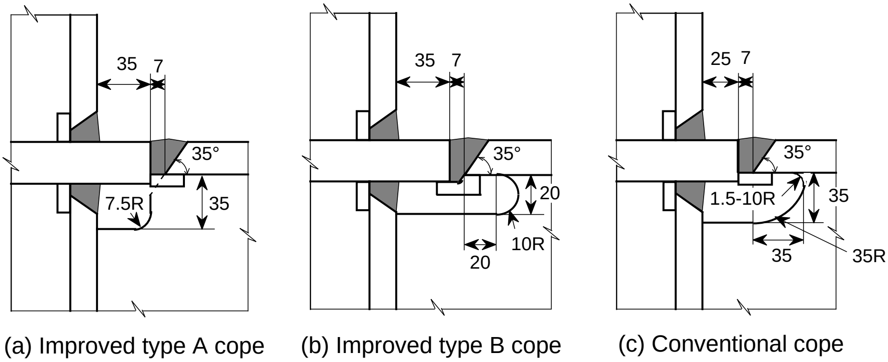



Figure 8.2 – Details of improved and conventional beam copes

The improved weld passes represent stringer passes substituting weave passes. The improved weld tabs represent flux tabs or steel tabs that were removed and ground smooth after welding.

The testing procedure followed by the large-scale investigation resembles the procedure recommended in the FEMA Design Criteria. The cumulative plastic deformation factor used in the large-scale investigation can be converted to the inter-storey drift angle used in the FEMA Design Criteria by the formula $\theta = 0.0081\eta^{0.46}$, in which $\theta$ denotes the storey drift angle and $\eta$ denotes the cumulative plastic deformation factor (see Annex A). The converted inter-storey drift angles in radians are shown in the parentheses comparing with the cumulative plastic deformation factors. The assessment of the large-scale investigation shows that connections with improved details are equivalent to prequalified connections conforming with the FEMA Acceptance Criteria (FEMA 2000a), as will be discussed in the following part of this section and also in section 8.3.

The inter-storey drift angles shown in table 8.1 correspond to those designated as the "qualifying drift angle capacities-strength degradation $\theta_{SD}$". Although the "drift angles-ultimate" are not recorded in the report (AIJ Kinki 1997), these angles are considered significantly greater than $\theta_{SD}$ and, therefore, not affecting the evaluation. This is because, even if brittle fracture occurs in a tension flange, the crack would not propagate into the compression flange, leaving a certain residual capacity to support gravity loads. The measured drift angles were those that extracted the rotations of the beams only from the total rotations and therefore do not include rotations due to deformations of the columns and connection panels. The other factor to be considered is that beam-to-column assemblies used in these tests had fairly large beam span-to-depth ratios, $L_b/h_b$, varying from 13 to 16, where $L_b$ signifies the centre-to-centre span. The elastic components of the beam rotations are of the order of 0.01 radians.

Taking these conditions into account, the following observations can be made by comparing the test results for shop-welded connections shown in table 8.1(a) with the FEMA criteria shown in table 7.1.

The 4 connections with improved weld tabs listed in table 8.1(a) have factored drift angle capacities greater than 0.04 radians, namely the limiting value specified for the special moment frame. However, the drift angle capacities of the connections with conventional beam copes exceed the limiting value only marginally. Beam copes with improved profiles are recommended to be used for the special moment frame.

The connection with no improvement at all shows the ductility being greater than 0.02 radians specified for the ordinary moment frame. The connections with conventional details are allowed to be used for the ordinary moment frame (see the footnote to table 7.1 for the definition of the special moment frame and ordinary moment frame).

The design procedure and fabrication requirements for connections applicable to the special moment frame will be proposed here.

(1) *Connection strength*

The ultimate flexural strength of the connection at the column face, $M_{j,cf}^*$, can be evaluated by

$$M_{j,cf}^* = M_{b,f,u} + M_{b,w,u} \qquad \text{.......... 8.1}$$

where $M_{b,f,u}$ utilizes the ultimate moment carried by the welded joints between the beam flange and diaphragm and is given by

$$M_{b,f,u} = A_{b,f}\,(h_b - t_{b,f})\, f_{b,u} \qquad \text{.......... 8.2}$$

The symbol $M_{b,w,u}$ signifies the ultimate moment carried by the welded web joint and is given by

$$M_{b,w,u} = m\, W_{pl,b,w,n}\, f_{b,y} \qquad \text{.......... 8.3}$$

where $W_{pl,b,w,n}$ signifies the plastic section modulus of the net area of the beam web considering reduction of cross section due to the cope holes, which can be calculated by

$$W_{pl,b,w,n} = \frac{1}{4}\,(h_b - 2t_{b,f} - 2s_v)^2\, t_{b,w} \qquad \text{.......... 8.4}$$

The symbol m in equation 8.3 represents the dimensionless moment capacity of the welded web joint, which is expressed as

$$m = 4\,\frac{t_c}{d_j}\sqrt{\frac{b_j f_{c,y}}{t_{b,w} f_{b,y}}} \text{ and } m \le 1.0 \qquad \text{.......... 8.5}$$

where $b_j = b_c - 2t_c$ and $d_j = h_b - 2t_d$ denote the width and depth of the face of the column web panel where the beam web is welded (see figure 8.3). The throat thickness of the fillet welds between the beam web and column flange is assumed to be greater than $\sqrt{3}t_{b,w}/2$. The ultimate moment capacity of welded web joints given by equations 8.3 to 8.5 is based on a yield line analysis by Suita and Tanaka (2000) and was shown to agree well with existing test results. There is a slight difference between $M_{j,end}^*$ defined in section 7.6 and $M_{j,cf}^*$. However, this difference will be ignored hereafter.

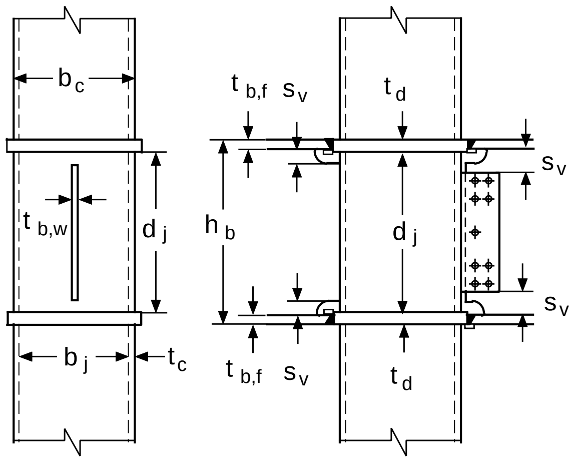



Figure 8.3 – Dimensions of welded web joint

(2) *Overstrength criteria*

Although Eurocode 3 recommends the overstrength factor given by equation 7.1, a more appropriate value for the factor is pursued directly from test results. As shown in figure A.5, the maximum moment at the column face, $M_{cf,max}$, attained during cyclic loading is plotted against the cumulative plastic deformation factor for all the specimens excluding field-welded connections with improved beam copes (the latter connections have details different from the rest of specimens). The figure indicates that the maximum moment, non-dimensionalised by the full-plastic moment of the beam, $M_{pl}$, increases with the cumulative plastic deformation factor. The increase in strength may be due to the effects of an increase in rotation range, cyclic hardening of material and plastic constraints at the welded joints. These effects are not taken into account in equation 8.1, which is a simple formula applicable to monotonically loaded connections.

The required overstrength factor is found to be about 1.2 or greater as observed from figure A.5, if an increase in strength due to cyclic loading is disregarded. The value of $\alpha = M_{j,cf}^*/M_{pl}$ calculated from equation 8.1 using measured material properties varies between 1.2 and 1.25, which is consistent with the above estimation.

The flexural strength given by equation 8.1 is calculated using nominal material properties $f_{b,y}$ and $f_{b,u}$, which may be different from the actual values of the yield and ultimate tensile strengths $\sigma_y$ and $\sigma_u$. The overstrength factor should be multiplied by the probable yield/tensile strength ratio $R_{y/u}$, that is:

$$R_{y/u} = \frac{(\sigma_y / \sigma_u)_m}{f_{b,y}/f_{b,u}} \qquad \text{.......... 8.6}$$

Thus, the overstrength factor is given as:

$$\alpha = 1.2\, R_{y/u} \qquad \text{.......... 8.7}$$

A statistical assessment of the steel used in fabrication is required to determine $R_{y/u}$. A default value of $\alpha$ = 1.3 is suggested because this value has been frequently used in actual design practice. The overstrength criterion can be written as

$$M_{j,cf}^{*} \geq \alpha\, M_{pl} \qquad \text{.......... 8.8}$$

The partial safety factors $\gamma_{Mweld}/\gamma_{M0}$ are taken to be unity in the above equation. However, the variability of flexural capacity is taken into account when evaluating the rotation capacity of beam-column assemblies. It should be noted, however, that the overstrength factor proposed here is based on a large-scale investigation (AIJ Kinki 1997). If the width/thickness ratios of the beam flange and web are much lower than those of the beams used in this large-scale investigation, a further experimental assessment is necessary to find an appropriate value of $\alpha$.

(3) *Profiles of beam cope*

Beam copes had better have the improved profile shown either in figure 8.2(a) or (b). A further improvement has recently been attempted in the profile (Nakagomi et al. 2001). The improved profile, as shown in figure 8.4, has no opening for the backing bar.

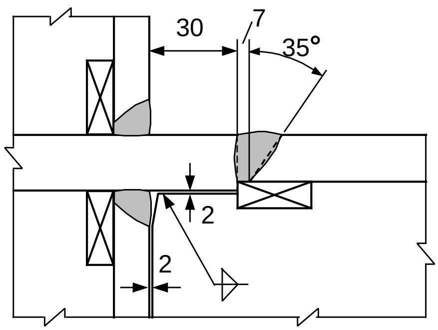



Figure 8.4 – Further improvement in beam end cutting with no access hole

Two pieces of backing bar are made to fit from both sides of the beam flange because the beam web interrupts the backing bar (this was also the case with improved type A cope shown in figure 8.2(a)). With this detail, a portion of the flexural moment in the beam web can be transmitted to the diaphragm directly. A series of tests of beam-to-column assemblies using these new connections demonstrated that the rotation capacity is satisfactory and comparable to dog bone connections described in section 8.4 (Suita et al. 1999). Another improved profile proposed is identical to the conventional profile (see figure 8.2 (c)), except that the radius of curvature at the toe of cope holes should be greater than 10 mm. Careful machining of the cope hole is mandatory with this detail. Material toughness should at least conform to the requirements of Eurocode 8.

(4) *Weld tabs*

Use flux weld tabs. Welders should be qualified for the use of flux tabs. If steel tabs are used, remove the tabs after welding and then grind smooth the ends of welds.

(5) *Backing bars*

It is allowed to leave backing bars as they are after final welding. However, avoid tack welding at points within a distance of 5 mm from the edges of the beam flange and from the beam web fillet.

(6) *Welding procedures*

Use stringer passes where practical. Limit heat-input to 40 kJ/cm and interpass temperature to 350 °C.

(7) *Quality control*

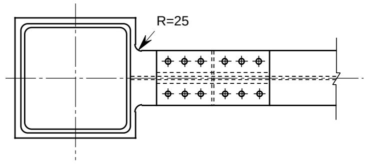
The quality of the CJP groove-welded joints should be equivalent to that of specimens tested. There was found no discontinuity constituting rejectable conditions per the AIJ UT Criteria (AIJ 1979) prior to testing. In principle, 100 % non-destructive testing as well as visual inspections are required for the CJP groove-welded joints. Note that these joints are to be classified as the seismic weld demand category A (high) according to FEMA 353 (2000a). This category is defined as “welds in which service stresses are anticipated to be at or beyond the yield stress level, with some inelastic strain demand into the strain hardening region anticipated.” The allowable defect size is dependent not only on the severity of a notch but also the material toughness, strain history, and so forth. Fitness-for-service criteria for assessment of weld defects were recently proposed (JWES 1997, IIW 2003), which were evaluated in the light of experimental results and actual structures damaged during the Kobe earthquake (Azuma et al. 2000, Shimanuki et al. 1999).

Ordinary moment frames are applicable to building structures in low-seismicity zones or isolated structures, for which hysteretic damping of input energy is unessential. For connections of ordinary moment frames, the fabrication requirements (3), (4), (6) and (7) may be waived.

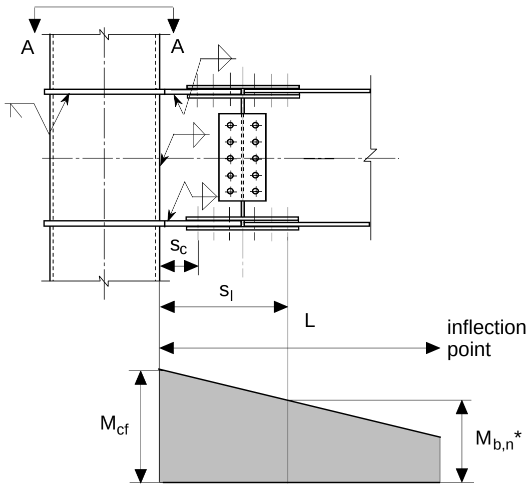
The design procedure and fabrication requirements proposed here are mainly based on a large-scale investigation (AIJ Kinki 1997). However, there exist several other experimental studies of beam-to-RHS column connections with through diaphragms. The results of these other studies do not contradict the proposals made herein. The maximum thickness of the beam flanges is 32 mm in these existing test results. Further, the required interstorey drift angle capacities were determined by following the FEMA Design Criteria, which imply that the minimum beam span-to-depth ratio should be limited to $L_b/h_b > 8.0$ in order that the connections are prequalified.

## 8.2 Bolted through diaphragm connections

The beam-to-column connections described here are one of the alternatives to welded connections shown in the previous section. The through diaphragms are extended sufficiently far to accommodate bolted beam splices used as field connections (see figure 8.5). Ochi et al. (1998) selected the thickness of the through diaphragms to be greater than the thickness of the beam flanges so that the stub-beam section extending from the column face had appropriate overstrength to allow desired yielding modes shown below. Alternatively, it is possible to use a through diaphragm with the same thickness as that of the beam flange, if horizontal haunches are prepared as shown in figure 8.6.



(a) Section A–A



(b) Side view and acting moments

Figure 8.5 – Bolted through diaphragm connections

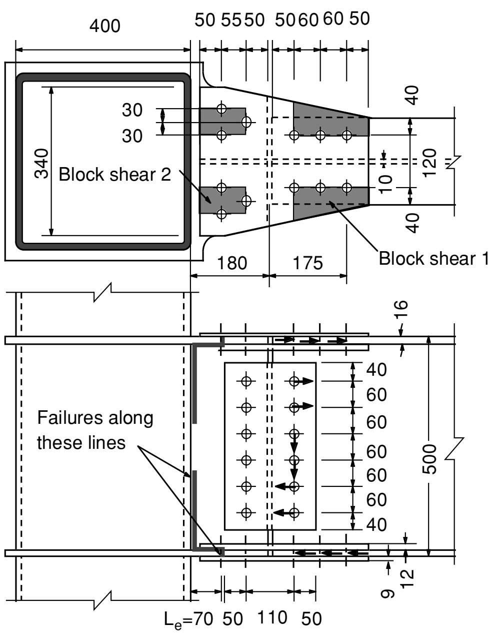


Figure 8.6 – Design example and assumed failure modes of beam splice and stub-beam

These connections are similar in ultimate behaviour to the bolted flange plate connections recommended in FEMA 350 (2000). Good inelastic behaviour is achieved with balanced yielding in the following three preferred mechanisms:

1. flexural yielding and local buckling of the beam adjacent to the beam splice;

2. yielding of the stub-beam;

3. yielding of the beam splice.

The flexural capacity of the beam at the net section at the last row of bolts away from the column determines the moment demand on these connection details. The moment capacity at the net section can be calculated by

$$M_{b,n}^* = (b_{b,f} - \frac{n}{2} d_h)\, t_{b,f}\, (h_b - t_{b,f})\, f_{b,u} + (h_b - 2t_{b,f} - x) x t_{b,w} f_{b,y} \qquad \text{.......................8.9}$$

with

$$x = \frac{h_b - 2t_{b,f}}{2} - \frac{nd_h}{2}\,\frac{t_{b,f} f_{b,u}}{t_{b,w} f_{b,y}}$$

in which $d_h$ denotes the diameter of bolt holes and n denotes the number of bolt holes at the last row (n = 2 in this example). The deduction in cross-sectional area of the beam flange on the compression side due to bolt holes is ignored in the above equation. In order to achieve sufficient flexural yielding of the beam, the following overstrength criterion must be met:

$$M_{b,n}^* \geq 1.2 M_{pl} \qquad \text{.......................................................................................8.10}$$

The required overstrength factor is smaller than that recommended in section 8.1, because yielding in the stub beam and beam splices participates in the inelastic rotation of this connection as well. The required flexural capacity at the column face is:

$$M_{cf} = \frac{L}{L - s_l} M_{b,n}^* \qquad \text{.......................................................................................8.11}$$

where $L/(L - s_l)$ accounts for increase in the beam moment due to a moment gradient (See figure 8.5).

The ultimate flexural capacity of the stub beam, which should be greater than or equal to $M_{cf}$, is given by the smaller of the values evaluated on the following two failure modes. The dimensions $b_{b,f}$, $h_b$, $t_{b,f}$ and $t_{b,w}$ denote those of the stub beam, while $f_{b,y}$ and $f_{b,u}$ denote the nominal yield and tensile strengths for the stub beam materials, in the following calculations.

If the net section failure through the first row of bolts closest to the column face is assumed, equation 8.9 can be used to calculate the moment capacity at this section, $M_{b,n}^*$. The moment capacity at the column face is given by

$$M_{j,cf}{}^{*} = \frac{L}{L - s_c} M_{b,n}{}^{*} \qquad \text{.......... 8.12}$$

where $s_c$ stands for the distance between the first bolts and column face. Note that the value of $M_{b,n}{}^{*}$ in the above equation is not equal to that used in equation 8.11.

If the net section failure through the first row of bolts accompanies a shear rupture of the beam web over the length $L_e$ (see figure 8.6), the ultimate moment capacity at the column face can be calculated by equation 8.1 and equations 8.13 – 8.14 shown below.

$$M_{b,f,u} = (b_{b,f} - nd_h)t_{b,f}(h_b - t_{b,f})f_{b,u} \qquad \text{.......... 8.13}$$

$$M_{b,w,u} = mW_{pl,b,w,n}f_{b,y} + \frac{L_e t_{b,w}(h_b - 2t_{b,f})f_{b,u}}{\sqrt{3}} \qquad \text{.......... 8.14}$$

The deductions for bolt holes are made on both the tension and compression sides of the beam flanges in equation 8.13 to simplify the equations.

The beam splice should be designed as a connection slip-resistant at the serviceability limit state following Eurocode 3 Clause 6.5. The number of bolts, edge distances, bolt spacings, and so forth should be determined to allow the desired failure modes (yielding of the beam splice, etc.). The design ultimate flexural load given by equation 8.11 should not exceed the design flexural and bearing resistances (see Eurocode 3, Clause 6.5.5) at the column face, where $\gamma_{M0}$ and $\gamma_{Mb}$ may be taken to be unity. However, shear failures of high-strength bolts should be avoided. The shear resistance of high-strength bolts should be evaluated with a due safety margin ( $\gamma$ = 1.25 in Eurocode 3). Standard details for beam splices, if any, may be utilised.

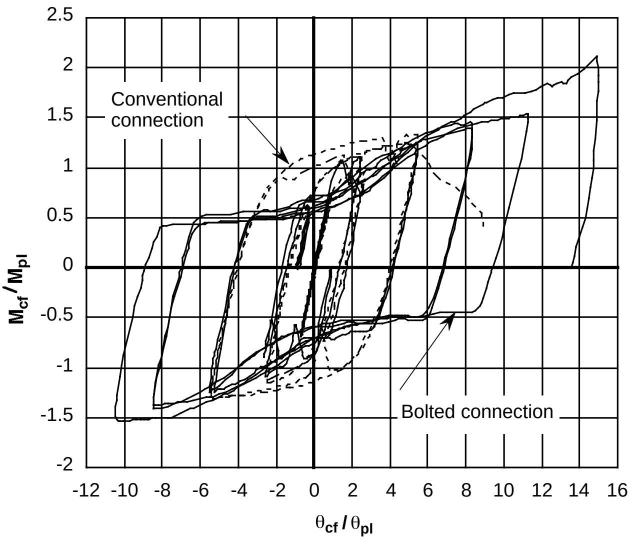



Figure 8.7 – Comparison of moment/rotation hysteretic curves between conventional and bolted through diaphragm connections

Figure 8.7 shows examples of moment, $M_{cf,}$ versus rotation, $\theta_{cf,}$ hysteresis loops, in which a bolted through diaphragm connection is compared with a conventional connection by welding. The bolted through diaphragm connection sustained combined tensile yielding and local buckling of the beam flange and web adjacent to the beam splice. Yielding also occurred in the splice plates and the stub beam (see figure 8.8). The hysteresis loops for the bolted through diaphragm connection showed a pinched form because bolts slipped. However, the loops showed significant hardening envelopes after the bolts slipped into bearing until very large rotation of the beam was achieved.

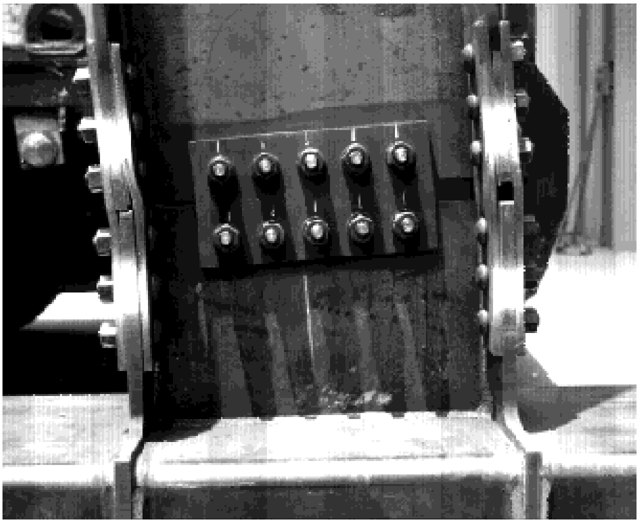


Figure 8.8 – Failure mode of bolted through diaphragm connection

Advantages of these bolted connections over welded connections lie in the fact that brittle fracture can be avoided by using sufficiently tough material for the beams, splice plates and diaphragms. The beam flanges at the bolted connections sustained plastic deformation largely in a plane stress state, suggesting that the flanges would fail by plastic instability rather than brittle fracture. Additionally, no welded joints exist at the stub-beam ends where the beam moments become highest. Thus, the demand for skilled welding is less for the new bolted connections. It is easy to achieve a cumulative plastic deformation factor greater than 100 for these connections. These connections are examples of those drastically improved over conventional welded ones (see section 8.1).

### 8.2.1 Design example for bolted through diaphragm connections

A beam-to-column connection with bolted through diaphragms shown in figure 8.6 is to be checked if the connection details are appropriate to allow for the desired failure modes under the influence of a strong earthquake. The column is the hot-finished square hollow section 400 x 400 x 16 of Grade EN-10210 S275J2H. The beam is the hot-rolled I-section 500 x 200 x 10 x 16 of Grade JIS G3136 SN400B. The plate materials are also of the same grade of steel. The high-strength bolts used are of Grade 10.9 with a nominal diameter of 20 mm. The nominal values of the yield and ultimate tensile strengths for each material are shown below:

| Material | Yield strength ($N/mm^2$) | Ultimate tensile strength ($N/mm^2$) |
|---|---|---|
| Square hollow section | 275 | 410 |
| I-section and plates | 235 | 400 |
| High-strength bolts | 900 | 1000 |

The ratio of the fully-plastic moment of the column to that of the beam is computed at 2.1, which shows that this beam-to-column assembly satisfies the strong column weak beam condition.

The beam centre-to-centre span is 8000 mm. Assume the inflection point at the centre of the span and check if a plastic hinge can form at sections adjacent to the beam splice. Assume that a shear force of 63 kN due to gravity loads acts on the connection.

**Moment demand at column face**

The flexural capacity of the beam at the net section at the last row of bolts can be calculated by equation 8.9. Namely,

$$M_{b,n}^* = [(200 - 22) \cdot 16 \cdot (500 - 16) \cdot 400 + (500 - 2 \cdot 16 - x) \cdot x \cdot 10 \cdot 235]10^{-6} = 672 \text{ kNm}$$

where

$$x = \frac{500 - 2 \cdot 16}{2} - 22 \cdot \frac{16 \cdot 400}{10 \cdot 235} = 174 \text{ mm}$$

The ratio of the net section flexural capacity to the beam fully-plastic moment is:

$$\frac{M_{b,n}^*}{W_{pl} f_{b,y}} = \frac{672}{2130 \cdot 235 \cdot 10^{-6}} = 1.34$$

which is a sufficiently large number to warrant formation of a plastic hinge in the beam. The required moment demand at the column face is given by equation 8.11 as:

$$M_{cf} = \frac{3800}{3800 - 355} \cdot 672 = 741 \text{ kNm}$$

**Flexural capacity of stub beam**

Equation 8.12 gives the flexural capacity, at the column face, of the stub beam section through the first bolt holes as:

$$M_{j,cf}^* = \frac{3800}{3800 - 70} \cdot 1012 = 1031 \text{ kNm}$$

where $s_c$ = 70 mm. The flexural capacity of 1012 kNm was calculated by equation 8.9 as:

$$M_{b,n}^* = [(340 - 2 \cdot 22) \cdot 16 \cdot (500 - 16) \cdot 400 + (500 - 2 \cdot 16 - x) \cdot x \cdot 10 \cdot 235]10^{-6}$$
$$= 1012 \text{ kNm}$$

where

$$x = \frac{500 - 2 \cdot 16}{2} - 2 \cdot 22 \cdot \frac{16 \cdot 400}{10 \cdot 235} = 114 \text{ mm}$$

If the net section failure through the first bolt holes accompanies shear ruptures of the beam web (see figure 8.6), equation 8.13 gives the flexural capacity of the beam flanges while equation 8.14 gives the flexural capacity of welded web joints to the column flange and to the diaphragms. Namely,

$$M_{b,f,u} = [(340 - 4 \cdot 22) \cdot 16 \cdot (500 - 16) \cdot 400] \cdot 10^{-6} = 781 \text{ kNm}$$

$$M_{b,w,u} = \left[0.897 \, \frac{10 \cdot (500 - 2 \cdot 16)^2}{4} \cdot 235 + \frac{70 \cdot 10 \cdot (500 - 2 \cdot 16) \cdot 400}{\sqrt{3}}\right] \cdot 10^{-6}$$
$$= 191 \text{ kNm}$$

where $L_e$ = 70 mm. The value of m of 0.897 used in the above equation was calculated by equation 8.5 as

$$m = 4 \cdot \frac{16}{468} \sqrt{\frac{368 \cdot 275}{10 \cdot 235}} = 0.897$$

The moment capacity at the column face is equal to the sum of $M_{b,f,u}$ and $M_{b,w,u}$ calculated above (see also equation 8.1). Namely,

$$M_{j,cf}^* = M_{b,f,u} + M_{b,w,u} = 972 \text{ kNm}$$

The above calculations show that the latter failure mode is more critical than the former failure mode. However, even with the latter failure mode the flexural capacity of the stub beam becomes much greater than the moment demand of 741 kNm. This is due to horizontal haunches prepared on the stub beam side to accommodate 6 bolts.

**Design of beam splice**

Details and dimensions of the beam splice are largely governed by fabrication requirements. The cross-sectional areas of the splice plates are significantly greater than those of the beam flanges and webs. There is no need to check the net section strength of the splice plates in this example. The following calculations of bolted joints are based on Eurocode 3 Clause 6.5.

The shear resistance of high strength bolts per shear plane is given as

$$V_b^* = \frac{0.6 \cdot f_{b,u} \cdot A_b}{\gamma_{Mb}} = \frac{0.6 \cdot 1000 \cdot \pi \cdot 10^2}{1.25} 10^{-3} = 151 \text{kN}$$

Shear planes are assumed to pass through unthreaded portion of the bolts. All the bolts are used as double shear joints and therefore the shear resistance of each bolt is equal to $2V_b^*$.

The bearing resistance of a bolt $B_b^*$ is determined by the thinnest plate on which the bolt bears. Either the beam flange with the thickness of 16 mm or the beam web with the thickness of 10 mm governs the bearing resistance at a bolt hole. The bearing resistance is a function also of the end distance, bolt spacing, bolt diameter and bolt hole diameter. The values of the bearing resistance calculated by using the bearing resistance equation in table 6.5.3 of Eurocode 3 are as follows:

| Material on which a bolt bears | When the end distance governs | When the bolt spacing governs |
|---|---|---|
| flange | 242 kN | 211 kN |
| web | 152 kN | 132 kN |

The partial safety factor $\gamma_{M0}$ was taken to be unity, because bolt hole elongation is one of the preferred failure modes. Thus, the design shear resistance of each bolt is always governed by the design bearing resistance $B_b^*$.

The flexural capacity of the beam splice can be calculated as the sum of the flexural capacity of the beam flange-to-splice plate joint with 6 bolts and that of the beam web-to-splice plate joint with 2 bolts. Namely,

$$M_{bs}^* = [(2 \cdot 242 + 4 \cdot 211) \cdot (500 - 16) + 2 \cdot 152 \cdot 240] \cdot 10^{-6} = 716 \text{ kNm}$$

Note that only the two bolts closest to the top flange and those closest to the bottom flange are assumed to carry flexural loads. The flexural capacity of the beam splice at the column face is (see equation 8.11 and figure 8.6):

$$M_{bs,cf}^* = 716 \cdot \frac{3800}{3800 - 180} = 751 \text{ kNm}$$

which is greater than the moment demand of 741 kNm. The value of 180 mm in the above equation stands for the distance from the column face to the section at which bending and shear loads are carried only by the beam splice.

The shear load is resisted by the two bolts at the centre. The shear capacity of the beam splice is given as:

$$V_{bs}^* = 2 \cdot 132 = 264 \text{ kN}$$

while the required shear capacity $V_{bs}$ is the sum of the shear loads due to gravity and earthquake loads. Namely,

$$V_{bs} = 63 + \frac{741}{3800} = 258 \text{ kN}$$

A block shear failure mode is possible on the splice plates with the total thickness of 12 + 9 mm as shown in figure 8.6. The shaded portions in figure 8.6 may tear out. According to Eurocode 3 Clause 6.5.2.2, the design resistances to block shear become 1599 kN and 1565 kN for the block shears 1 and 2, respectively. The partial safety factor $\gamma_{M0}$ was taken to be unity, because shear and tensile ruptures are preferred failure modes. The block shear resistances are slightly greater than the bearing resistances of 6 bolts in the flange, which are equal to $2 \cdot 242 + 4 \cdot 211 = 1358$ kN. Thus, the block shear failure is less critical.

The above calculations suggest that, in the beam-column assembly adopted in this example, a plastic hinge forms in the beam section adjacent to the beam splice accompanying local buckling of the beam flanges and webs. The next critical section is the net section through the last bolt holes away from the column within the beam span. Some bolt hole elongation can be anticipated. However, note that the ultimate strength equations for the bolt hole bearing and block shear specified in Eurocode 3 are more conservative than similar equations recommended in other codes like the AISC LRFD Specification (1999).

A beam-column assembly with details similar to this example was recently tested. The assembly showed an excellent plastic rotation capacity sustaining local buckling in the beam flanges and web at sections just outside the beam splice (Kurobane 2002). The beam flange necking due to tensile yielding was observed at the section of the last bolts. These failure modes were close to those anticipated at the design stage.

## 8.3 Connections with through diaphragms for field welding application

Field-welded connections can be more economical than shop-welded connections, because beam splices may be omitted if the connections are field-welded. However, the quality control of field-welded joints and the repair of weld flaws detected after welding are more difficult than in shop-welded connections. Thus, field-welded connections frequently require reinforcements by some means to suppress strength demands on welded joints, as will be described in a later part of this section.

### 8.3.1 Connections with improved details

Once again, the results of the large-scale tests (AIJ Kinki 1997) for field-welded connections, shown in table 8.1(b), are compared with the FEMA criteria shown in table 7.1. Only connections with improved beam copes, improved weld passes, improved weld tabs and no weld defect have factored inter-storey drift angle capacity greater than 0.04 radians and are applicable to the special moment frame.

The improved field-welded connections mentioned above were further improved in a recent study (Miura et al. 2002). An example of these new improved connections is illustrated in figure 8.9. Note that this connection has no beam cope. This connection may be slightly more economical and showed a better rotation capacity than the original ones tested in the large-scale investigation. The design procedure and fabrication and erection requirements are proposed based on these new connection details.

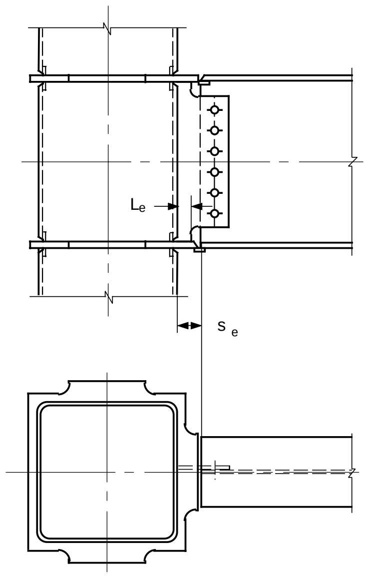



Figure 8.9 – Improved field-welded connection

(1) *Connection strength*

Equations 8.1 and 8.2 are applicable without any modification to calculate the ultimate flexural strength of the connection. However, equation 8.14 can be used to calculate the flexural strength of the welded web joint. This is because the fillet welds at the top and bottom ends of the shear tab participate in carrying bending moments in the web. Note that, when calculating the flexural strength of the welded web joint, $t_{b,w}$, $f_{b,y}$ and $f_{b,u}$ represent the thickness and nominal yield and tensile strengths of the shear tab, while $t_{b,f}$ represents the thickness of the extended diaphragm. The bolted web connection should be designed as a connection slip-resistant at the serviceability limit state following Eurocode 3 Clause 6.5.

(2) *Overstrength criteria*

The same overstrength factors as those proposed for shop-welded connections are applicable to this connection. However, it should be noted that the space $s_e$ between the beam end and column face is not negligibly small (see figure 8.9).

The overstrength criteria given by equation 8.8 should be rewritten as

$$M_{j,cf}{}^{*} \geq \frac{L}{L - s_e} \alpha M_{pl} \qquad \text{.......................................} 8.15$$

(3) *Welded joint details*

Although no cope hole is needed to be prepared for this connection, the space $s_e$ should be sufficiently large, say $s_e \geq 60$ mm, to conduct sound CJP groove-welding to the end of the bottom beam flange. With regards to weld tabs, backing bars and welding procedures, the same requirements as those recommended for shop-welded connections should be applied to this connection as well.

(4) *Quality control*

The quality of the CJP groove-welded joints should be equivalent to that of shop-welded joints.

### 8.3.2 Connections for ordinary moment frames

All the field-welded connections, except those with no improvement at all, have factored inter-storey drift angle capacity greater than 0.02 radians, as seen in table 8.1(b), and are applicable to the ordinary moment frame. Namely, conventional connections with an improvement only in weld tabs can be prequalified for the ordinary moment frame.

When calculating flexural strength of the welded web joint, $t_{b,w}$ and $f_{b,y}$ denote the thickness and nominal yield strength of the shear tab, while $s_v$ stands for the space between the beam flanges and shear tab (see figure 8.3). The other design and fabrication requirements are identical to those recommended for shop-welded ordinary moment frame connections. It should be noted, however, that the weld defects referred to in table 8.1(b) are those created at starting and stopping ends of welded joints by welders who were not skilled at flux tabs and are those detected after testing by careful inspections. There was found no discontinuity constituting rejectable conditions according to the AIJ UT Criteria (AIJ 1979) before testing. The seismic weld demand category B (medium) according to FEMA 353 (2000a) may be applicable to the ordinary moment frame. This category is defined as "welds in which service stresses are anticipated to be near or slightly exceed yield level, but for which negligible inelastic strain demand is anticipated." The requirements for quality control may be slightly relaxed accordingly as compared with those for the special moment frame.

### 8.3.3 Reinforced connections

Several proposals have been made regarding reinforced connection details applicable to the special moment frame. The most popular details belong to the connections with horizontal haunches. The width of beam flange is gradually increased towards the beam end as shown in figure 8.10 so that the welded connection is sufficiently strong when the plastic hinge forms outside the haunch within the beam span. The flexural capacity at the section where the haunch starts, $\alpha M_{pl}$, is governed by local buckling of the beam flange and web, when the beam is adequately restrained against lateral instability.

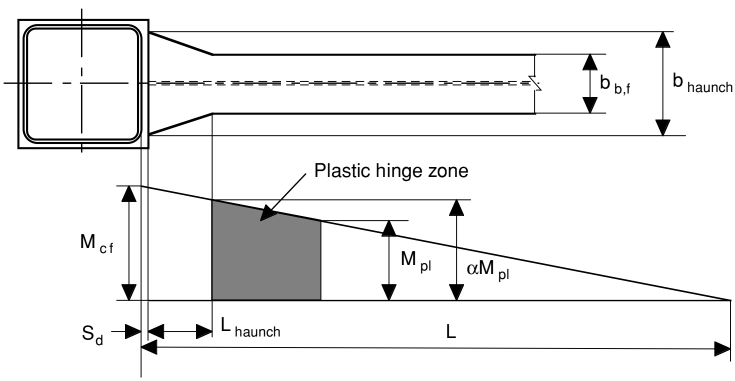



Figure 8.10 – Beam-to-column connection with horizontal haunches

Tanaka (1999) used Kato and Nakao's empirical equations (1994), shown in table 8.2, to determine the magnitude of $\alpha M_{pl}$.

| Grade of steel | Non-dimensionalized maximum moment $\alpha$ | Symbols used |
|---|---|---|
| $f_u$= 400 N/mm² | $\frac{1}{\alpha} = \frac{0.4896}{\alpha_f} + \frac{0.0460}{\alpha_w} + 0.7606$ | $\alpha_f$ = Local buckling parameter for beam flange $= \frac{E}{\sigma_{b,f,y}}\left(\frac{t_{b,f}}{b_{b,f}/2}\right)^2$ |
| $f_u$= 490 N/mm² | $\frac{1}{\alpha} = \frac{0.2868}{\alpha_f} + \frac{0.0588}{\alpha_w} + 0.7730$ | $\alpha_w$ = Local buckling parameter for beam web $= \frac{E}{\sigma_{b,w,y}}\left(\frac{t_{b,w}}{(h_b - 2t_{b,f})}\right)^2$ |
| $f_u$= 570 N/mm² | $\frac{1}{\alpha} = \frac{0.1999}{\alpha_f} + \frac{0.0748}{\alpha_w} + 0.7672$ | $\sigma_{b,f,y}$ = measured yield strength of beam flange<br>$\sigma_{b,w,y}$ = measured yield strength of beam web |

Table 8.2 – Flexural capacity of cantilevered I-beams governed by local buckling of plate elements (Kato and Nakao, 1994)

He postulated that the moment demand at the end of the haunch (at the column face) $M_{cf}$ should be within the fully-plastic moment of the section at the end of the haunch, ignoring the web. Namely,

$$M_{cf} \leq b_{haunch} t_{b,f}(h_b - t_{b,f}) f_{b,y} \qquad \text{......8.16}$$

in which $b_{haunch}$ stands for the flange width of beam section at the end of the haunch. $M_{cf}$ is given by

$$M_{cf} = \frac{L}{L - s_d - L_{haunch}} \alpha M_{pl} \qquad \text{......8.17}$$

The length of horizontal haunch should be as small as possible but be large enough to avoid tensile failures in the haunch. In order to evaluate the ultimate capacity of the haunch, he assumed two fracture paths as shown in figure 8.11.

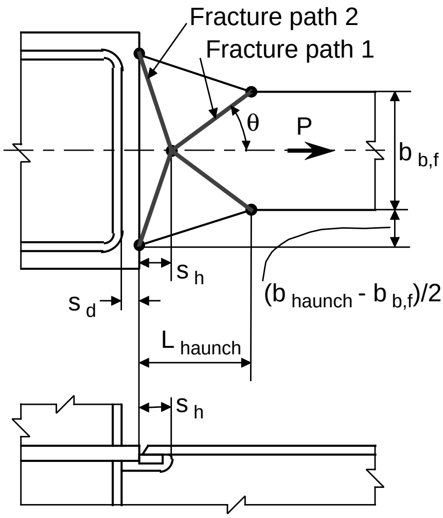



Figure 8.11 – Fracture paths in horizontal haunches

Further, an assumption was made that the fracture paths, each being inclined to the tensile load P by $\theta$, can carry the load:

$$P_u = 2L_{fracture}t_{b,f}(\sin\theta + \frac{\cos\theta}{\sqrt{3}})f_{b,u} \qquad \text{......8.18}$$

where $L_{fracture}$ represents the length of each segment of these fracture paths. Tanaka found that the above equation agrees well with the test results of beam-to-column connections with horizontal haunches. Finally, he proposed the following formula for the optimum length of the haunch, $L_{haunch}$, which ensures sufficient overstrength to prevent tensile failures of the haunches:

$$L_{haunch} = s_h + \sqrt{s_h^2 + (b_{haunch}^2 - b_{b,f}^2)/4} \qquad \text{......8.19}$$

where $s_h$ denotes the distance between the edge of the through diaphragm and the toes of the beam cope.

Kato and Nakao's formulae underpredict the maximum moment at the beam end when the beam flanges and web buckle locally, especially when the width/thickness ratio of the plate elements decreases (as $\alpha$ approaches 1.1). Kato admitted this underprediction and proposed an 8 % increase in $\alpha$ values over those given by the formulae in table 8.2 (Kato 2000). Nevertheless, equation 8.16 is employing a significantly conservative assumption, as compared with the design strength equations recommended for shop-welded connections in section 8.1. Tanaka's proposal, as a whole, leads to conservative connection designs. Therefore, the seismic weld demand category should be classified as B (medium). Further details on this connection design can be obtained from Kajima Technical Research Institute, Kajima Corporation, (phone: +81-424-89-8439). No licensing agreement is required.

Although the above investigation was based on a series of tests, in which each of the beam flanges with haunches was cut from a single piece of plate, the connections with the same configuration can be fabricated by welding trapezoidal rib plates to the flanges of rolled H-section beams (Sugimoto and Takahashi 1999). Figure 8.12 shows details of welded haunches.

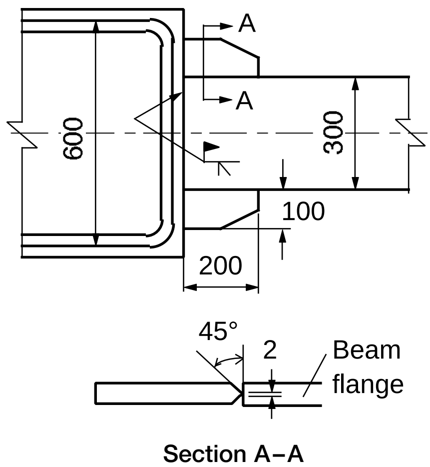



Figure 8.12 – Details of welded horizontal haunches

Test results using these connections were also found to demonstrate sufficient rotation capacity. In these tests the moment $\alpha M_{pl}$ at the section where the haunches start to prevent tensile failure was postulated as

$$\alpha M_{pl} = (h_b - t_{b,f})\, b_{b,f}\; t_{b,f}\; f_{b,u} + \frac{1}{4}(h_b - 2t_{b,f})^2\, t_{b,w}\, f_{b,y} \qquad \text{.................................8.20}$$

which is much larger than the value bounded by local buckling of plate elements of ordinary beam sections. The width of the haunch $b_{haunch}$ is determined to fulfil the inequality 8.16. The length of the haunch is determined assuming that the full plastic moment of the beam web, $(h_b - 2t_{b,f})^2\, t_{b,w}\, f_{b,y}/4$, is carried by shear stresses acting along the beam web fillet over that length.

The proposal made by Sugimoto and Takahashi (1999) is even more conservative than Tanaka's proposal. The seismic weld demand category can be classified as B.

The horizontal haunches described above have been proposed by engineers in construction companies. Fabricators feel that these details are costly. A new connection detail proposed by a fabricator is shown in figure 8.13, in which a cut is prepared along the edge of the through diaphragm so that the beam flange fits into the cut, creating a U-shaped welded connection at the beam end. The design procedure for this connection will be discussed in section 8.5 on connections with internal diaphragms.

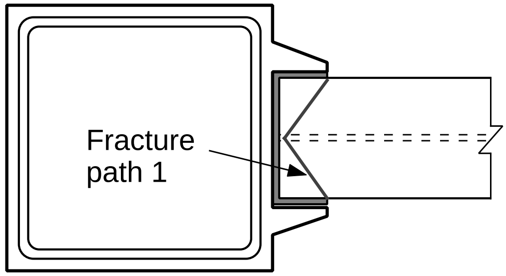



Figure 8.13 – Connection with U-shaped welded joints

### 8.4 Reduced beam section (RBS) connections

These connections utilize cuts or drilled holes in both the top and bottom flanges to reduce the flange area over a length of the beam near the ends of the beam span. Plastic hinges form over segments of the beam with reduced flange sections away from the columns within the beam span, thus limiting the maximum bending moments at the beam ends. The FEMA Design Criteria (2000) recommend a circular cut for the reduced beam section with the following geometry (see figure 8.14). This connection is also called the dog bone connection.

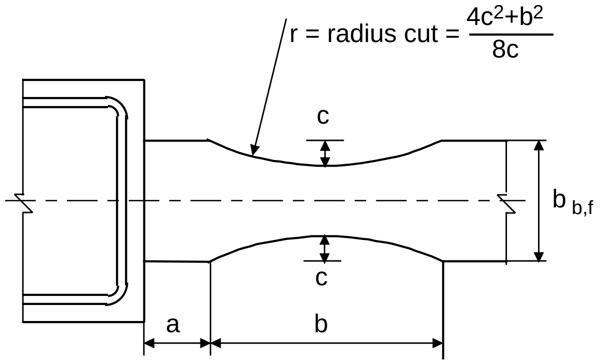



Figure 8.14 – Geometry of radius cut RBS

The designer should select the dimensions a and b according to the following guidelines:

$a \cong (0.5 \text{ to } 0.75)b_{b,f}$    $b \cong (0.65 \text{ to } 0.85)h_b$

The remaining dimension that must be chosen is c, the depth of the cut. The value of *c* will control the maximum moment developed within the RBS, and therefore will control the maximum moment at the end of the beam (see equation 8.11 for example). The FEMA Design Criteria recommend that the maximum moment at the column face be chosen to be less than 100 percent of the beam's expected plastic moment. The lowest of the maximum moments recorded in the large-scale investigation (AIJ Kinki 1997) was equal to 1.1 $M_{pl}$, where $M_{pl}$ is the fully plastic moment calculated using the measured yield strength of the beam material. Therefore, FEMA criteria are judged to be sufficiently safe, even if they are applied to beam-to-RHS column connections with through diaphragms, including field-welded connections. The FEMA Design Criteria further recommend that c should be less than or equal to $0.25b_{b,f}$.

Suita et al. (1999) tested beam-to-column assemblies with the RBS. These specimens had conventional beam copes. The maximum moment at the beam end was designed to reach $M_{pl}$. Test results for these specimens showed a satisfactory rotation capacity comparable to that for companion specimens with improved beam copes but without the RBS. No crack was found at the beam flange-to-through diaphragm welds. Specimens finally failed by local buckling and ductile tensile failure at the reduced sections. Although the number of specimens with the RBS was only three, the results were reproducing well the behaviour of I-section beam-to-I-section column connections described in the SAC Seismic Design Criteria (1999a).

The FEMA Design Criteria classify these connections as the seismic weld demand category B, unless the connections have weak panels whose large shear strain may induce local bending of the beam flanges (see section 7.7)

## 8.5 Connections with internal diaphragms

The through diaphragm interrupts the column twice at each connection. Instead, if an internal diaphragm as shown in figure 8.15 is used, the column has to be cut only once at each connection, which considerably contributes to saving fabrication costs. However, welding of the diaphragm has to be executed from the end of the column.

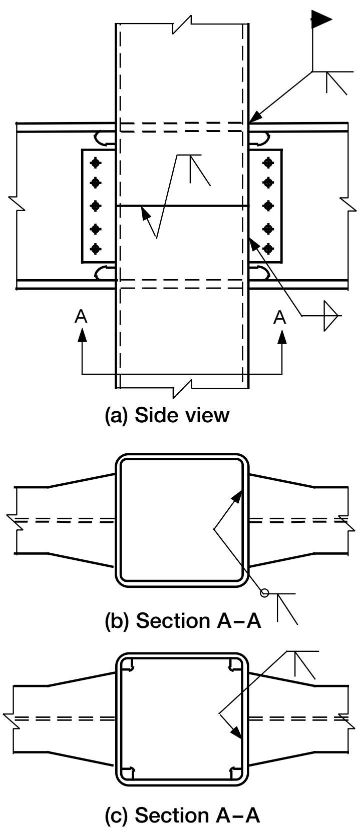



Figure 8.15 – Beam-to-column connections with internal diaphragms

According to past investigations weld defects were frequently observed at corner regions if the internal diaphragm was welded around the inside surface of the column wall as shown in figure 8.15(b). If cope holes were prepared in the internal diaphragm at corners as shown in figure 8.15(c), it is possible to eliminate weld defects. Welding of internal diaphragms requires considerable skill. Welders should practice welding using mock-up specimens before manufacturing actual connections. It is desirable to develop welding robots for welding internal diaphragms for the future.

Cold-formed RHS with internal diaphragms installed prior to manufacturing the sections are also available. The patents for these columns have already expired. An example of one of the details is shown in figure 8.16.

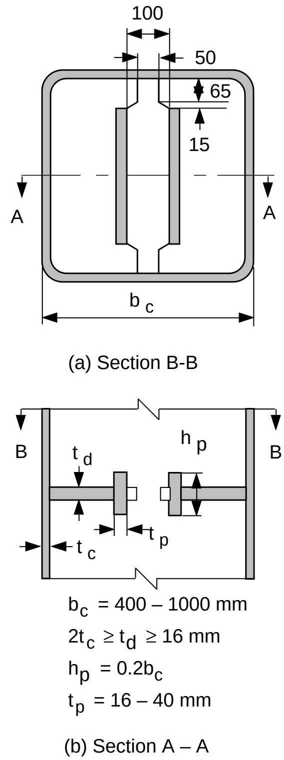



Figure 8.16 – RHS with prefabricated internal diaphragms

The diaphragms are groove-welded to the inside of cold-formed channel sections at predetermined positions. Then the two channel sections are butt-welded from both the inside and outside of the section to produce one RHS section with the internal diaphragms. Each of the diaphragms has a slit at the centre. The diaphragm acts as an axially loaded member when the beam flange force is parallel to the slit. The diaphragm acts as a flexurally loaded member when the beam flange force is perpendicular to the slit. The positions and dimensions of the diaphragms depend on the positions and dimensions of the beams. Thus, RHS with internal diaphragms have to be ordered after frame design is completed. Specific qualification and design information on these sections may be obtained from Nittetsu Column Co. Ltd., (phone: +81-45-623-4681, fax: +81-45-623-4688).

Improvements of welded connections proposed for beam-to-column connections with through diaphragms, such as improved profiles of beam cope and horizontal haunches, are applicable to connections with internal diaphragms as well. Simpler profiles of beam cope, as shown in figure 8.17, can be used when no through diaphragm exists (AIJ 1995).

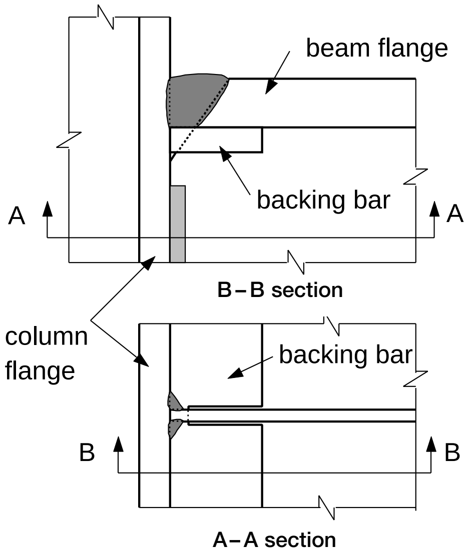



Figure 8.17 – Improved profile of beam cope with simple configuration

Connections with cover plates are especially suited to connections with internal diaphragms. These connections are relatively economical, compared with some other connections like those with horizontal haunches, and have limited architectural impact. However, designers frequently prefer horizontal haunches to cover plates for top flanges because they want to have flush top flanges for placing profiled steel decking over the beams without any attachments. In general, RHS columns with internal diaphragms offer more opportunities than those with through diaphragms, for attempting an improvement in the connection strength.

Figure 8.18 shows an example specimen of a cover-plated connection tested by Engelhardt and Sabol (1998).

Ten out of 12 connections tested by them were able to achieve satisfactory plastic rotations. However, one specimen sustained brittle fracture starting at the CJP groove welds at the beam end. The other unsuccessful specimen fractured at the column flange. The SAC Interim Guidelines (1999) state: "Although apparently more reliable than the former prescriptive connection, this configuration is dependent on properly executed beam flange to column flange welds. Further these effects are somewhat exacerbated as the added effective thickness of the beam flange results in a much larger groove weld at the joint, and therefore potentially more severe problems with brittle heat affected zones (due to excessive heat input) and lamellar defects in the column." The SAC Seismic Design Criteria (1999a) recommend using this type of connection only when highly redundant framing systems are employed and also to limit an area of the cover plate to about 3/4 of the beam flange.

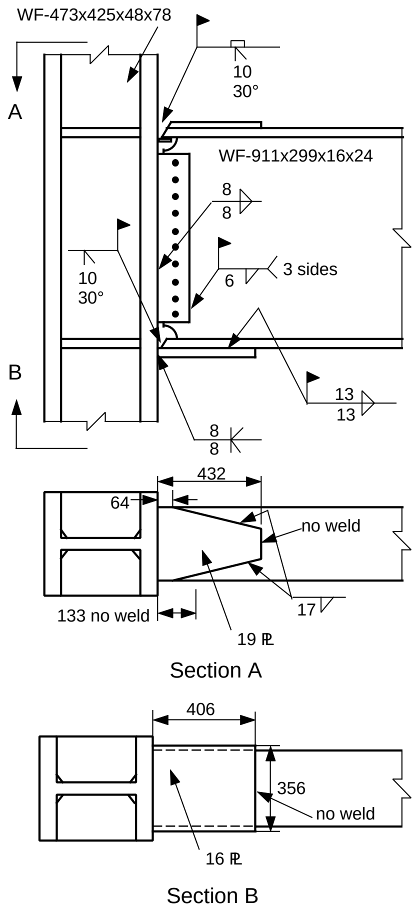



Figure 8.18 – Beam-to-column connection with cover plates (Engelhardt and Sabol 1998)

Although recommendations by SAC are based on test results for connections with I-section columns, details of welded joints are similar to joints with RHS section columns with internal diaphragms. The same recommendations may be applicable to both cases. A few small size cover-plated connections with RHS columns were tested at Kajima Technical Research Institute (Tanaka and Sawamoto 2001). These specimens showed excellent plastic rotation capacity, although the thickness of the beam flanges was only 10 mm.

One of the economical connection details using the internal diaphragm is shown in figure 8.19, in which a flange plate with bolt holes is shop-welded at the position of the bottom flange.

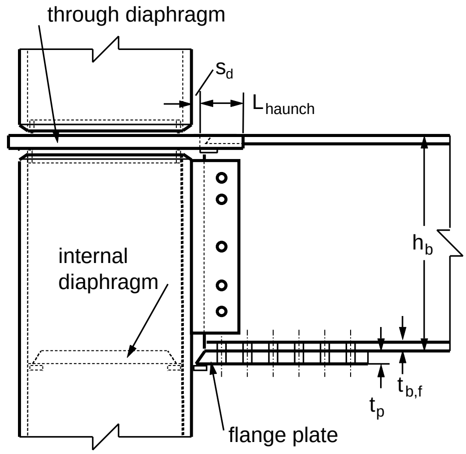


Figure 8.19 – Beam-to-column connections with combined internal and through diaphragms

The top flange is field-welded using the U-shaped welded joints referred to in section 8.3. Note that no beam cope is used for the connection shown in figure 8.19. The internal diaphragm at the position of the bottom flange was welded from the end of the column. Details of the bottom flange connections are the same as those of the bolted flange plate connections recommended in FEMA 350 (2000). If the bottom flange connection is designed to be slightly stronger than the top flange connection, and if the bolted web connection is assumed to carry only shear load in the beam, the moment capacity at the end of the horizontal haunch is given by

$$M_{j,cf}^* = \left\{ b_{b,f} t_{b,f} f_{b,u} + \frac{2}{\sqrt{3}} (L_{haunch} - s_h) t_{b,f} f_{b,u} \right\} (h_b - \frac{t_{b,f}}{2} + \frac{t_p}{2}) \quad \text{.....................8.21}$$

where $t_p$ signifies the thickness of the flange plate. The space $s_h$ is equal to 7 mm (= root opening) in these connection details. The same fracture paths as those proposed by Tanaka (1999) are applicable to this connection (see figures 8.11 and 8.13). However, the fracture path 2 does not usually control the joint capacity with this detail and is not shown in figure 8.13. Equation 8.18 was used for the derivation of the above equation. The over-strength requirement is that $M_{j,cf}^*$ given by equation 8.21 should exceed $M_{cf}$ given by equation 8.17, where $\alpha \geq 1.3$ is recommended. These connections were found to perform well if the length of the haunch was so proportioned as to have sufficient overstrength (Kurobane et al. 2001, Miura et al. 2002).

## 8.6 Connections with external diaphragms

Connections with external diaphragms have been mainly studied at Kobe University (AIJ 1996). The design formulae for these connections have been included in the Architectural Institute of Japan Recommendations since 1980 (AIJ 1990). Both circular and square hollow section columns are applicable. The studies by Kamba et al. (1983) and Tabuchi et al. (1985) afforded the basis for the design formulae.

Recently, however, RHS column-to-beam connections with external diaphragms were studied by research groups other than Kobe University (Matsui et al. 1996, Ikebata et al.

1999, Mitsunari et al. 2001), which helped not only to enhance the reliability of existing design provisions but also to widen the validity range of the formulae. Further, Kamba modified the current design formula for connections to CHS columns based on a series of numerical analysis results (Kamba 2001), which also helped to enhance the reliability of the formula.

The current AIJ design provisions give the yield strengths of the connections and follow an allowable stress design format. These yield strength equations are herein rewritten in the ultimate limit state design format with a few amendments taking into account the results of recent studies. The AIJ Recommendations use the ratio of yield to ultimate resistances of 0.7 for connection design, unless there exist definite experimental evidences showing that another value of this ratio is more appropriate. The past test results for connections with external diaphragms indicated that the ultimate strength was significantly greater than the yield strength divided by 0.7. However, the ultimate loads were attained after large plastic deformation of the diaphragms and column walls, frequently accompanying cracks at re-entrant corners of the diaphragms. Therefore, the ultimate resistances of these connections are assumed to be 1/0.7 times the yield strengths of the connections, and are shown in table 8.3.

The flexural capacity of the connections with the external diaphragms can be calculated by equation 8.22 shown below.

$$M_{j,cf}^* = P_{b,f}^*(h_b - t_{b,f}) \quad \text{.......... 8.22}$$

The flexural capacity of the welded web joint is ignored because the stiffening effects of the column walls have already been taken into account in the ultimate resistance equations in table 8.3.

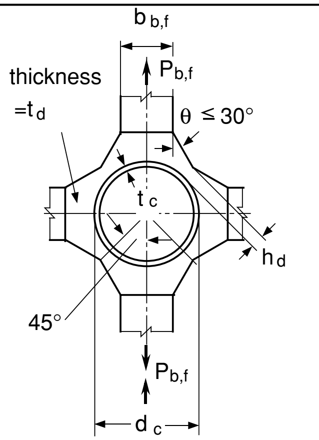
The required flexural capacity at the column face can be given by

$$M_{cf} = \frac{L}{L - L_{haunch}} \alpha M_{pl} \quad \text{.......... 8.23}$$

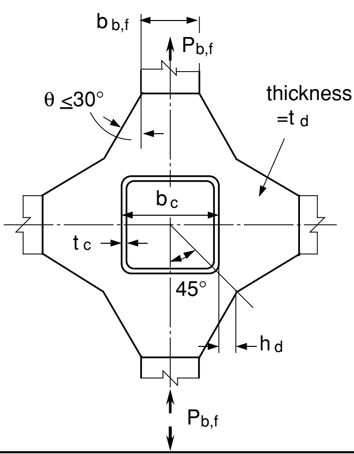
where $L_{haunch}$ shows the distance between the column face and the end of the horizontal haunch, which is equivalent to the length of the external diaphragm measured from the column face (see figures 8.20(a) and (b)). The overstrength factor of $\alpha$ = 1.2 is recommended because the external diaphragms sustain large plastic deformation, participating considerably in overall plastic rotation at the beam end.

The validity ranges of equations 1 and 2 in table 8.3 are shown in the same table. Although the equations in table 8.3 were based on test results for connections in which the beam flanges were welded to the external diaphragms, the beam flanges were bolted to the diaphragms as shown in figure 8.20(b) in recent studies (Ikebata et al. 1999, Mitsunari et al. 2001). The same design formulae as described above were found applicable to these bolted connections as well. The maximum thickness of the beam flange and external diaphragm was 25 mm for CHS column connections, while the maximum thickness of the beam flange and column wall was 16 mm and the maximum thickness of the external diaphragm was 22 mm for RHS column connections, in the existing tests.

These connections with external diaphragms may fail by cracks starting at re-entrant corners of the diaphragm or of the welded joints between the diaphragm and beam flange.

| Shape of external diaphragm | Ultimate resistance equation |
|---|---|
|  | $P_{b,f}{}^{*} = 19.6\left(\frac{d_c}{t_c}\right)^{-1.54}\left(\frac{h_d}{d_c}\right)^{0.14}\left(\frac{t_d}{t_c}\right)^{0.34}\left(\frac{d_c}{2}\right)^{2} f_{c,y}$ (1)<br><br>Symbols:<br>$f_{c,y}$ = Yield strength of column material<br>$P_{b.f}$ = Axil load in tension or compression flange |

| Range of validity | | | |
|---|---|---|---|
| $14 \le \frac{d_c}{t_c} \le 36$ | $0.05 \le \frac{h_d}{d_c} \le 0.14$ | $0.75 \le \frac{t_d}{t_c} \le 2.0$ | $\theta \le 30°$ |

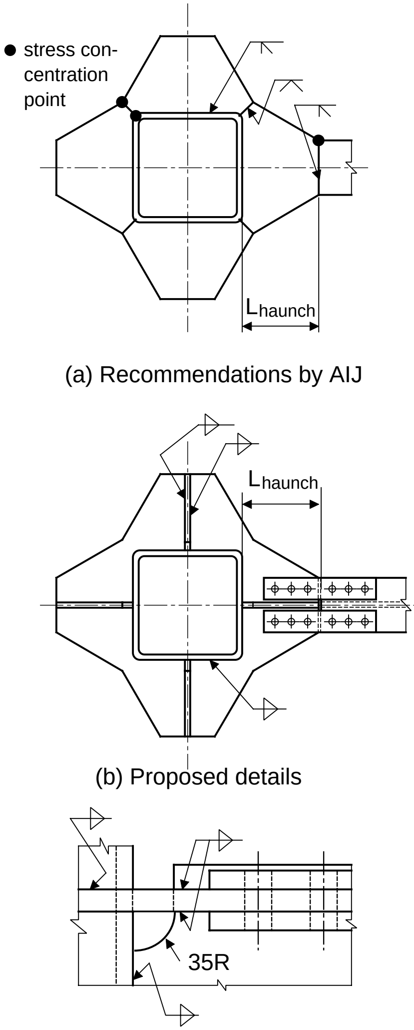
| Shape of external diaphragm | Ultimate resistance equation |
|---|---|
|  | $P_{b,f}{}^{*} = 3.17\left(\frac{t_c}{b_c}\right)^{2/3}\left(\frac{t_d}{b_c}\right)^{2/3}\left(\frac{t_c + h_d}{b_c}\right)^{1/3} b_c^2\, f_{d,u}$ (2)<br>where<br>$\frac{b_c/2 + h_d}{t_d} \le \frac{240}{\sqrt{f_{d,y}}}$<br>Symbols:<br>$f_{d,y}$ = Yield strength of diaphragm material<br>$f_{d,u}$ = Ultimate tensile strength of diaphragm material<br>$P_{b.f}$ = Axil load in tension or compression flange |

| Range of validity | | | |
|---|---|---|---|
| $17 \le \frac{b_c}{t_c} \le 67$ | $0.07 \le \frac{h_d}{b_c} \le 0.4$ | $0.75 \le \frac{t_d}{t_c} \le 2.0$ | $\theta \le 30°$ |

Note:
Symbols: b = Width d = diameter h = Height t = Thickness θ = Slope of diaphragm
Subscript: b = Beam c = Column d = Diaphragm

Table 8.3 – Ultimate resistance equations for connections with external diaphragms (Kamba 2001, Tabuchi et al. 1985)

Sharp corners should be avoided at these critical points, with a minimum corner radius of 10 mm or greater. Especially for the connection to the RHS column, stress and strain concentrations are inevitable at points around the corners of the column because the diaphragms and column webs cross at a right angle here (see figure 8.20(a)). Cracks frequently develop in the welds, column walls or diaphragms at these points.




(a) Recommendations by AIJ

(b) Proposed details

(c) Details of welded joints at flange centre

Figure 8.20 – Beam-to-column connections with external diaphragms

The AIJ design provisions (1990) recommend to use CJP groove-welded joints at the corners of the column to fabricate a continuous external diaphragm (see figure 8.20[a]). The AIJ design provisions further recommend that CJP groove-welded joints be used between the external diaphragm and column face. When these connections are used for the special moment frame, the CJP groove welded joints should be classified as the seismic weld demand category A (see section 8.1).

Mitsunari et al. (2001), however, reported that diaphragms with fillet-welded joints at the centres of the flanges and with no welded joints at re-entrant corners of the diaphragms (see figure 8.20(b),(c)) showed much better performance at the ultimate limit state than those with details recommended by the AIJ (see figure 8.20(a)). Further, recent studies (Matsui et al. 1996, Ikebata et al. 1999, Mitsunari et al. 2001) showed that fillet-welded joints can be substituted for the CJP groove welded joints between the external diaphragm and column face (see figure 8.20(b)). The size of the fillet welds in these studies was about a half of the diaphragm thickness. If the size of the fillet welds required becomes too large, CJP groove welds have to be used like a proprietary connection using an external diaphragm in cast steel (Nakano et al. 1999).

The local deterioration of material toughness due to cyclic loading significantly promoted formation of cracks at the points of strain concentration. When the connections with external diaphragms are used for the ordinary moment frame, the demand on the weld quality can be relaxed to some extent as compared with the welded joints strained into inelastic regions cyclically. The seismic weld demand category B may be applicable to the CJP groove-welded joints for the ordinary moment frame.

When a frame is at a periphery of a building, designers frequently want to move the beam to the exterior side of the column (see figure 8.21). In such a connection a side plate as shown in figure 8.21(b) is efficient for transferring an axial load in the beam flange to the column.

Matsui et al. (1996) proposed the following empirical equations to evaluate the strength of connections with side plates on both sides of the column as shown in figure 8.21(c).

$$P_{p,u} = 2h_p t_p f_{p,u} + 2(\beta t_c + t_d) t_c f_{c,u} \qquad \text{.......... 8.24}$$

in which $h_p$, $t_p$, and $f_{p,u}$ signify the height, thickness and tensile strength of the side plate, respectively. The first term in the above equation represents the load carried by the side plates themselves, while the second term represents the load carried by the welded joints between the beam and column flanges. $\beta$ is a coefficient that is equal to 3 when the column is empty but is equal to 4 when the column is filled with concrete. Matsui and co-workers furthermore proposed the following rule for the connection in which the external diaphragms and the side plates are used in combination. Namely, the strength of such a connection is given by the smaller of the strength of the connection with the side plates only and the strength of the connection with the external diaphragms only. Thus, the ultimate resistance $P_{b,f}{}^*$ of the connection shown in figures 8.21(a) and (b) is evaluated as the smaller of the values given by the following two equations:

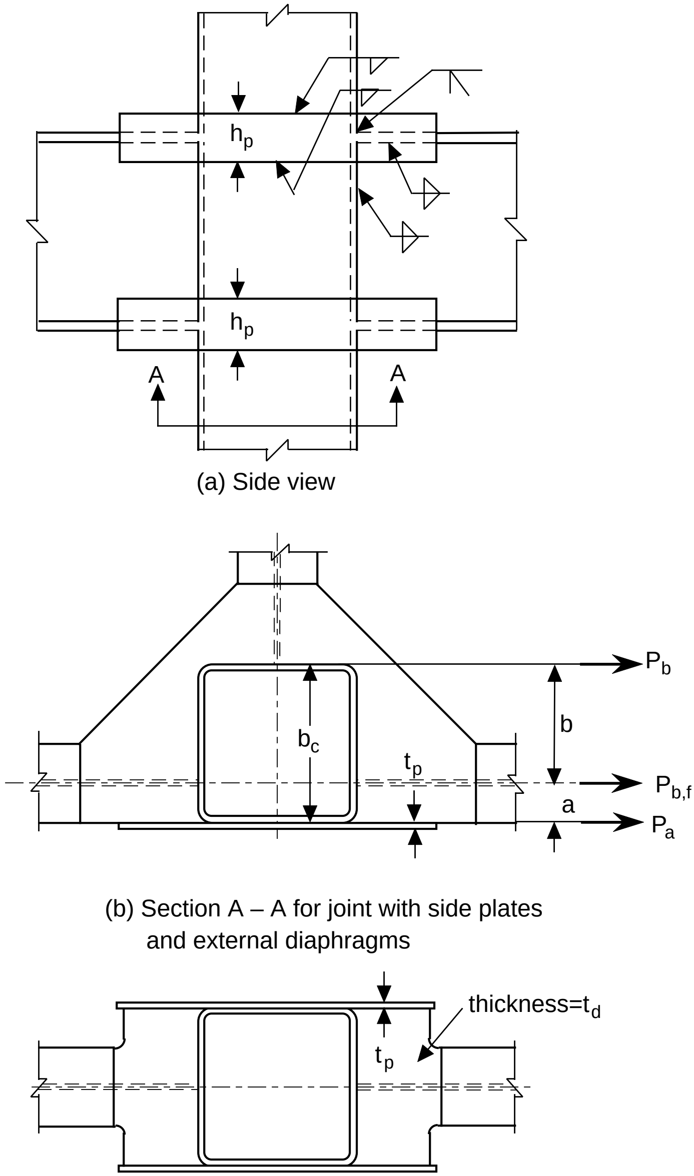
$$P_{b,f}{}^* = \frac{b_c}{2b} P_{p,u} \qquad \text{.......... 8.25}$$

$$P_{b,f}{}^* = \frac{b_c}{2a} P_{d,u} \qquad \text{.......... 8.26}$$

in which $P_{d,u}$ is the ultimate strength of the connection with external diaphragms and is the same as $P_{b,f}{}^*$ given by equation 2 in table 8.3 and $b_c$ signifies the width of column. Note that, since an eccentricity exists in this connection, the axial load $P_{b,f}{}^*$ is distributed between $P_a$ and $P_b$ in the following proportion:

$$\frac{P_a}{P_b} = \frac{b}{a} \qquad \text{.......... 8.27}$$

For these connection details, the validity range of θ (see table 8.3) can be relaxed to allow θ ≤ 45° based on the past test results (Matsui et al. 1996, Ikebata et al. 1999).




(a) Side view

(b) Section A – A for joint with side plates and external diaphragms

(c) Section A – A for joint with side plates only

Figure 8.21 – Beam-to-column connections with external diaphragms and side plates

## 8.7 End plate connections with blind bolts

Connections described here offer a straightforward solution for beam-to-RHS column connections by bolting. Beams with end plates are directly connected to column faces by tension bolts. Closed column sections necessitate the use of blind bolts. Further, the column walls have to be reinforced to prevent local distortion of the column walls and to fulfil the requirements for full strength connections. The column walls may be partially thickened over the areas where the end plates are attached (see figure 8.22).

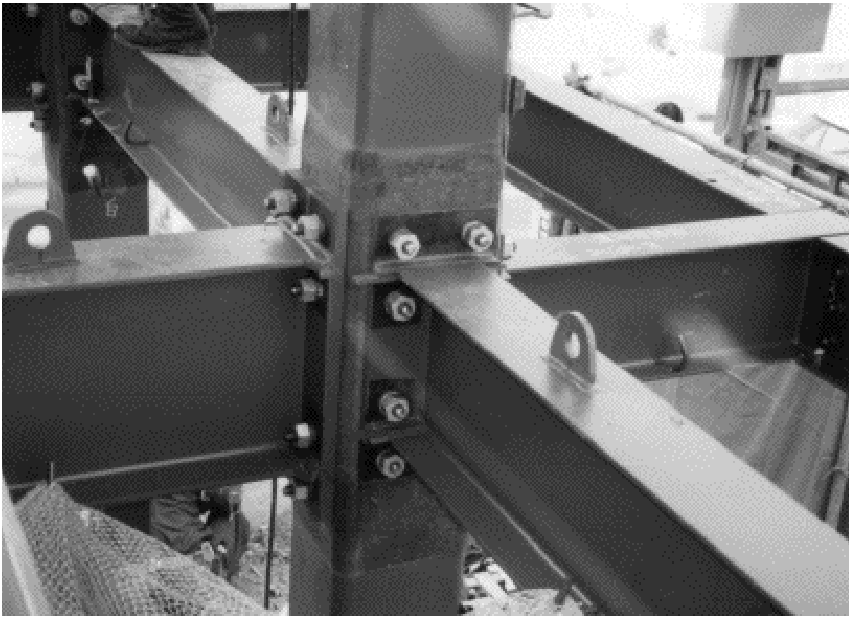


Figure 8.22 – End plate connection with blind bolts

Two proprietary devices were developed to realise these connections. One of the devices concerns the development of new blind bolts, which had to be not only strong enough to carry bending moments at the beam ends but also easy to install and tighten at erection sites. The new blind bolts being used for these connections are those called MUTF (Metric Ultra Twist Fastener) manufactured by Huck International Inc. (Huck 1994, see section 3.3). MUTF20 and MUTF27 bolts were used for the development of these connections. The number 20 or 27 represents the nominal diameter of the bolts. MUTF bolts are roughly equivalent in strength to ASTM A325 bolts with the same nominal diameter. However, the actual diameter of the bolt heads and sleeves of these bolts is greater than the nominal diameter. The diameter of bolt holes is thus specified as 2 to 4 mm greater than the nominal diameter to allow bolt heads to go through.

The other device is for manufacturing partially thickened RHS. Hollow sections are heated by electromagnetic induction to a temperature at which material plastic flow can easily occur. The ends of the RHS are pushed by a hydraulic ram. While the induction coil travels towards the predetermined end point, water is sprayed just behind the coil to cool the sections for preventing buckling of the tube walls (see figure 8.23). The tube walls are tapered at the junctions between the thickened walls and the walls with original thickness.

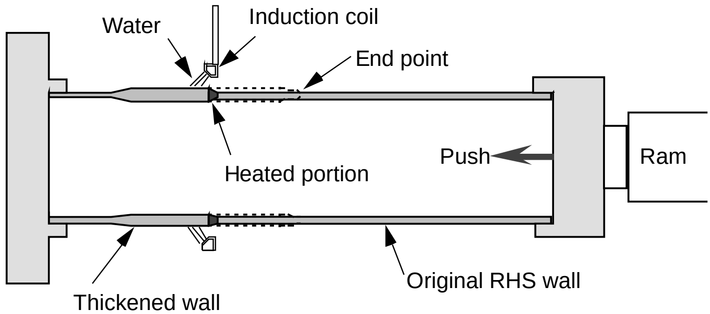



Figure 8.23 – Manufacturing process for partially thickening sections

The rapid heating and cooling processes cause the tube material to be hardened. The thickened sections are post-heat treated to restore material properties. Since the RHS used for this type of construction is cold-formed, the original sections show a high yield stress to ultimate tensile strength ratio. The post heat treatments bring about improved material properties (e.g. higher toughness and greater ductility) as compared with original cold-finished sections. Material properties of thickened sections with and without post-heat treatments are reported elsewhere (Tanaka et al. 1996).

The standard connections shown in table 8.4 were officially approved by the Japanese Ministry of Construction as the moment resisting connections that fulfilled the requirements specified in the Japanese building code. Table 8.4 shows dimensions of the end plate, nominal size and number of bolts for each combination of beams and columns.

| | Square hollow column section | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | $b_c$ = 200 | | 250 | | | 300 | | | 350 | | | 400 | | | |
| | $t_c$ = 9 | 12 | 9 | 12 | 16 | 12 | 16 | 19 | 12 | 16 | 19 | 12 | 16 | 19 | 22 |
| Rolled beam section | Partially thickened column section | | | | | | | | | | | | | | |
| | $b_c$ = 209 | 206 | 259 | 262 | 258 | 312 | 316 | 313 | 362 | 366 | 363 | 412 | 416 | 413 | 410 |
| $h_b x b_{b,f} x t_{b,w} x t_{b,f}$ | $t_c$ = 18 | 18 | 18 | 24 | 24 | 24 | 32 | 32 | 24 | 32 | 32 | 24 | 32 | 32 | 32 |
| 200X100X5.5X8 / 198X99X4.5X7 | 400x175x22[1] | | 400x200x22[1] | | | | | | | | | | | | |
| 250X125X6X9 / 248X124X5X8 | 450x175x22[1] | | 450x200x22[1] | 450x200x19 | | 450x250x19 | | | 450x300x19 | | | 450x350x19 | | | |
| 300X150X6.5X9 / 298X149X5.5X8 | 500x175x22[1] | | 500x200x22[1] | 500x200x19 | | 500x250x19 | | | 500x300x19 | | | 500x350x19 | | | |
| 350X175X7X11 / 346X174X6X9 | 550x175x28[2] | | 550x200x28[2] | 550X200X28 | | 550x250x19 | | | 550x300x19 | | | 550x350x19 | | | |
| 400X200X8X13 / 396X199X7X11 | | | | 600x250x28 | | 600x250x28 | | | 600x300x28 | | | 600x350x28 | | | |
| 450X200X9X14 / 446X199X8X12 | | | | 650x200x36 | | 650x300x28 | 650x250x32 | | 650x300x28 | | | 650x350x28 | | | |
| 500X200X10X16 / 496X199X9X14 | | | | | | 700x300x28[3] | 700x250x32[3] | | 700x300x28[3] | | | 700x350x28[3] | | | |
| 600X200X11X17 / 596X199X10X15 | | | | | | | 800x300x32[3] | | | 800x300x32[3] | | | 800x350x32[3] | | |

The number and nominal size of bolts are: 8-MUTF27 except that 1)8-MUTF20; 2)10-MUTF20 and 3)10-MUTF27

Table 8.4 – Standard details of end plate connections

A series of tests were conducted on beam-to-column assemblies with the standard connections (Fukuda and Furumi 1997). Anti-symmetrical shear loads were applied to cruciform specimens as shown in figure 8.24. The maximum loads were governed by combined local and lateral buckling of the beams, showing satisfactory rotation capacity. One of the advantages of these connections over other connections with diaphragms is that flexural moments in the beam web can be transferred to the column more easily.

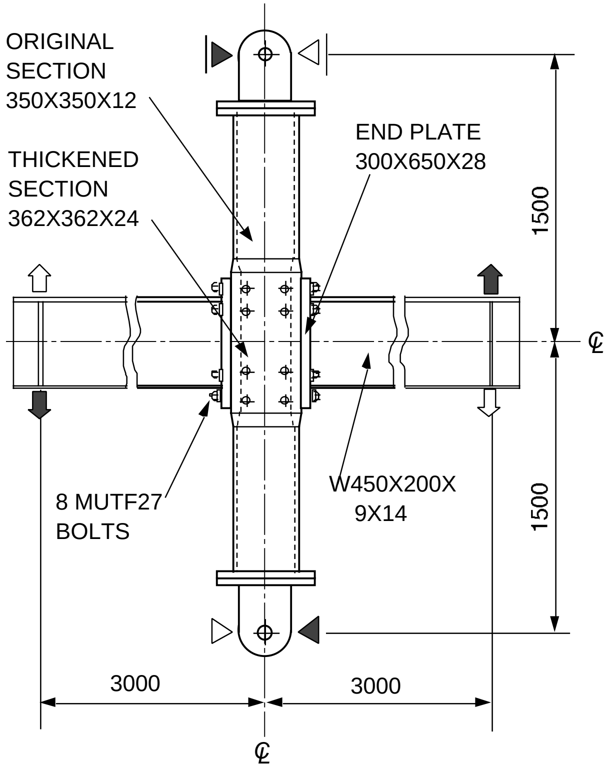



Figure 8.24 – Schematic diagram for anti-symmetric loading

Fukuda and Furumi (1997) emphasise the following advantages of these connections over conventional connections:

1. reduction in time to be taken to fabricate and erect a structure;

2. flexibility to accommodate beams with different depths framing into each column;

3. avoidance of brittle fracture.

Further details about the connection design can be obtained at Daiwa House Industry Co. Ltd., Sakyo 6-6-2, Nara 631-0801, Japan, phone: +81-742-70-2143, fax: +81-742-72-3064

## 8.8 Rigid connections for structures in low seismicity zones

Connection designs described in the preceding sections are usable also for structures for which the design forces resulting from earthquake motions are not governing. However, the overstrength requirements that have to be met by these connections make all of them uneconomical. It may be possible to simplify connection details if the ductility demands for structures are lower (see section 7.1). The FEMA criteria (2000) allow to use connections to I-section columns with no reinforcement, other than weld metal, for Ordinary Moment Frame applications. Nevertheless, investigations on those full strength moment connections between hollow section columns and beams, which are suitable for structures in low seismicity zones, are less.

When plastic global analysis shows that the required rotation at a beam end is smaller than those postulated in table 7.1, the overstrength factor $\alpha$ can be reduced accordingly. When a class 2 section is used for a beam, the resistance of the cross section is limited by its local buckling resistance. In this latter case the beam can be taken as capable of developing its fully plastic moment resistance (see Eurocode 3 Clause 5.3), and therefore,

$$\alpha = \frac{\gamma_{Mweld}}{\gamma_{M0}} = 1.1 \qquad \text{.......} 8.28$$

can be proposed. The reduction in $\alpha$ gives a significant influence on the required dimensions of the connection with the external diaphragms described in section 8.6.

Packer and Henderson (1997) have collated design guidance for some connections usable in low seismicity zones, from which one example connection is reproduced in figure 8.25. The connection has doubler plates reinforcing the RHS column faces.

Based on analytical and experimental investigations, the following 4 failure modes were identified:

a) "effective width" rupture of the beam tension flange plate where it is welded to the doubler plate;

b) punching shear failure of the doubler plate at the beam tension flange plate;

c) web crippling of the column side walls near the beam compression flange;

d) punching shear of the column face along the edge of the doubler plate, either near the beam tension flange or near the beam compression flange.

The above failure modes are very similar to those observed in truss joints of RHS members, which were studied extensively in the past (Wardenier 1982, Packer et al. 1992). Further examples of beam-to-hollow section column moment connections, for quasi-static load conditions or low seismicity areas, are given by Packer and Henderson (1997).

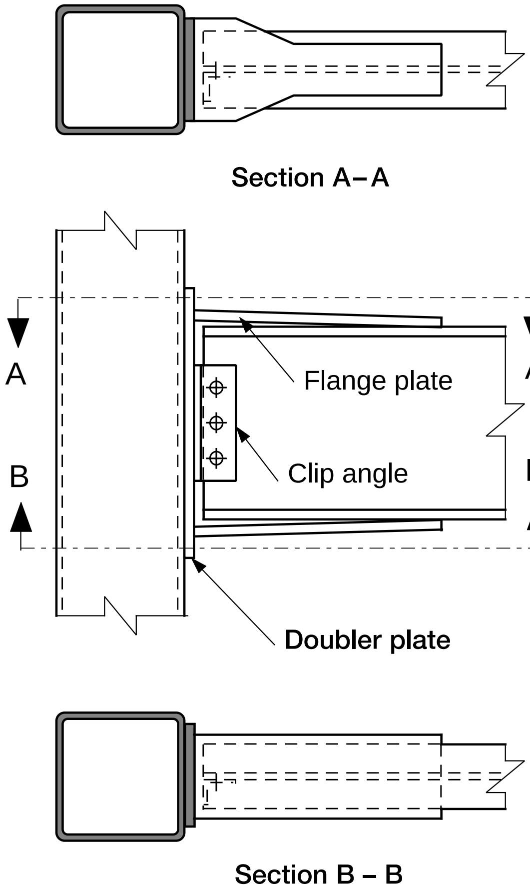



Figure 8.25 – Beam to column connection with doubler plates

# 9 Connections to concrete filled columns

## 9.1 Introduction

Concrete filling of hollow steel section columns is a procedure sometimes undertaken to enhance the column compressive resistance or increase the fire resistance of the column. CIDECT Design Guide No. 4 (Twilt et al. 1995) is a valuable resource for evaluating the fire resistance of hollow section columns, covering bare tubular columns, tubes painted with intumescent “paints”, water-filled tubes and concrete filled tubes (with and without additional steel reinforcement). A very thorough and contemporary treatment on the design of concrete filled hollow section columns, based principally on Eurocode 4, is given in CIDECT Design Guide No. 5 (Bergmann et al. 1995). Another international perspective on concrete filled tubular column design has also recently been compiled by the Association for International Cooperation and Research in Steel-Concrete Composite Structures (1997). Advice on the placement of concrete inside hollow section members is given in CIDECT Design Guide No. 7 (Dutta et al. 1998). One of the most important aspects to note is that small vent holes must be drilled through the walls of any hollow section filled with concrete, at either end of the member (or above and below each floor level), regardless of the design function of the concrete filling. These holes are to release the steam generated in the event of a fire and prevent the column from bursting.

## 9.2 Simple shear connections

### 9.2.1 Load entry to the column

In the design of composite columns, full composite action of the cross-section is assumed. This implies that there is a good bond between the steel and concrete and no significant slip occurs between the two, hence strain compatability exists between the steel and concrete. An approximate upper limit for this “natural bond” stress, recommended by Eurocode 4, is 0.4 $N/mm^2$ (Wardenier 2002). Many structural codes stipulate that the concrete be loaded directly in bearing, thereby requiring expensive detailing, whereas cheaper connections would be produced if beam shear attachments could be made just to the exterior of the column, without any penetration of the tube to provide direct bearing on the concrete. A number of research studies have investigated the concrete-to-steel tube bond strength, plus the decrease in composite column strength, as a result of part of the load being applied via intermediate shear connections. At such connections a modest amount of joint rotation takes place and the steel tube “pinches” onto the concrete core. These investigations have been reviewed by Dunberry et al. (1987) who then in turn conducted a further extensive experimental study of concrete filled RHS columns loaded through shear joints typical of those found in practice. Using stocky (not “slender” walled) cross-section columns, single shear plate and tee connections, it was concluded that the load transfer mechanism to the column was primarily affected by the rotational behaviour of the joint (and thus the ability to produce lateral bearing forces on the concrete), the length of the connection along the column, the load eccentricity and conditions in the column above the connection. In summary, it was recommended that a reduction factor be applied to the concrete strength, for determining the composite column capacity, for all types of simple shear connections to RHS which do not involve penetration of the hollow section (e.g. “through-plate” connections). This concrete core strength reduction factor $\alpha_{c,2}$ was given by:

$$\alpha_{c,2} = 1 - 1.2\zeta[\alpha_{c,1} A_{c,c} f_c / (A_c f_{c,y} + \alpha_{c,1} A_{c,c} f_c)] \quad \text{......................................9.1}$$

where $\alpha_{c,1} = 0.85$ and $\zeta$ is the ratio of the (factored) load applied at the shear connection, considering all sides of the column, to the total column (factored) load. $f_c$ is the 28-day cylinder compressive strength of the concrete, which is approximately 0.8 of the cube compressive strength. Thus, considering the concrete filled RHS column ABCD in figure 9.1 for example, this reduction factor $\alpha_{c,2}$ could be applied at section X-X when determining the resistance of the composite column AB.

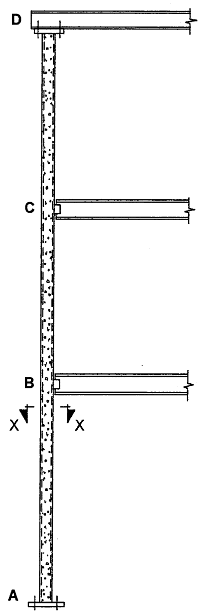



Figure 9.1 – Recommended method for introducing beam shear reactions to concrete filled columns at roof and floor levels.

Dunberry et al.(1987) also noted that slip between steel and concrete could be expected at the shear connection and extending for a distance of approximately $3b_c$ to $3.5b_c$ below, and $b_c$ to $2b_c$ above, the connection. For shear connections at the top of a column, grouting and providing a steel cap plate (to ensure a beneficial load transfer mechanism) was also recommended. Some structural steel specifications (for example CSA 1994) hence conservatively require that the concrete core be loaded directly in bearing for the uppermost level. At all lower levels the simple shear connections from the beams can be made directly to the outside of the hollow section, with no shear connectors within the hollow section. This is illustrated in figure 9.1. Since the primary cause of failure in the tests by Dunberry et al. (1987) was local buckling of the RHS below the shear connection, but only after yielding had been obtained, the foregoing recommendations are applicable only to non-slender RHS cross-sections.

Subsequent research publications on this topic have also confirmed that the “pinching effect”, produced by loading a concrete filled hollow section via welded shear attachments, has a beneficial effect on the slip resistance of the steel-concrete interface (Shakir-Khalil 1993). If a “blind bolting” connection technique was used, for example when bolting an element such as a beam end plate to a concrete-filled RHS column (France et al. 1999a), the protrusion of the “blind bolt” beyond the inside face of the RHS would act as a mechanical shear connector, thereby further increasing the steel-to-concrete slip resistance.

### 9.2.2 Connection design

Section 9.2.1 dealt with the effect of simple shear connections on the performance, and thus the design, of the concrete filled hollow section as a column member. For the design of the connection itself, it is generally recommended that the same criteria as given in chapter 5 (for hollow sections without concrete filling) be used. Concrete filling of the hollow section column prevents inward deformation of the column face, so the one column face rotational failure mode identified in chapter 5 (for just stiffened seat connections, in section 5.9) need not be considered with concrete filled columns. However, there is one important provision to these recommendations relating to fire conditions.

If a concrete filled hollow section column has complete external fire protection, then simple shear connection design may follow the guidance given above. If the concrete core is used for full or partial fire resistance, then the steel and concrete will expand at different rates in a fire situation, with the steel shell softening and shedding load. When this happens it would be unwise to rely on friction or bond at the steel-concrete interface to transfer load into the concrete. Hence, in such situations a "through-plate" simple shear connection (see figure 5.6) is recommended, so that beam reactions will be transferred reliably into the concrete core during a fire (Kodur and MacKinnon 2000).

## 9.3 Semi-rigid connections

### 9.3.1 Introduction

All the semi-rigid connections dealt with in chapter 6 can also be used with the columns filled with concrete, however the connection properties will change. In general the strength and stiffness will increase but in many cases the rotation capacity will decrease. The compression side of the connection at the column face will act as a stiff part since the loads are resisted by the concrete infill of the column. At the tension side the column face can only marginally deform and the deformations are generally not sufficient to allow a yield line pattern resulting in a punching shear failure at relatively small deformations and a small deformation capacity. As a consequence several connections, which with unfilled columns behave as semi-rigid partial strength connections, after filling with concrete behave as rigid (partial strength) connections.

A reduced deformation or rotation capacity has as a consequence that the connections are more sensitive to secondary bending moments, for example caused by induced deformations due to settlements. In case of a small rotation capacity which does not allow redistribution of bending moments only an elastic design approach is allowed.

In this chapter the connection types will be discussed in the same sequence as in chapter 6 although the available information is considerably less.

### 9.3.2 Unreinforced welded hollow section beam and column connections

These connections may occur in particular frames or Vierendeel girders. Concrete filling of the column or chord is generally only used for repair purposes, for example if the strength or stiffness of the connection without concrete filling is not sufficient and the connection configuration cannot be changed anymore.

#### 9.3.2.1 CHS beam and column members

Research on connections between circular hollow sections with the through member (chord or column) filled with grout or concrete has especially been carried out in order to strengthen existing offshore platforms. Several research programmes, however, are confidential and the results are not available or have been published with insufficient information for a full interpretation, e.g. the EC Composite Jacket Project, Tebbett et al. (1979 and 1982). Some information can also be found in Lalani et al. (1985,1996) and Marshall (1979) although most of this work is related to the effect of concrete filling on the stress concentration factors for fatigue design.

Makino et al. (2001) report the results of an investigation on axially loaded X- and K-joints. These investigations showed that the connection strength for tension can be based on the chord punching shear criterion, as given in figure 6.2. For compression loading no joint failure was recorded.

Recently, Morahan and Lalani (2002) have given additional information for the reduction in stress concentration factor for fatigue design of grouted joints and the comparison of some test data with the ISO (1997) formulations. It was found that there exists a good agreement.

Based on these investigations it seems to be acceptable to design moment connections of circular hollow sections on the basis of the punching shear criterion given in figure 6.2. It is expected that this is conservative because due to the stiffness of the concrete the moment resistance arm is longer than in the case of an unfilled chord or column.

No design formulae exist for the rotational stiffness, but if the compression loaded side is assumed to be rigid, then the stiffness is about twice that of the unfilled counterparts.

#### 9.3.2.2 RHS beam and column members

Packer et al. (1991, 1993) describe investigations on axially loaded X- and K-joints. Based on these investigations they give recommendations for axially loaded T-, Y-, X- and K-joints. For compression-loaded T-, Y- and X-joints they propose a strength function based on the concrete bearing strength and for tension-loaded T-, Y- and X- joints they recommend the same strength as for the joints without concrete filling of the chord.

For K-joints the strength can be based on a concrete bearing strength criterion for the compression brace and for the tension brace on chord punching shear and the brace effective width criterion.

For moment connections Szlendak (1998) presents data, however, these data could not be verified; and further no test evidence is available.

Based on investigations on axially loaded joints and the findings for the CHS-to-CHS connections it seems also logical to base the strength of moment connections on the column punching shear strength criterion and the brace effective width criterion of the connection without concrete filling. Thus, a smaller moment arm is assumed than the actual one, however, due to the constraining effect of the concrete, the local deformation capacity for punching shear may be lower resulting in an actual lower effective width $b_e$ and $b_{ep}$. These effects may compensate each other. The formula for the brace effective width criterion is given in figure 6.5.

Punching shear can only occur if $b_b \leq b_c - 2t_c$. The punching shear criterion can be obtained by assuming the punching shear area along the sides of the beam to be fully effective and along the top and bottom flange an effective punching shear width $b_{ep}$ is used, with:

$$b_{ep} = \frac{10}{b_c/t_c} b_b \qquad \text{.......... 9.2}$$

Similar to the CHS-to-CHS connections, no reliable design formulae exist for the rotational stiffness but also here the stiffness is considerably larger than that of the unfilled counterparts. Also here a factor of two seems to be acceptable.

### 9.3.3 Unreinforced welded I-beam-to-hollow section column connections

The most common moment connections with concrete filled columns are those with an I-beam. Many investigations have been carried out in Japan but most of them are mainly dealing with stiffened CHS columns and are dealt with in section 9.4. Morita (1994) reports on work in Japan on unstiffened connections with concrete filled RHS columns. As indicated in chapter 6, de Winkel (1998) and Lu (1997) investigated also the effect of concrete filling on the connection strength.

#### 9.3.3.1 I-beam-to-CHS column connections

De Winkel (1998) investigated a welded connection (figure 6.10) with a plate and a concrete filled column, however for tension loading a punching shear failure occurred at a load just above the yield load of the plate but below the punching shear strength according to the formula given in figure 6.11, thus no real conclusion could be drawn. On the other hand the ultimate loads for the tests with one flange plate as well as for two flange plates (see figure 6.10) were more than two times the loads observed for the connections without concrete filling.

In the case of compression loading the yield load of the plate could just about be obtained.

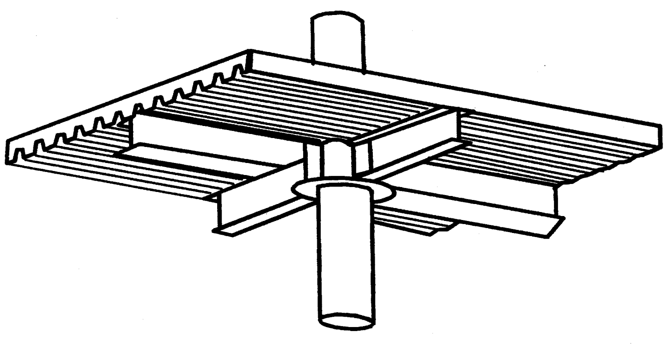


Figure 9.2 – Connection of an I-beam with a concrete slab to a (concrete filled) CHS column

Chiew and Dai (1999) and Mulia et al. (2001) investigated these connections in combination with a composite slab and concluded that a model based on the resistance of the reinforcement of the composite slab with a moment arm equal to the distance between the reinforcement and the bottom beam flange is too conservative. The capacities based on this model underestimated their test results considerably. Based on their tests they determined a parametric equation for the resistance.

Based on the investigations it is proposed here for connections between I-beams and concrete filled CHS columns to use for compression loading the yield load of the flange provided that $f_{b,y} \cdot t_b \leq f_{c,y} \cdot t_c$. For the flange loaded in tension the strength can be based on the punching shear strength.

In those cases where the rotation capacity is very important it is recommended to make the connection stronger than the connected beam.

#### 9.3.3.2 I-beam-to-RHS column connections

Morita (1994) has analysed the behaviour of semi-rigid composite connections with a concrete slab with the I-beam welded to the RHS column. He concludes that the strength for tension loading can be based on a modified yield line model for the column face. He assumes that parts of the beam flanges at the sides are yielding with the consequence that the yield lines in the column face parallel to the column sides are located at a distance smaller than the beam flange width. However, the tests used for verification all have a beam flange thickness considerably smaller than the column thickness. Using a larger beam flange thickness may result in punching shear failures as observed by Lu (1997) for the tests with a plate connected to a concrete filled RHS. She also observed for tension loading a plastification of the chord face but directly followed by cracks in the chord face parallel to the sides of the column. Thus the punching shear failure pattern differs from that of the unfilled counterpart, although the strength was in agreement with that calculated with the punching shear criterion for connections with unfilled chords which is given by:

$$N_b^* = \frac{f_{c,y} t_c}{\sqrt{3}} (2t_p + 2b_{e,p}) \qquad \text{......9.3}$$

with $b_{e,p}$ according to equation 9.2. It must be born in mind that for this test with $\beta = 0.4$ the strength of all criteria (chord face plastification, chord punching shear and plate effective width, see also figures 6.18 and 6.21) were very close to each other.

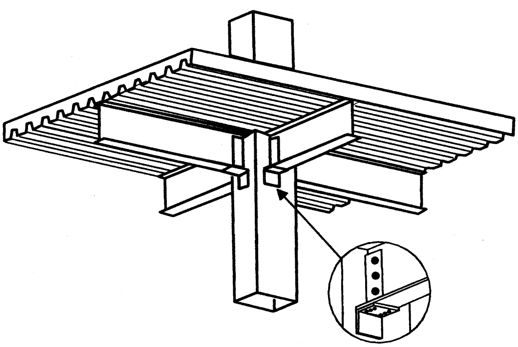


Figure 9.3 – Connection of an I-beam with a concrete slab to a (concrete filled) RHS column

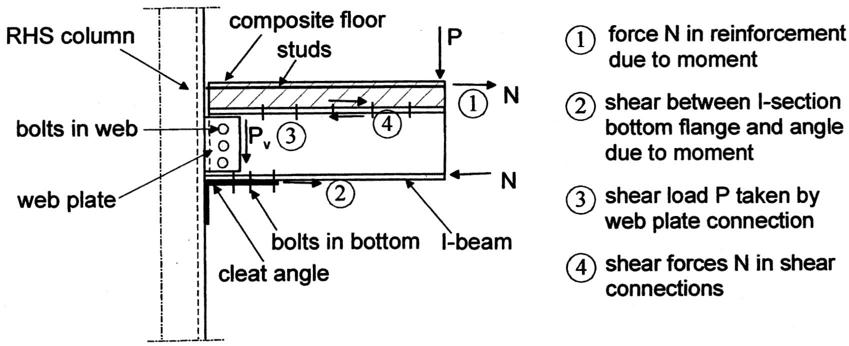



Figure – 9.4 Resisting forces in the connections of figures 9.2 and 9.3

Considering the above-mentioned criteria, it is recommended to design the tension side of the connection similar to that for a connection with no concrete filling of the columns, shown in figure 6.21. This gives a larger reserve in strength than for the connections with unfilled columns, which is recommended here because the deformation capacity decreases. It must further be noted that the influence of the column loading on the connection strength has not been investigated, but it is expected that the influence is not as high as given by the function f(n) in figure 6.21. In figure 6.21 the punching shear criterion (equation 9.3) is not given, but if the beam flange thickness exceeds the column thickness this criterion should also be checked.

### 9.3.4 Bolted hollow section beam and column connections

Bolted connections between hollow section members are generally made with flange plates, gusset plates, angles cleats or cut-outs of open sections as discussed in section 6.4. Filling of the columns with concrete does not principally change the design approach, thus reference is given to section 6.4.

### 9.3.5 Bolted I-beam-to-hollow section column connections

Bolted connections with plates or stubs welded to the columns are principally similar to the welded connections already discussed.

#### 9.3.5.1 I-beam-to-CHS column connections

Most of the bolted I-beam-to-CHS column connections have a stiffening plate through or around the column to which the beam flange is connected. These connections generally behave as rigid and are discussed in chapter 8.

Sometimes a welded stub equal to the connected beam is welded to the column and the beam is bolted with plates to this stub. This connection is comparable to the welded connection dealt with in section 9.3.3 and shown in figure 6.11.

De Winkel (1998) investigated a bolted connection (figure 9.2) where the beam web is connected by bolts to a shear plate welded to the column, the bottom beam flange is welded to a ring welded around the column, and the top flange is provided with shear connectors for the connection with the concrete floor slab. Here, the resistance of the floor slab reinforcement or the connection resistance of the bottom flange determines the moment capacity of the full connection. If the resistance of the bolted bottom flange connection is stronger than the resistance of the reinforcement, there is no difference with the strength of the unfilled counterpart. If the reinforcement is governing, the moment capacity of the full connection is determined by it and it makes the design easy, however in this case the reinforcement should have sufficient ductility to provide sufficient rotation capacity. The slip of the bolted connection may influence the connection stiffness.

#### 9.3.5.2 I-beam-to-RHS column connections

Similar to the connections with CHS columns sometimes a welded stub equal to the connected beam is welded to the column and the beam is bolted with plates to this stub. This connection is comparable to the welded connection dealt with in section 9.3.3 and shown in figure 6.21.

Lu (1997) investigated a bolted connection (figure 9.3) where the beam web is connected by bolts to a shear plate welded to the column, the bottom beam flange is bolted to an angle welded to the column face, and the top flange is provided with shear connectors for the connection with the concrete floor slab. Similar to the connection with a CHS column, the resistance of the floor slab reinforcement or the connection resistance of the bottom flange determines the moment capacity of the full connection. Slip of the bolted connection may influence the connection stiffness.

France et al. (1999) studied the moment-capacity and rotational stiffness of flush and extended endplate connections to concrete-filled tubular columns with flowdrilled connectors. Due to the concrete filling the axis of connection rotation shifted towards the compressive flange of the beam. At the tension side the plasticity is confined to the column face only and a yield line pattern is formed. Compared to the unfilled column connection there is a considerable increase in stiffness, depending on the column face slenderness. The strength of these connections can thus be based on the capacity of the tension zone as discussed in section 6.5.2 with a moment arm to the bottom beam flange.

The research in this field has demonstrated that filling of the column with concrete increases the moment capacity and stiffness considerably in such a way that sometimes semi-rigid partial strength connections may become semi-rigid or rigid partial or full strength connections. Although the moment capacity can be quantified the stiffness cannot yet be given in design formulae. In those cases where it is required the designer has to make use of the moment rotation diagrams in literature.

### 9.3.6 Examples

Similar examples as given in section 6.6 could be given here. The procedure is the same only the stiffness, the design capacity and the deformation capacity differ. The stiffness of connections with concrete filled columns is roughly two times that of the unfilled counterparts. In most cases column face plastification is not the governing strength criterion but punching shear.

Since the deformation capacity of the connections has been reduced considerably these connections should be carefully designed. The best way is to design these to be stronger than the connected beam (full strength) or as pin-ended connections with sufficient deformation capacity.

## 9.4 Rigid (full strength) connections

The concrete-filled hollow section column is frequently a preferred option to the plain column for seismic design, because increased strength of the column due to confinement of concrete as well as continuous bracing of the steel tube to delay local buckling and enhanced ductility of the column in cyclic loading can be fully utilised. Full-strength beam-to-concrete-filled column connections are required to make the best use of the structural merits of these columns. The following guidelines for the connection design are based on the AIJ standard for steel reinforced concrete structures (AIJ 2001a), although a few modifications are made by taking results of more recent studies into account.

Eurocode 4 and the AIJ standard use different approaches for the calculation of the strength of concrete-filled tubular members. The latter adopts the method of superposition which postulates that the ultimate strength of a member is given by the sum of the ultimate strengths of the concrete part and steel part. The superposition method requires simpler calculations than the other method considering composite actions between the concrete and steel parts. According to the plastic theory, the superposition method gives a lower-bound solution for a structure made of ductile materials. However, this method may produce an error on the unsafe side if applied to composite members, because concrete is not sufficiently ductile. Therefore, the strength of composite members evaluated using the superposition method should always be verified by comparing it with experimental results.

The method of superposition was found effective and used extensively to calculate the strength of composite connections, as will be shown in the following guidelines.

### 9.4.1 Shear strength of column web panel

The design shear strength for the column web panel $V_{c,w}^*$ can be calculated by

$$V_{c,w}^* = 1.2\left(A_{c,p} \cdot \beta \frac{f_c}{10} + A_{c,w} \frac{f_{c,y}}{\sqrt{3}}\right) \qquad \text{.......9.4}$$

where for CHS columns $\beta = 2 \dfrac{h_{c,w} - 2t_{c,w}}{h_b - 2t_d} \le 4.0$

and for RHS columns $\beta = 2.5 \dfrac{h_{c,w} - 2t_{c,w}}{h_b - 2t_d} \le 4.0$

In the above equation $A_{c,p}$ and $f_c$ denotes the cross-sectional area of the concrete panel and the cylinder strength of the concrete, while $\beta$ is a function of the depth to height ratio of the concrete panel. See section 7.7 and figure 9.5 for the meaning of the other symbols.

The above formula was derived from the yield strength of the panel multiplied by a factor of 1.2 to convert it to the ultimate limit state design strength. The yield strength was evaluated simply as the sum of the yield strengths of the concrete and steel parts, where the concrete part carries the shear load as a diagonal compression strut, whose strength was assumed to be governed only by the compressive strength of concrete (see figure 9.5).

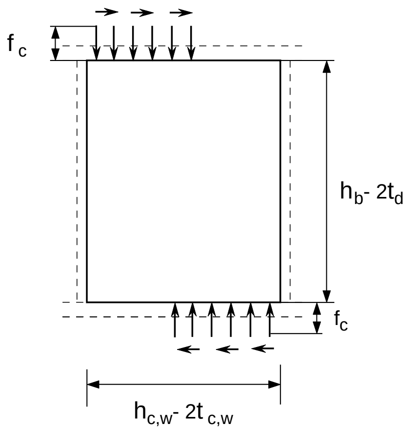



Figure 9.5 – Stresses in concrete panel

The yield strength equation was first proposed more than 20 years ago (Kurobane 1978). Extensive test results achieved after that, however, showed no need to improve the original formula. Equation 9.4 predicted the test results with a constant safety margin but showed no systematic tendency when the concrete strength, width-to-thickness ratio of steel and connection details (external and internal diaphragms) were varied.

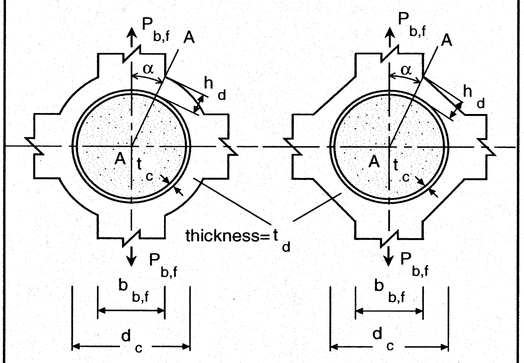
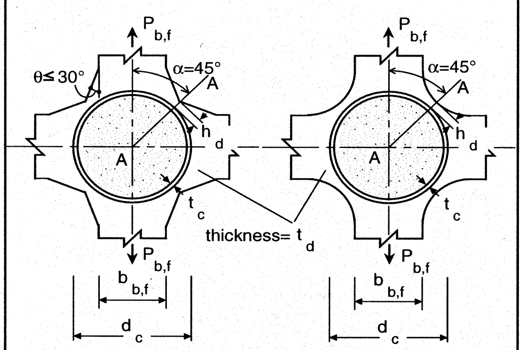
Equation 9.4 involves a few modifications to the design formula of the AIJ standard (2001a). The AIJ formula underestimates the ultimate strength of the panel by about 60 % for CHS panels and 30 % for RHS panels. The AIJ formula prescribes that the shear capacity of the concrete core decreases with the concrete strength. This effect was ignored, which gave no unsafe error (Fujimoto et al. 2001, Kawano and Matsui 2001) for the range, $f_c \leq 60$ N/mm$^2$, over which the AIJ standard is applicable. Further, the shear capacity of the CHS panel was increased by 27 % as a result of adopting the new formula (equation 7.4) specified by the AIJ Recommendations (2001).

The column web panel can show a hardening envelope and stable hysteresis loop after reaching the load given by equation 9.4. This contributes to dissipating energy input during strong earthquake motions.

The shear capacity of the column web panel decreases as the axial load in the columns increases. However, the shear load applied to the panel also decreases because the flexural capacity of the columns decreases with the axial load. Equation 9.4 allows a safe margin for the effect of the axial load unless the column web panel is designed to have a smaller cross section than the cross sections of the columns (see Kurobane 1978, AIJ 1990).

### 9.4.2 Flexural strength of beam-to-column connections

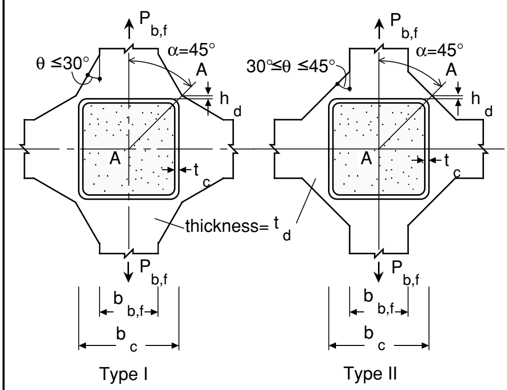
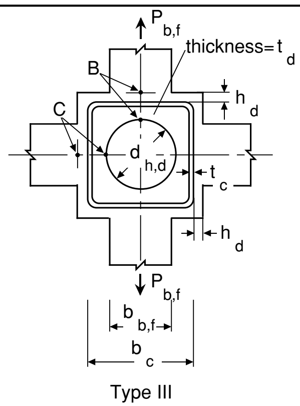
The connections with external, through or internal diaphragms, as described regarding connections to plain columns in chapter 8, are applicable to beam-to-concrete-filled column connections. When through or internal diaphragms are used, holes should be prepared to let concrete fill inside the columns and connection panels. Since the infilled concrete restrains local distortions of the column walls and diaphragms, these restraining effects can be taken into account when determining the dimensions and details of connections.

| Shape of external diagram | Ultimate strength equation |
|---|---|
| <br>Type I Type II | $P_{b,f}^* = 3.09f_1(\alpha)A_1f_{c,y} + 1.77f_2(\alpha)A_2f_{d,y}$ (1)<br>where<br>$f_1(\alpha) = \sin\alpha$<br>$f_2(\alpha) = \sqrt{2\sin^2\alpha + 1}$<br>$A_1 = \left\{(0.63 + 0.88\frac{b_{b,f}}{d_c})\sqrt{d_ct_c} + t_d\right\}t_c$<br>$A_2 = h_dt_d$<br>Symbols:<br>$f_{c,y}$ = Yield strength of column material<br>$f_{d,y}$ = Yield strength of diaphragm material<br>$P_{b,f}$ = Axil load in tension flange<br>$\alpha$ = Slope of critical section |
| <br>Type III Type IV | $P_{b,f}^* = 2.19A_1f_{c,y} + 2.53A_2f_{d,y}$ (2)<br>where<br>$A_1 = \left\{(0.63 + 0.88\frac{b_{b,f}}{d_c})\sqrt{d_ct_c} + t_d\right\}t_c$<br>$A_2 = h_dt_d$<br>Symbols:<br>See above. |
| Range of validity | |
| $20 \leq d_c/t_c \leq 50$    $h_d/d_c \leq 0.3$ | $0.25 \leq b_{b,f}/d_c \leq 0.75$ |

Note:
Symbols: b = Width d = Diameter h = Height t = Thickness θ = Slope of diaphragm
Subscript: b = Beam c = Column d = Diaphragm f=Beam flange

Table 9.1 - Ultimate resistance equations for connections with external diaphragms to concrete-filled circular columns (AIJ 2001a)

Tables 9.1 and 9.2 show ultimate limit state design formulae for CHS column connections with external diaphragms and those for RHS column connections with external and through diaphragms, both being extracted from the AIJ Standard (2001a).

| Shape of external diaphragm | Ultimate resistance equation |
|---|---|
|  | Type I connection:<br>The design resistance is the larger of the values calculated by Eqs. 1 and 2.<br>$P_{b,f}^* = 3.74\left(\frac{t_c}{b_c}\right)^{\frac{2}{3}}\left(\frac{t_d}{b_c}\right)^{\frac{2}{3}}\left(\frac{t_c + h_d}{b_c}\right)^{\frac{1}{3}} b_c^2 f_{d,u}$ (1)<br>$P_{b,f}^* = 2.86(4t_c + t_d)t_c f_{c,y} + 3.30 h_d t_d f_{d,y}$ (2)<br>If $f_{c,y} \geq f_{d,y}$, then calculate with $f_{c,y} = f_{d,y}$.<br>Type II connection:<br>The design resistance is given by Eq. 2 .<br>Symbols:<br>$f_{c,y}$ = Yield strength of column material<br>$f_{d.y}$ = Yield strength of diaphragm material<br>$f_{d,u}$ = Ultimate tensile strength of diaphragm material<br>$P_{b.f}$ = Axial load in tension flange |
|  | Type III connection:<br>The design resistance is the smaller of the values given by Eqs. 3 and 4.<br>$P_{b,f}^* = 1.43(b_c + 2h_d - d_{h,d})^2 \frac{b_{b,f} t_d}{d_h^2} f_{d,y}$ (3)<br>$P_{b,f}^* = 1.43(b_c + 2h_d - d_{h,d}) t_d f_{d,y}$ (4)<br>Symbols: See above. |
| Range of validity | |
| $20 \leq b_c/t_c \leq 50$, $0.75 \leq t_d/t_c \leq 2.0$, $t_d \geq t_{b,f}$, $h_d/b_c \geq 0.1 t_{b,f}/t_d$ (Type I), $h_d/b_c \geq 0.15 t_{b,f}/t_d$ (Type II) | |

Note:

Symbols: b = Width d = Diameter h = Height t = Thickness θ = Slope of diaphragm

Subscript: b = Beam c = Column d = Diaphragm f = Beam flange h = hole

Table 9.2 – Ultimate resistance equations for connections with external and through diaphragms to concrete-filled square columns (AIJ 2001a)

The ultimate resistances in these formulae are represented in terms of the axial tensile load $P_{b,f}^*$ at the beam end. Again these formulae were derived from the yield strength equations, which were multiplied by a factor of 1/0.7 to convert them to the ultimate resistance equations (see section 8.6).

Connections with external diaphragms to CHS columns are divisible into two groups: the first group includes types I and II while the second group includes types III and IV, as shown in table 9.1. For connections of types I and II a critical steel section on the line A-A through the centre of the column and the intersection between the beam flange and diaphragm was assumed. For connections of types III and IV a critical steel section on A-A through the narrowest section of the diaphragm was assumed. Each steel section is assumed to have a T section consisting of a cross section of the diaphragm with the height $h_d$ and a portion of the column wall with the effective width $b_e$ (see figure 9.6).

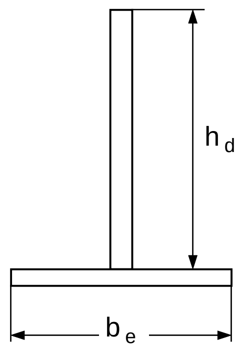



Figure 9.6 – Critical steel section through diaphragm and column wall

The yield strength of connections was equated to the resultant of the axial and shear strengths of these steel sections calculated by using the lower bound theorem of plastic analysis (Kurobane et al. 1987). The effective width $b_e$ was determined based on experimental results. These yield strength equations agree well with recent test results (Fukumoto et al. 2000, Kato 2001), except for a specimen in high-strength steel with yield strength of 748 N/mm$^2$.

Two design equations were provided for connections with external diaphragms to RHS columns (see table 9.2). Equation 1 in table 9.2 was derived from the ultimate strength formula for connections to plain steel columns (see equation 2 in table 8.3) but with the resistance factor greater than that required for connections to steel columns. This increased resistance factor is to take into account restraining effects given by the concrete core. Equation 2 in table 9.2 was derived from the yield strength equation, namely a lower bound solution of plastic theory following the procedure identical to that used for connections to CHS columns (see table 9.1). Attention should be paid to the fact that for connections to RHS columns (the connections of types I and II in Table 9.2) the critical steel section assumed lies on the line A-A through the narrowest section of the diaphragm irrespective of the connection type, and also to the fact that the height of diaphragms $h_d$ is defined differently between tables 9.1 and 9.2.

Equation 1 in table 9.2 tends to underestimate the restraining effect of the concrete core when $h_d/b_c$ becomes greater than about 0.15. This equation is applicable only to the connection of type I with a small $h_d/b_c$ ratio.

Equation 3 in table 9.2 was derived from the full plastic strength of a fictitious beam with the critical steel section B-B. Equation 4 in table 9.2 was derived from the full plastic strength of a fictitious tension bar with the critical steel section C-C.

The design provisions shown in table 9.2 were first proposed by Matsui (1981) but were found to compare well with the results of more recent experimental and numerical studies (Kawaguchi et al. 1997, Kawano et al. 1998).

The AIJ standard provides the design guides for connections with through diaphragms to CHS columns and for connections with internal diaphragms to RHS columns, besides those shown in tables 9.1 and 9.2. The design equations were derived from the results of a series of yield line analyses and tests (Fu and Morita 1998, Morita et al. 1991). The details are not shown here because these equations are rather complex.

Local buckling of plate elements always governed the ultimate load of the compression flange-to-concrete-filled column connections according to past tests. No damage in the welded joints or in the concrete was observed. Thus connections between the compression flanges and concrete-filled columns are not considered critical. No design formula has been prepared for these connections.

The flexural resistance of the connections shown in tables 9.1 and 9.2 can be calculated by equations 8.1 and 8.3 with

$$M_{b,f,u} = P_{b,f}^{*}(h_b - t_{b,f}) \quad \text{.......... 9.5}$$

Note that no formula exists to calculate the flexural capacity of welded web joints to concrete-filled columns. Equations 8.3 to 8.5 for steel columns can be substituted for the strength equations for concrete-filled column connections, although this introduces errors on the safe side (see also section 8.6). If one wants to ignore the flexural capacity of welded web joints, the formula for connections to plain columns (equation 8.22) can be used to determine the design strength of the connections to concrete-filled columns.

The required flexural capacity at the column face is given by equation 8.23, where $L_{haunch} = h_d$ for the connection of type III in table 9.2. An overstrength factor of $\alpha = 1.3$, rather than $\alpha = 1.2$, is recommended. The other requirements regarding joint details, fabrication and quality control are identical to those recommended for the connections with external diaphragms and through diaphragms to plain steel columns (see sections 8.1, 8.3 and 8.6).

Significant examples of full-strength connections to concrete-filled columns studied outside of Japan include split-tee bolted connections (Ricles et al. 1997). High-strength bolts that pass through the column, post-tensioned after the concrete was cured, carry the axial load in the beam flange. These connections achieved good ductility, demonstrating sufficient plastic rotation of the beams outside the connections and ductile shear deformation of the column web panels. The study is still in progress.

# 10 Bracing and truss connections to columns

The recommendations in sections 10.1, 10.2 and 10.4 are applicable for predominantly static loading. For bracing connections under seismic loading some guidance is given in section 10.3.

## 10.1 Bracing connections to RHS columns

### 10.1.1 Longitudinal plate-to-RHS columns

Longitudinally-oriented plates welded to I-section columns in “pin-jointed”, steel braced frames have been the traditional method for attaching brace members via a bolted site connection. Indeed, in braced frames such plates are extremely convenient for connecting one or more bracing members at any bracing member angle. This practice has been carried over to RHS columns, but the connection region is inherently more flexible with a RHS member. Figure 10.1 shows these two cases of longitudinal plate connections.

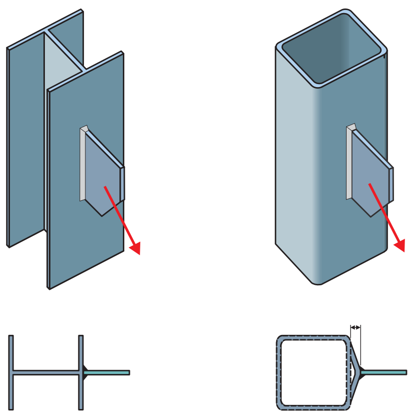


Figure 10.1 – Longitudinal branch plate connections in braced structures

For the case of the I-section column, the plate is welded at (or close to) the centre of the column flange so the load from the plate is transferred to the web of the column directly. The situation is different for the case of the RHS column because the load from the brace plate must be carried indirectly through the flexible column connecting face to the remote column webs. A conventional longitudinal plate-to-RHS member connection tends to result in excessive distortion or plastification of the RHS connecting face. Such a connection results in a low design resistance that is governed by the formation of a yield line mechanism. It is worth noting that the “chord face yielding” expression for both welded RHS T- or Y- connections, and also for welded longitudinal plate connections, as given in tables 2 and 12 of CIDECT Design Guide No. 3 (Packer et al. 1992), *cannot* be used directly because the reduction factor f(n) to take account of compressive loading in the main (column) member has been found to be inapplicable at such low branch plate-to-RHS width ratios (~ 0.05 to 0.25). However, a more recent extensive experimental, analytical and numerical study on longitudinal plate-to-RHS connections has resulted in validated limit states design procedures for these types of connections (Cao et al. 1998, 1998a; Kosteski 2001, Kosteski and Packer 2003a). The design guidelines are based on consideration of an ultimate deformation limit and a serviceability deformation limit, for the RHS connecting face.

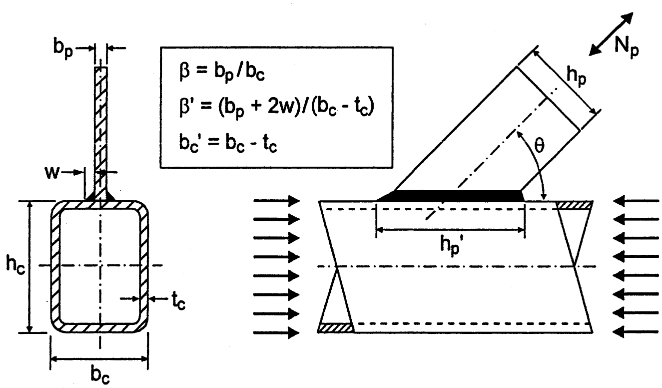



Figure 10.2 – General longitudinal plate-to-RHS member connection

The recommended design procedure can be summarised as follows, for longitudinal plate-to-RHS member connections as shown in figure 10.2. (Note that the direction of the branch plate load is inclined to the RHS column at an angle $\theta$. This implies that any resultant force orientation can be accommodated, hence any combination of shear force parallel to the column and load normal to the column, as discussed in section 5.1). It is applicable for both square and rectangular column members, loaded in axial compression, axial tension or axial load plus bending. Similarly, the design procedure is applicable to bracings (attached to the longitudinal plates) loaded in either axial tension or axial compression. The connection should be checked at both the factored load level and the unfactored (service) load level, as given below.

1. The factored resistance of the RHS connecting face, corresponding to a yield line mechanism in the column face, is given by $N_p^*$ (Cao et al. 1998, 1998a; Kosteski and Packer 2000, 2003a), where (see figure 10.2):

$$N_p^* = \frac{2 f_{c,y} t_c^2}{(1 - \beta')\sin\theta} \cdot \left( \frac{h_p'}{b_c'} + 2\sqrt{1-\beta'} \cdot \sqrt{1-n^2} \right) \qquad \text{.............................................10.1}$$

Equation 10.1 is a factored resistance expression and hence includes an implied resistance factor, $\phi$, of 1.0 (or alternatively an EC3 partial resistance factor of unity) for limit states design. This is because the connection is deflection-critical and the calculated yield load $N_p^*$ is well below the connection fracture load.

In equation 10.1, n is a factor to take into account the influence of the total normal stress in the column member, and is the ratio of the net applied normal stress to the yield stress ($\sigma_c / f_{c,y}$) in the RHS connecting face, where the applied normal stress is due to axial load and/or bending moment. For pure axial load in the column this is the axial force divided by the column cross-sectional area. If bending moment is also present in the column, the normal axial stress should be increased or decreased by the elastic bending stress at the plate joint. The stress $\sigma_c$ should be calculated on the side of the connection that produces the least unfavourable impact on the connection resistance (often termed the "preload" side). With the net applied normal stress in the RHS connecting face capable of being tensile (positive n) or compressive (negative n), and the bracing member axial load capable of being tension or compression, there are four possible scenarios for the sense of these loads. Kosteski (2001), by means of parametric Finite Element analysis, has shown that only one of the four possible loading combinations (namely, simultaneous branch plate tension combined with main member connecting face tension) would be considered overly conservative if designed using the $\sqrt{(1 - n^2)}$ reduction factor in equation 10.1. Thus, the use of the $\sqrt{(1 - n^2)}$ reduction factor is recommended for all load direction cases.

2. The serviceability load limit for the RHS connecting face, for use in checking the service or unfactored connection load, is given by $N_{p,s1\%}$. This can be determined from the factored resistance $N_p^*$ in equation 10.1, for various class (wall slenderness) categories of the RHS column, as follows (Kosteski 2001):

for class 1: $N_{p,s1\%} = N_p^*/(1.5 - 0.9\beta')$ ............................................10.2a

for class 2: $N_{p,s1\%} = N_p^*/(2.0 - 1.25\beta')$ ............................................10.2b

for class 3: $N_{p,s1\%} = N_p/(2.7 - 2\beta')$ ............................................10.2c

Notes: (a) Limit of applicability for equations 10.1 and 10.2 is $b_c/t_c \leq 40$
(b) It is acceptable, and more conservative, to use equations 10.1 and 10.2 by replacing $b_c$ for $b_c'$, $\beta$ for $\beta'$, and setting w = 0.

A comprehensive Finite Element study (Kosteski 2001) has concluded that the non-uniform stress distribution in the connecting branch plate (typically treated using an effective width factor) is of negligible consequence, for practical connections at the branch plate load level commensurate with the connection yield load $N_p^*$. Thus, a reduction in the branch plate effectiveness (or effective width) need not be considered in the branch plate design (or the design of the branch plate weld).

The foregoing design recommendations were derived from research in which the longitudinal branch plate was located along the RHS member axis. A slight variant is sometimes produced when the longitudinal branch plate is offset from the column centreline so that the centreline of the bracing member can coincide with that of the column. This should cause minimal difference in connection behaviour and this detailing arrangement is also acceptable.

### 10.1.1.1 Longitudinal plate-to-RHS column design example

A multi-storey steel-framed building has a storey height of 4.5 m, a bay width of 6.0 m and is composed of "pin-jointed" or "simple" connections throughout. Stability against lateral loading is achieved by X-bracing in one bay. The X-braces in a lower storey are CHS 89 x 3.8 Grade 350W (cold-formed hollow sections with a nominal yield stress of 350 N/mm$^2$) and the columns are square RHS 178 x 178 x 6.4, Grade 350W also. Under a particular load combination, the factored tensile force in a brace, $N^+$ = 250 kN and the factored compression load in the column at the connection, $N^-$ = 500 kN. (The corresponding unfactored or service load in the brace is $N_{un}^+$ = 167 kN). Check the viability of a longitudinal plate connection.

*Tensile resistance of CHS brace:*

$N^* = \phi_1 A_g f_y = (0.9)(1020)(0.350)$ CSA Specification

$= 321$ kN

or $= \phi_1 (0.85) A_e f_u = (0.9)(0.85)(1020)(0.450)$ CSA Specification

$= 351$ kN

A resistance factor of $\phi_1 = 0.9$ has been used for yielding, and an effective resistance factor of 0.9(0.85) = 0.77 for fracture.

∴ Limiting tensile resistance of brace ($N^*$) = 321 kN > $N^+$ = 250 kN. ✓

It is assumed that a thick T-stub is welded to the end of the CHS brace member and hence the effective net section of the CHS is the same as the gross section of the CHS. This is easily achievable (Packer and Henderson 1997), based upon a conservative load dispersion angle of 2.5 : 1 from each face of the T-stub web (see figure 10.3).

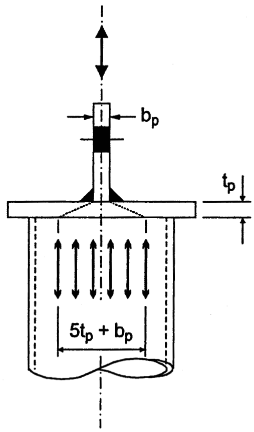



Figure 10.3 – Load dispersion for T-stub connection on the end of a CHS member

*Slenderness of CHS brace member:*

$$L_e/r = \frac{7500}{30.1} = 249 < 300.\ ✓$$ CSA Specification

These X-braces are relatively heavily loaded, but this is feasible for the bottom storey of a multi-storey building.

For the longitudinal plate, try a 200 x 10 mm plate, using hot-rolled steel with a nominal yield stress of 300 N/mm². The plate-to-column angle ($\theta$) is such that $\tan\theta = 6000/4500$, hence $\theta = 53.1°$. Bolt the brace end (T-stub web) to the longitudinal plate with 2 M22 A325M (Grade 8.8) bolts, located in 24 mm diameter drilled holes, and oriented in a line in the direction of the load.

*Shear resistance of bolts (threads excluded):*

$V_b^* = 2 \times 127$ kN　　CSA Specification

$= 254 \text{ kN} > N^+ = 250 \text{ kN}$

*Tensile resistance of longitudinal plate, due to yielding of the gross cross section:*

$N^* = \phi_1 A_g f_y = 0.9(200)(10)(0.300)$　　CSA Specification

$= 540$ kN

*Tensile resistance of longitudinal plate, due to net section fracture at the bolt hole:*

$N^* = \phi_1 (0.85) A_e f_u = 0.9(0.85)(200 - 24)(10)(0.450)$　　CSA Specification

$= 606$ kN

*Bearing resistance of longitudinal plate, at the bolt holes:*

$= 3\ \phi_3\ t_p\ d_b\ n\ f_{p,u}$　　CSA Specification

$= 3\ (0.67)(10)(22)(2)(0.450) = 398$ kN

$\therefore$ Limiting resistance of plate $= 398 \text{ kN} > N^+ = 250$ kN. ✓

*Weld resistance:*

For the fillet weld to the column, try a 6 mm (leg size) fillet weld using an electrode with an ultimate strength of the consumable = 480 N/mm$^2$. If one ignores the orientation of the load to the weld axis (which is conservative), a simple method for treating the weld gives a factored resistance of 0.914 kN/mm (CSA Specification).

$$\therefore \text{Weld resistance} = 2\left(\frac{200}{\sin 53.1°}\right)(0.914)$$

$$= 457 \text{ kN} > N^+ = 250 \text{ kN. ✓}$$

*Factored resistance of the RHS connecting face:*

$b_c/t_c = 178/6.35 = 28.0 \leq 40$ ✓

$$n = \sigma_c/f_{c,y} = \frac{-500/(4250)}{0.350} = -0.336$$

$w = 6$ mm

$$\beta' = \frac{10 + 2(6)}{178 - 6.35} = 0.128 \qquad h_p' = \frac{200}{\sin 53.1°} + 2(6) = 262 \text{ mm}$$

$$\therefore N_p^* = \frac{2(0.350)(6.35)^2}{(1 - 0.128)\sin 53.1°}\left(\frac{262}{171.6} + 2\sqrt{1 - 0.128}\,\sqrt{1 - 0.336^2}\right) \quad \text{eqn. (10.1)}$$

$= 133 \text{ kN} < N^+ = 250$ kN $\therefore$ no good

*Serviceability load limit for the RHS connecting face:*

Although the connection has already been proven inadequate, this final calculation is performed to demonstrate the procedure. This RHS 178 x 178 x 6.4 Grade 350W column can be shown to be a class 2 (CSA 1994) section, since:

$$\text{Slenderness of the “flat”} = \big((178 - 4(6.35)\big)/6.35 = 24.0 \le 525/\sqrt{f_y} = 28.1$$

$$N_{p,s1\%} = 133/(2.0 - 1.25(0.128)) \qquad \text{equation (10.2b)}$$

$$= 72 \text{ kN} < 167 \text{ kN} \;\therefore\; \underline{\text{no good}}$$

This example illustrates how this type of connection is really only suited to lightly-loaded braces, as the connection resistance is relatively low due to its high degree of flexibility. Hence, alternative, stronger connection arrangements need to be considered, such as:

- a transverse plate-to-RHS connection
- a through plate-to-RHS connection
- a stiffened plate-to-RHS connection.

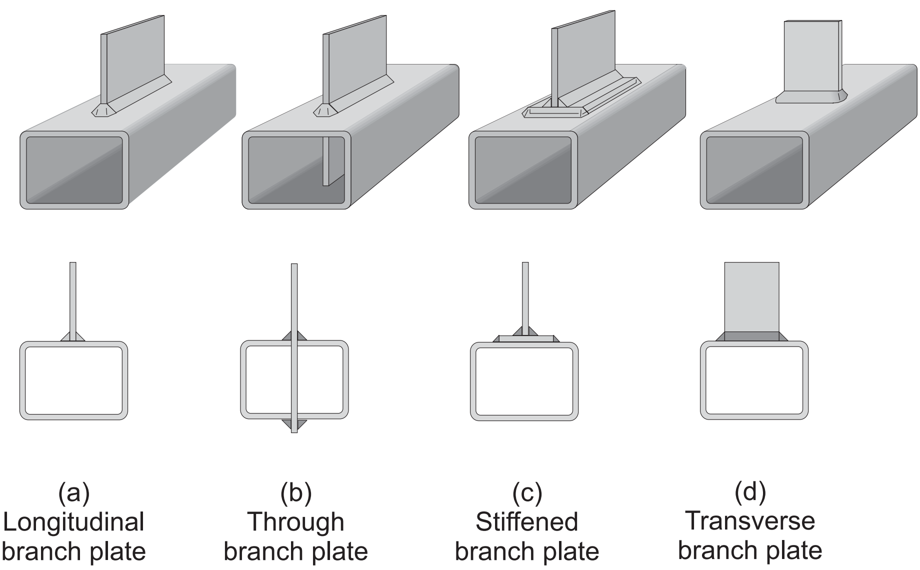



Figure 10.4 – Branch plate-to-RHS member connection types

These connection types are illustrated in figure 10.4 and are considered in subsequent sections in this chapter. Another method for partial stiffening of a longitudinal branch plate, when adjacent to a column baseplate, is shown in figure 10.5. In general though, one should note that in many instances, especially in low-rise and single-storey buildings where wind braces are proportioned often on the basis of maximum allowed slenderness ($L_e/r$) ratios and the brace member forces are low, the unstiffened longitudinal plate connection will be adequate.

Figure 10.5 – Diagonal CHS bracing member connection to a RHS column via a longitudinal branch plate

### 10.1.2 Longitudinal “through-plate”-to-RHS columns

This type of plate connection is illustrated in figure 10.4(b). The longitudinal “through-plate” connection can be expected to have approximately double the strength of a single plate connection, by causing plastification of two RHS column faces rather than one. An experimental and Finite Element study (Kosteski 2001; Kosteski and Packer 2001, 2003a) has in fact confirmed the hypothesis that a through-plate connection can be designed as having two times the strength of the corresponding single plate connection. Thus, the factored resistance of a longitudinal through-plate connection is twice that given by equation 10.1 and the serviceability load limit for the connection is twice that given by equation 10.2.

While the single plate connection is one of the least expensive plate-to-column connections, the through-plate connection is deemed to be the most expensive because of the slotting procedure (Sherman 1996). Designers should also bear in mind that a part of the through-plate protrudes beyond the far side of the RHS (see figure 10.4(b)) and this may affect connections to that face of the RHS column.

### 10.1.3 Stiffened longitudinal plate (T-stub)-to-RHS columns

Considerable research has been undertaken recently on this type of connection (Kosteski and Packer 2000; Kosteski 2001; Kosteski and Packer 2001, 2001a, 2003; Yeomans 2001) which is illustrated in figures 10.4(c) and 10.6.

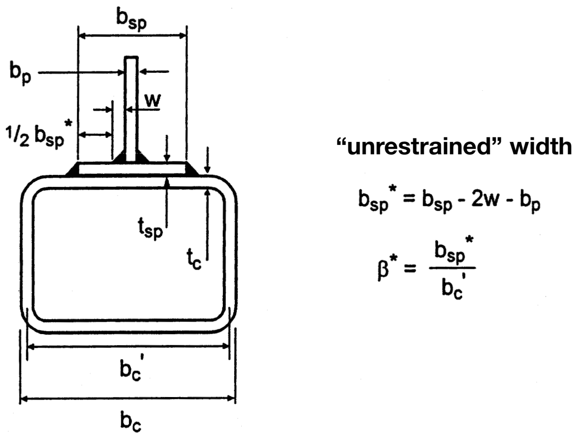



Figure 10.6 – Nomenclature for stiffened longitudinal branch plate connections to RHS

A stiffened longitudinal branch plate connection can ultimately achieve a much higher limit states design resistance, equivalent to the enlarged "footprint" of the stiffening plate, as opposed to the modest footprint of the branch plate itself. In order to achieve the maximum benefit of this enlarged footprint, the stiffening plate must be "effectively-rigid" with respect to the RHS connecting face, such that a plastification mechanism does not occur in the stiffening plate itself. By means of experiments and Finite Element-generated numerical data, an empirical expression has been determined (Kosteski 2001; Kosteski and Packer 2001a, 2003) for the minimum stiffening plate thickness to achieve this effective rigidity. This stiffening plate thickness $t_{sp}$ is given by:

$$t_{sp} \geq 0.5\ t_c \exp(3\beta^*) \quad \text{.......... 10.3}$$

where $t_{sp}$ is the thickness of the stiffening plate lying parallel to the RHS column face. The term $\beta^*$ is the ratio of the "unrestrained" stiffening plate width to the RHS "effective" width, expressed by:

$$\beta^* = b_{sp}^*/b_c' = (b_{sp} - 2w - b_p)/(b_c - t_c) \quad \text{.......... 10.4}$$

Figure 10.6 shows this nomenclature on an illustration of a stiffened longitudinal branch plate connection. Equation 10.3 was developed to be a single consolidated design equation that satisfies both the strength and serviceability limit state conditions. Thus, once the connection has an "effectively rigid" stiffening plate of width $b_{sp}$ (see figure 10.6), the connection resistance can be determined from equation 10.1 by using the footprint dimensions of the stiffening plate.

For the stiffened longitudinal branch plate connection, some other limit states also have to be checked, which are:

(a) Yielding of the longitudinal branch plate, due to uneven loading, given by Kosteski (2001):

$$N_p^* = 0.8\ \phi_1\ f_{p,y}\ h_p\ b_p \quad \text{.......... 10.5}$$

where $\phi_1$ is a resistance factor taken to be 0.9 for a plastification failure mode and 0.8 is an effective width factor.

(b) Column side wall failure, when the width ratio of the "rigid" stiffening plate to the RHS width is approximately unity (i.e. when $b_{sp} \approx b_c$). For such situations, expressions for the RHS side wall resistance (different under tension and compression loading) are given in table 3 of CIDECT Design Guide No.3 (Packer et al. 1992).

(c) Punching shear of the RHS connecting face, when $0.85b_c \leq b_{sp} \leq b_c - 2t_c$. An expression for the punching shear resistance of the RHS connecting face (applicable for either tension or compression branch plate loading) is given in CIDECT Design Guide No.3 (Packer et al. 1992).

### 10.1.4 Transverse plate-to-RHS columns

This type of plate connection is not commonly used to connect bracing members to columns, as the transverse plate would need to be inclined at the angle of the bracing member, as shown in figure 10.7.

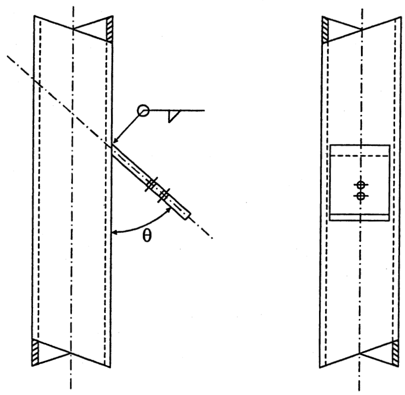



Figure 10.7 – Inclined transverse branch plate welded to the "flat" of a RHS column

This enforces tighter construction and fabrication tolerances than the more lenient longitudinal branch plate connection. The transverse plate connection, particularly if approaching the width of the RHS column member, does, however, provide a stronger and less flexible connection than the longitudinal plate connection. If a transverse plate is used it is recommended that it have a width equal to the "flat" of the RHS column connecting face and that the plate be welded all around. Transverse plate-to-RHS column connections are discussed in detail in chapter 6 dealing with semi-rigid I-beam connections to RHS columns. This is because the bending moment in an I-beam directly welded to a RHS column can be essentially considered as a force couple acting in the beam flanges, thereby reducing the connection design to that of a pair of axially-loaded transverse branch plates. If an inclined transverse branch plate is welded to an RHS column, as in figure 10.7, the connection resistance should be checked by considering just the component of bracing member force perpendicular to the RHS connecting face.

## 10.2 Bracing connections to CHS columns

### 10.2.1 Longitudinal plate-to-CHS columns

Longitudinally-oriented plates welded to CHS columns produce connections with a relatively higher strength and stiffness than their RHS counterparts, as the load applied by the plate is resisted by ring action with a CHS column rather than plate bending with a RHS column. The design recommendations for these connections are principally based on tests carried out in Japan (Kurobane 1981) and are also given in CIDECT Design Guide No. 1 (Wardenier et al. 1991). For connections with longitudinal plates on one or two (opposite) sides, two failure modes should be checked at the factored load level, under either branch plate axial tension loading or axial compression loading, as given below.

1. The factored resistance corresponding to plastification of the CHS member, $N_p^*$, where:

$$N_p^* = 5 f_{c,y}\, t_c^2 \left(1 + 0.25(h_p'/d_c)\right) f(n)/\sin\theta \qquad \text{..........10.6a}$$

The inclusion of the $\sin\theta$ term acknowledges that the axial load in the branch plate is likely inclined at an angle of $\theta$ to the axis of the CHS column (as in figure 10.2), and that the connection behaviour is principally governed by the component of force acting perpendicular to the CHS member axis. In equation 10.6a the term $(h_p'/d_c)$ has an upper limit of 4.0, but this should be an acceptable bound for practical connections. The term f(n) is a function to account for the unfavourable influence of compression loads in the column, where:

$$f(n) = 1 + 0.3n - 0.3n^2, \text{ but } \leq 1.0, \text{ for } n < 0 \text{ (compression)} \qquad \text{..........10.6b}$$

or

$$f(n) = 1.0 \qquad \text{for } n \geq 0 \text{ (tension)} \qquad \text{..........10.6c}$$

In equation 10.6, n is the ratio $\sigma_c/f_{c,y}$ with $\sigma_c$ being the normal stress in the CHS column at the plate joint, due to axial load plus bending (if applicable), on the side of the connection which produces *the higher* f(n) value.

2. The factored resistance corresponding to punching shear of the CHS member, $N_p^*$, where:

$$N_p^* = (2b_p + 2h_p')\, t_c\, (f_{c,y}/\sqrt{3})/\sin\theta \qquad \text{..........10.7}$$

In equations 10.6a and 10.7, when calculating $h_p'$ it is acceptable, and more conservative, to use a value of w = 0.

### 10.2.2 Longitudinal "through-plate"-to-CHS columns

Longitudinal "through-plate" connections to CHS members (similar to figure 10.4(b)) have not been studied by the research community. Equation 10.6a is a simplification of design formulae originally given for T-connections (plate on one side of the CHS) and X-connections (plates on both opposite sides of the CHS), with the simplification being conservative and aimed at covering various load conditions (Wardenier et al. 1991). It would thus seem prudent to also apply equation 10.6a, without modification, to longitudinal through-plate connections for the plastification failure mode, in the absence of more evidence. If the plate is inclined at an angle $\theta$ the connection region along the CHS is considerably lengthened, so the direct application of equation 10.6a should be conservative. For the failure mode of punching shear of the CHS member, the factored resistance should logically be taken as twice that given by equation 10.7.

### 10.2.3 Stiffened longitudinal plate (T-stub)-to-CHS columns

Stiffened longitudinal plate (or T-stub) connections to CHS members are impractical and not likely to be commonly used, as the stiffening (doubler or collar) plate must initially be saddled to fit the curvature of the CHS member. Thus, these connections are not given detailed consideration here although some research information is available on a similar topic (Choo et al. 1998).

### 10.2.4 Transverse plate-to-CHS columns

Transverse plate connections to CHS members are relatively common in triangulated CHS structures because many fabricators try to avoid fully-welded, tube-to-tube, profiled-end connections, and resort to slotting plates into the ends of diagonal CHS members. The general comments made in section 10.1.4 are again applicable here, and the design of transverse plate-to-CHS member connections is discussed in detail in chapter 6. One should again note that if an inclined transverse branch plate is welded to a CHS column similar to that shown in figure 10.7, then the connection resistance should be checked by considering just the component of bracing member force perpendicular to the CHS axis.

## 10.3 Bracing connections to RHS and CHS columns under seismic loading

Bracing members are important lateral load bearing elements in earthquake-resistant design. A common design philosophy for connections under severe seismic loading, in order for the (bracing) member to exhibit sufficient deformation capacity, is that the ultimate strength of the connection be greater than or equal to 1.2 times the axial yield load of the (bracing) member, as noted in section 7.4.

Although not demonstrated by research, some of the connections shown in figure 10.4 may not satisfy a connection overstrength requirement in an energy-dissipative structural frame. The connection resistance formulae $N_p^*$ given in this chapter are approximate “yield-strength” expressions. Connection ultimate strength equations are not available but the Architectural Institute of Japan (AIJ 1990) proposes that the yield strength of a flexible connection be divided by 0.7 to obtain an “ultimate strength”, when information is not available. On this basis, the calculated $N_p^*$ should satisfy:

$$N_p^*/0.7 \geq 1.2A_g f_y \quad \text{.......... 10.8a}$$

i.e. $$N_p^* \geq 0.84(A_g f_y)_{bracing} \quad \text{.......... 10.8b}$$

## 10.4 Truss connections to columns

There are many alternatives for connecting trusses to hollow section columns in simple (“pinned”) construction. Trusses with all types of sections are easily connected to either

RHS or CHS columns, with the prime factors usually being ease of fabrication and ease of erection. The truss-to-column connections are usually configured for site-bolting and normal design considerations for structural steelwork apply. Some typical arrangements for hollow section truss-to-hollow section columns are illustrated in figure 10.8.

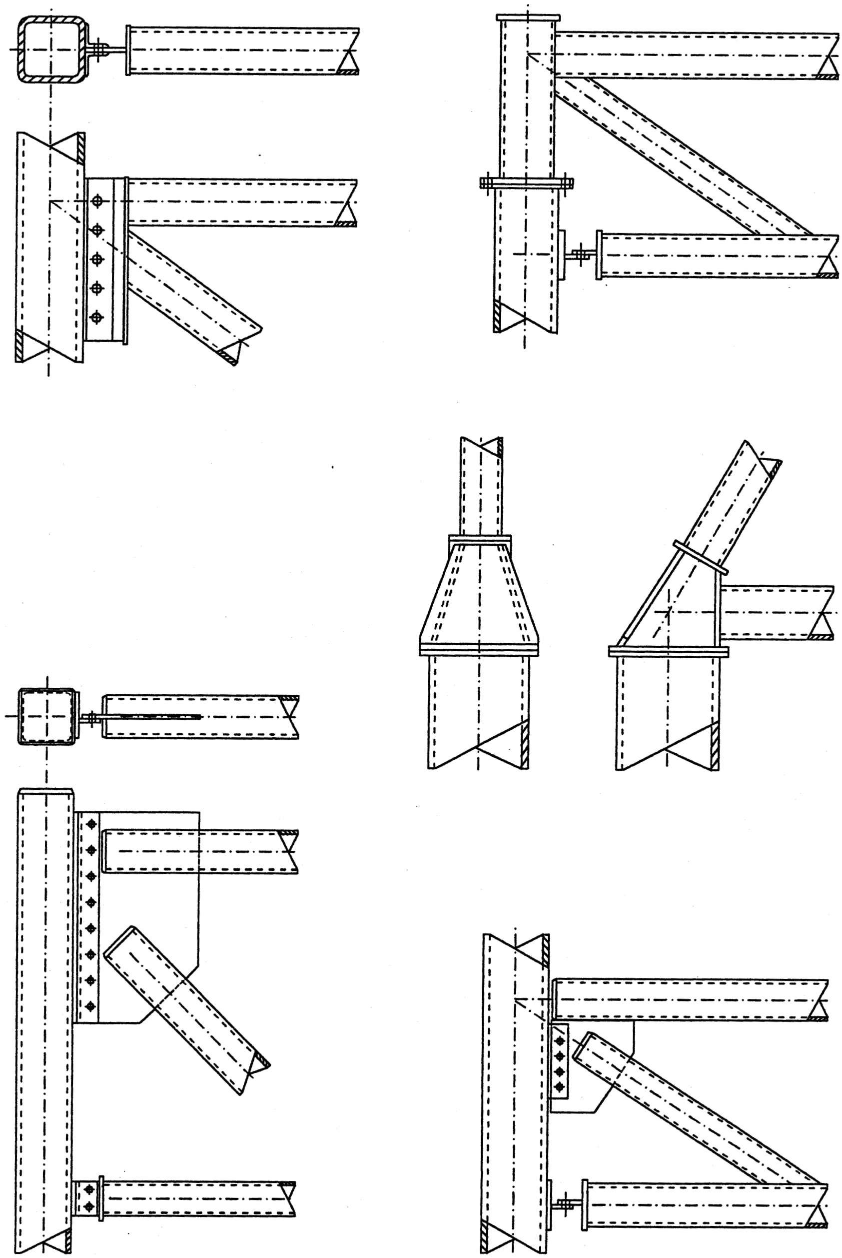


Figure 10.8 – Typical truss to hollow section column connection arrangements

# 11 Column splices

Most structural hollow sections can normally be supplied in lengths of up to 12 to 15 m, and in some cases longer lengths may be available from some manufacturers. Therefore, unless the building is more than 4 to 5 floors high, a single length can generally be used. If more than one length is required, then a splice joint of some kind is needed to join them together.

Unlike most structural sections, any thickness changes, in a nominal size, are made on the inside of the section and the outside dimensions remain the same. This means that, unless the nominal size changes, all the beam lengths remain the same, which can result in savings in fabrication, identification and erection.

Column splices are generally made using either site bolted end plates or site welding. Other methods, such as using bolted side (fish) plates, either externally or internally (see section 11.1.2), shot fired nails (see section 11.3), etc. can be used, but are generally only used in special applications.

Generally columns are compression loaded members with only small moments. As a result the whole of the cross section will be in a compressive stress regime. However, this does not necessarily mean that column splice connections do not have to be designed for tension loads, because generally these joints have to be designed to give some degree of continuity between the two sections being joined together. In moment frames, when the moments are high and tension does occur, and in the case of uplift, the design will usually be determined by the actual applied loads. Different countries have different requirements for the minimum required continuity at the splice and often specify some nominal load or percentage of the column capacity or the sum of the loads applied to the beams immediately below the splice as a minimum requirement. In seismic areas full member capacity may be required, which, in many cases, will require a full penetration, full strength butt (groove) weld.

In most cases these methods can also be used to design a) pinned bases under uplift and b) moment bases.

## 11.1 Plain columns

### 11.1.1 Bolted end plates

This is probably the most commonly used splicing method, because it makes on-site procedures much easier.

Blank end plates, rather than 'ring' end plates, are nearly always used, unless special conditions have to be taken into account, such as if any rebar in a concrete filled column is required to be continuous through the splice joint. The weld joining the hollow section to the end plate will normally be a fillet weld, but on some occasions a butt (groove) weld may be required. Both types of weld are shown in figure 11.1.

#### 11.1.1.1 Circular columns

Circular column splices can be designed in a similar way to bolted tension flanges, described in CIDECT Design Guide No. 1 (Wardenier et al. 1991). Using the symbols shown in figure 11.1 (and incorporating a resistance factor of 0.9):

$$\text{Flange plate thickness, } t_p \geq \sqrt{\frac{2N^+}{0.9\, f_{p,y}\, \pi\, f_3}} \qquad \text{.......................................11.1}$$

$$\text{Number of bolts, } n \geq \frac{N^+}{0.9\, N_b^*}\left(1 - \frac{1}{f_3} + \frac{1}{f_3 \ln(r_1/r_2)}\right) \qquad \text{.......................................11.2}$$

where $f_3 = \frac{1}{2k_1}\left(k_3 + \sqrt{k_3^2 - 4\, k_1}\right)$

$k_1 = \ln(r_2/r_3)$ $\quad$ $k_3 = k_1 + 2$ $\quad$ ln = natural logarithm

$r_1 = d_c/2 + 2e_1$ $\quad$ $r_2 = d_c/2 + e_1$ $\quad$ $r_3 = (d_c - t_c)/2$

$e_1 = e_2$ is another condition for this simplified method

$N_b^*$ = bolt tension resistance, with no allowance for prying forces

$f_{p,y}$ = yield strength of the flange plate material

The term $\left(\frac{1}{f_3 \ln(r_1/r_2)} - \frac{1}{f_3}\right)$ in the inequality 11.2 represents the prying ratio per bolt.

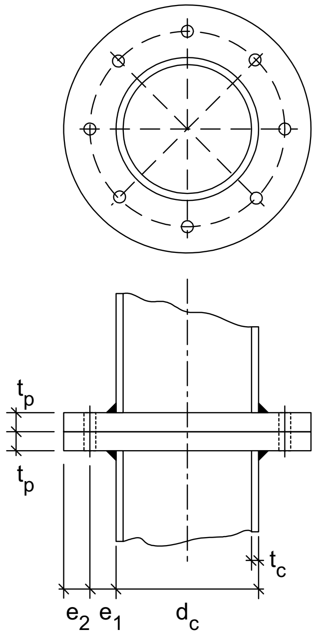



Figure 11.1 – Circular column with site bolted end plates

### 11.1.1.2 Square and rectangular columns

A considerable amount of research has now been undertaken on bolted flange-plate connections between RHS, under axial tension loading, and this can be applied to tension-loaded column splices. Relatively simple methods, for connections bolted on just two sides of the RHS, have been proposed by Packer et al. (1989, 1992) and Packer and Henderson (1997), based on variations of a classical two-dimensional T-stub prying model, dating back to the 1960s (Struik and de Back 1969). However, bolting on just two sides of a column splice is uncommon since the member is likely to sustain some bending moments. Recent (2002) Occupational Safety and Health Administration (OSHA) legislation in the U.S. now in fact requires (for all but small lightly loaded posts) all columns to have baseplates with a minimum of four anchor bolts, and a baseplate designed for a minimum overturning moment about either axis, even for nominally "pinned" bases. This requirement has been introduced to ensure greater column stability, and hence safety, during erection. Extending this same philosophy to bolted flange-plate column splices would lead to further support for bolting on all four sides of such connections.

Research on tension-loaded flange-plate connections for square hollow sections, with bolts on all four sides, has been undertaken by Mang (1980), Kato and Mukai (1982) and Caravaggio (1988), but none of these resulted in practical design procedures. A more recent study by Willibald et al. (2002) has proposed a variation on a design method originally postulated in the AISC HSS Connections Manual (1997), which is itself an extrapolation of the two-dimensional T-stub model. Willibald et al. (2003) have then showed that this design procedure (the modified AISC model) is also the preferred approach for the design of rectangular (in addition to square) hollow section flange-plate connections bolted on all four sides. This latter publication (Willibald et al. 2003) recommends the following design procedure.

In a typical design procedure the factored load as well as the size of the RHS are known (and also the outer dimensions of the flange-plate may be chosen) before designing the connection. The unknowns of the connection are hence the number of bolts, their diameter and grade and locations on the plate as well as the flange-plate thickness. As a first limit states design step, the load per bolt can be calculated assuming that no prying action is taking place in the connection. Bolts of the right size and grade have to be chosen so that:

$$N_b^* \geq N^+/n \qquad \text{.......... 11.3}$$

The trial selection now has to be checked and the required flange-plate thickness has to be found. The design method reverts back to the original T-stub model by Struik and de Back (1969), with the inner yield line forming adjacent to the RHS outer face and the outer yield line following the bolt line. From test results it is deduced that the bolt forces act somewhere between the bolt axis and the edge of the bolt head, resulting in a slight shift of the outer yield line. Thus the effective dimensional parameters a′ and b′ are calculated as follows (see figure 11.2):

$$a' = a + d_b/2 \qquad \text{.......... 11.4}$$

with the dimension a limited to a maximum of 1.25b in the calculations (but not necessarily for the plate dimensions), and

$$b' = b - d_b/2 \qquad \text{.......11.5}$$

The coefficient $\rho$ can be determined:

$$\rho = b'/a' \qquad \text{.......11.6}$$

A "temporary prying factor" "$\beta'$" is then calculated by:

$$\beta' = (1/\rho)\left(N_b^* / (N^+ / n) - 1\right) \qquad \text{.......11.7}$$

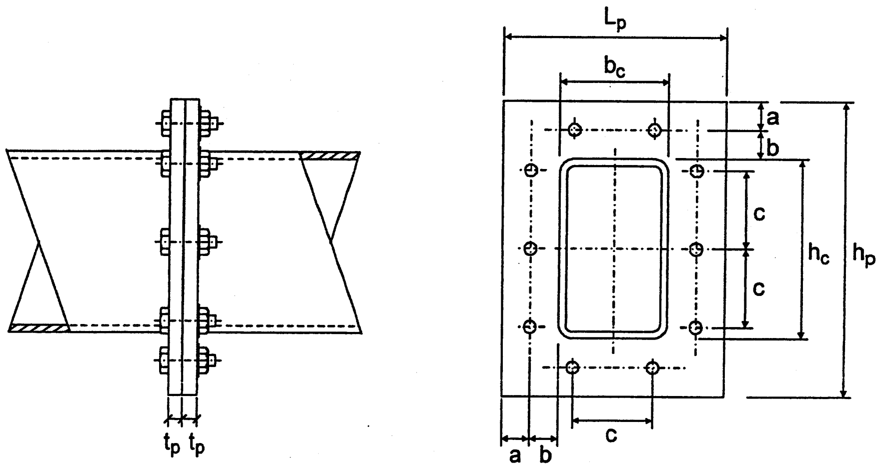



Figure 11.2 – Layout of site-bolted RHS column flange-plate connection

Another coefficient, $\delta$, which includes the influence of the bolt pitch p (see figure 11.3) is calculated using:

$$\delta = 1 - d_h/p \qquad \text{.......11.8}$$

where $d_h$ is the bolt hole diameter.

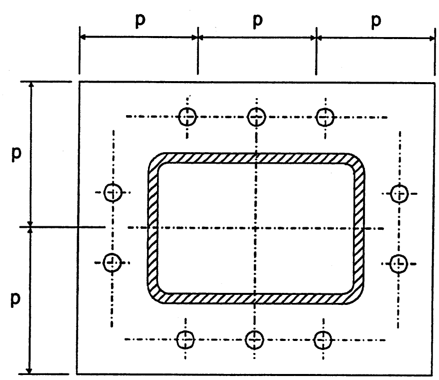



Figure 11.3 – Definition of bolt pitch p for RHS flange-plate connections

The bolt pitch p is calculated as the flange height/width divided by the number of bolts parallel to the flange height/width on that one side. If the plate is not square, p should be chosen as the minimum value of the bolt pitch for the long and the short side (assuming equal values of a and b, as shown in figure 11.3, on both the long and short sides of the plate).

Using the results of equations 11.7 and 11.8 the coefficient $\alpha'$ is calculated:

If $\beta' \geq 1.0$ then $\alpha' = 1.0$

$$\text{If } \beta' < 1.0 \text{ then } \alpha' = (1/\delta)\left(\beta'/(1-\beta')\right) \qquad \text{.....11.9}$$

but $0 \leq \alpha' \leq = 1.0$

Finally, by comparing the plastic moment per unit flange-plate width ($t_p^2\, f_{p,y}/4$) with the bolt strength and the level of prying $\alpha'$, the required flange-plate thickness can be calculated as follows:

$$t_{required} = \sqrt{\frac{4(N^{+}/n)b'}{\phi_1 p\, f_{p,y}\,(1+\delta\alpha')}} \qquad \text{.....11.10}$$

A resistance factor of $\phi_1 = 0.9$ is recommended.

### 11.1.2 Bolted side plates

This method is only suitable for square and rectangular hollow section columns and typical details are shown in figure 11.4. The plates can be on two or four sides, depending upon the loads that need to be carried, and either on the inside or outside of the hollow section.

A gap, as shown in figure 11.4, or no gap can be left between the ends of the hollow section column. However, in the latter case fabrication and erection is much more difficult, because, in effect, there needs to be an almost zero tolerance on the column lengths and the bolt holes to ensure that full bearing takes place. In most cases, therefore, the plates and bolts should be designed to take the full column load.

Depending upon the column thickness for external plates, and the plate thickness for internal plates, the inner thickness can be either conventionally drilled and threaded or, if it is too thin for this, a single sided bolting system (see chapter 3) can be used.

As an alternative to the arrangement shown in figure 11.4, the plates could be welded to the end of one column section and bolted to the other.

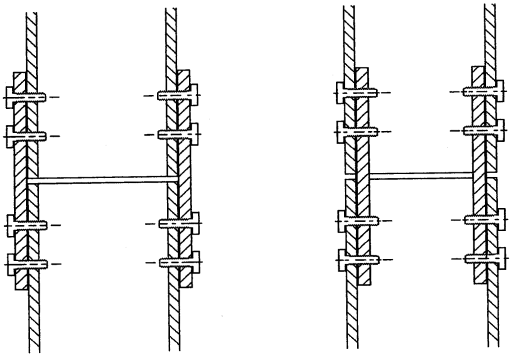




Figure 11.4 – Bolted side plates (left external, right internal)

### 11.1.3 Welding

This is suitable for all hollow sections and typical details are shown in figure 11.5. The welds can be full or partial penetration butt (groove) welds and should be designed to carry

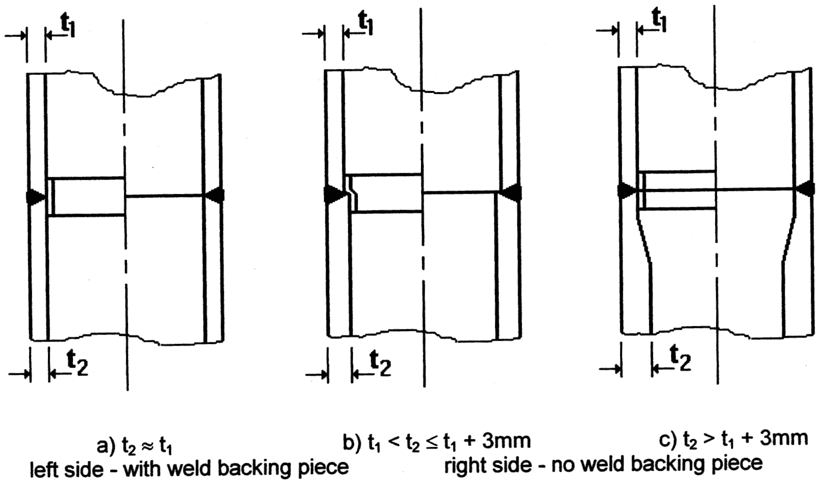



Figure 11.5 – Plane column splices – site welding with a weld backing piece

the required loads specified by the design code or specification being used. Weld backing pieces are normally required for full depth welds.

The welds should be made to their full size around the whole periphery of the hollow section.

### 11.1.4 Welded column splices in seismic areas

This section describes design requirements and practices for column splices to be used for moment resisting and braced frames in seismic areas. Column splices are important elements to maintain integrity of structures because the splices are, like columns themselves, critical gravity load bearing elements and therefore should be designed to allow a significant safety margin.

The AISC Seismic Provisions (1997a, 2000) specify that column splices in special moment frames should not be located within 1200 mm of the beam-to-column connections. If the column clear height is less than 2400 mm, the splice can be located at the mid-height of the column. These requirements are intended not only to reduce flexural demand on the splice but also to simplify field erection and construction due to increased accessibility. Additionally, the AISC provisions require to use CJP groove welded joints for the column splices of special moment frames. Then, the column splices have strength closely comparable to the columns and no further strength calculation is required.

The AIJ Design Recommendations (2001) allow the use of PJP groove welded joints for the column splices in moment resisting and braced frames, provided that stresses in the column splices are kept within an elastic regime. Note that welded joints should be continuous throughout the periphery of the column as shown in figure 11.6.

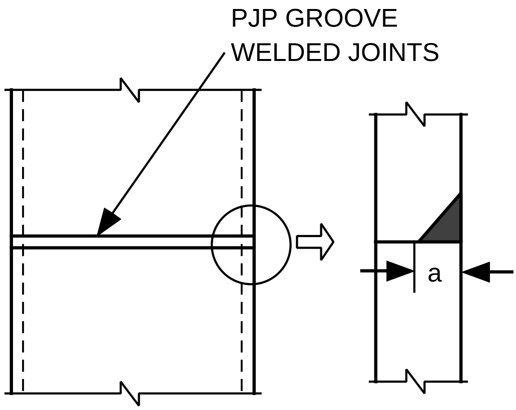



Figure 11.6 – Column splice with PJP welded joint

The design axial, flexural and shear loads at the splice are those at ultimate limit state determined by the plastic global analysis, which are denoted as $N_{c,s}$, $M_{c,s}$ and $V_{c,s}$. For RHS columns the following design formulae are provided:

$$\left(\frac{\alpha M_{c,s}}{M_{weld,y}} + \frac{N_{c,s}}{N_{weld,y}}\right)^2 + \left(\frac{V_{c,s}}{V_{weld,y}}\right)^2 \leq 1.0 \qquad \text{.......11.11}$$

in which the factored design flexural load $\alpha M_{c,s}$ should not be less than half of the nominal yield moment of the column. Namely,

$$\alpha\ M_{c,s} \geq 0.5 W_{el} f_{c,y} \qquad \text{.......11.12}$$

In the above equations,

| | |
|---|---|
| $f_{c,y}$ | design yield strength of weaker of column materials joined with matching filler metal |
| $M_{weld,y} = W_{el,weld} f_{c,y}$ | yield moment of PJP groove welded joints |
| $N_{weld,y} = A_{e,weld} f_{c,y}$ | axial yield force of PJP groove welded joints |
| $V_{weld,y} = A_{e,w,weld} f_{c,y}/\sqrt{3}$ | shear yield force of PJP groove welded joints |
| $W_{el}$ | elastic section modulus of weaker of columns joined |
| $\alpha = 1.2$ | overstrength factor |

where

| | |
|---|---|
| $A_{e,weld}$ | effective cross-sectional area of PJP groove welded joints |
| $A_{e,w,weld}$ | effective cross-sectional area of PJP groove welded joints to column web |
| $W_{el,weld}$ | elastic section modulus of effective cross section of PJP groove welded joints |

No design formulae for PJP welded CHS column splices are provided in the AIJ Recommendations.

Welding should be performed to obtain the effective throat thickness "a" so that all the loads $N_{c,s}$, $\alpha M_{c,s}$ and $V_{c,s}$ are transmitted by the welds. It is not recommended to take into account contact stresses, because end preparation for full contact for bearing calls for an unrealistically high accuracy. It is to be noted that in equation 11.11 the resistance of PJP groove welds per unit area is taken to be equal to that of CJP groove welds, which results in a 13 to 41 per cent increase in the design strength of PJP welds over the design strength specified in Eurocode 3. This increased resistance is based on a statistical survey of the past extensive test results on PJP welded joints as well as on PJP groove welded column splices (AIJ 2001).

The AIJ Recommendations show design formulae for bolted column splices as well. These formulae are for high-strength bolts in slip-resistant connections and applicable only to open section columns. Although the details are not shown herein, the design procedure for bolted connections is identical to that described for PJP welded joints. The splice is designed not to exceed the yield load of plate elements and the slip resistance of bolted joints under the loads $N_{c,s}$, $\alpha M_{c,s}$ and $V_{c,s}$.

The AISC seismic provisions also allow the use of PJP groove welded column splices for frames excluding special moment frames, if the splices are located at the mid-height regions of columns as indicated earlier. However, the AISC imposes much more stringent requirements than the AIJ. For example, the AISC stipulates that the PJP groove welded column splices have a strength of 200 % of the required strength.

The above requirements for column splices with PJP groove welded joints are those intended for common frame configurations. Extra strength and detailing may be required for conditions such as: columns in tall stories, large changes in column sizes at the splice, or where there is the possibility of a singe curvature moment gradient in the column (SEAOC 1996). Column splices with CJP welded joints should be adopted in such cases and also for CHS columns.

## 11.2 Concrete filled columns

Information on concrete filling methods and the design of concrete filled hollow section columns is given in CIDECT Design Guide No. 5 (Bergmann et al. 1995). Any welding of connections for primary members should be carried out before concrete filling, but those with low heat input, such as for secondary members, can be made after filling.

The splice can be either welded or bolted. If continuity of the concrete and any rebar through the splice is required, then filling should generally be done on site. In this case, the concrete should be stopped below the splice, about 150 mm below for a welded splice, and the rebar continued through the splice (see figure 11.7). After the next section of column has been erected and fixed in place and any rebar has been installed, the concrete filling can continue.

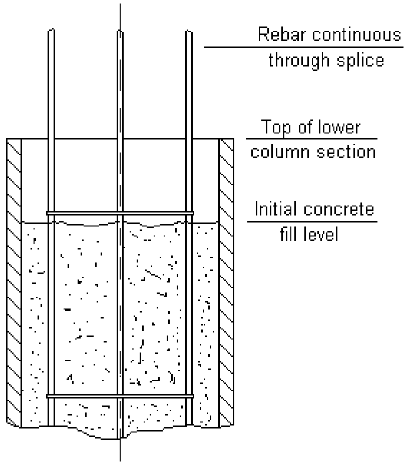



Figure 11.7 – Concrete filled column splice, with continuity of concrete and rebar

### 11.3 Nailing of poles

Although not yet used in a building environment, the use of shot fired nails (or powder actuated fasteners) is possible in some specific applications, such as electricity transmission or distribution line poles. This method is only suitable for applications where circular hollow sections are used, but it may be an economical alternative to welding or bolting.

Some section views of nailed connections are shown in figure 11.8. The left side drawing shows a mating tube-in-tube splice connection and the right side a sleeved connection. The sleeved connection enables two tubes of the same outside diameter to be joined and the sleeve could also be shop-welded to one of the pole sections, thereby aiding erection and reducing on-site nailing.

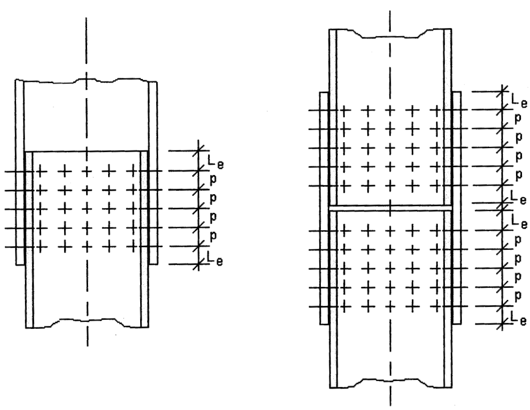


Figure 11.8 – Splice connection using shot fired nails

Under static loading, the critical failure modes for these connections are: nail shear, bearing of the (tube) base metal and net section fracture of the (tube) base metal. The latter failure mode is only applicable under tension loading on a connection, but nail shear and bearing failure are applicable under both tension and compression loadings. The net load on a nailed connection, or part of the circumference of a nailed connection, in a pole splice can be determined from the applicable load combinations of axial load or axial load plus bending. The above three failure modes have been verified for non-stainless as well as stainless steel nails, and even in both statically-loaded and fatigue-loaded connections (Packer 1996, Kosteski et al. 2000, Lecce and Packer 2003). Further description of this nailing technology can be found in CIDECT Design Guides No. 6 (Wardenier et al. 1995) and No. 7 (Dutta et al. 1998).

Unlike standard bolts, there are no published national or international standards regarding the dimensions or strengths of nails (or powder actuated fasteners). Hence the geometric properties and guaranteed minimum mechanical strengths need to be obtained from the manufacturer. The following connection resistance expressions (for a group of n nails) are currently advocated (Lecce and Packer 2003) for the three potential failure modes:

nail shear: $$V_n^* = \phi_3 \; n \; A_n \; f_{n,u} / \sqrt{3} \qquad \text{.......................... 11.13}$$

tube bearing:
(either tube) $$B_c^* = \phi_3 \; d_n t_c \; n \; f_{c,u} \qquad \text{...........................11.14}$$

tube net section fracture:
(either tube) $$N_c^* = \phi_2 \; ( A_c - d_n \; n_r \; t_c ) f_{c,u} \qquad \text{...........................11.15}$$

In the foregoing, resistance factors of $\phi_2 = 0.75$ and $\phi_3 = 0.67$ are recommended, n is the total number of nails under consideration, $n_r$ the number of nails per row, $d_n$ the nail shank diameter, $A_c$ the gross cross-sectional area of the tube (column) under consideration, $A_n$ the nail cross-sectional area, $t_c$ the thickness of the tube under consideration, $f_{c,u}$ the ultimate tensile strength of the tube (column) material, and $f_{n,u}$ the ultimate tensile strength of the nail material.

## 11.4 Design example

### 11.4.1 Bolted end plates

The following example is based on a column carrying a compressive axial load and a bending moment. The design methods for bolted flange splices are generally based on axial loading only, so a hypothetical/effective axial load, based on the actual factored applied compression axial ($N_c$) and moment ($M_c$) loads, must be used, e.g.

$$\text{effective axial load} = \left(-N_c/A_c \pm M_c/(W_{el,c} \text{ or } W_{pl,c})\right) A_c \qquad \text{............................11.16}$$

If the effective axial load is tensile, then the splice must be designed for the maximum of the effective axial tensile load and the minimum nominal tensile load specified in the design specification being used (see introduction to chapter 11). If the effective axial load is compressive, then the splice must be designed for the minimum nominal tensile load required by the design specification being used (see introduction to chapter 11).

**11.4.1.1 Bolted end plates and circular columns**

Assumptions:

a) the column is wholly in compression,
b) the column is 406.4 x 12.5 with a yield strength of 355 $N/mm^2$,
c) the plate will have a yield strength of 275 $N/mm^2$,
d) the required minimum nominal tensile load ($N_c$ or $N^+$) required by the design specification is about 20% of the column capacity, say 1100 kN.

Determine: a) the plate thickness, $t_p$, and b) the number of bolts, n.

1) Size of bolts will be M24 or M20 grade 8.8.

*Bolt tension capacity from EC3,*

$$N_b^* = 0.9\, f_{b,u}\, A_b/\gamma_{Mb}$$
$$= 0.9\,(0.800)(353)/1.25 = 203 \text{ kN} \quad \text{for M24}$$
$$= 0.9\,(0.800)(245)/1.25 = 141 \text{ kN} \quad \text{for M20}$$

2) *Layout of end plates* (figure 11.1) $e_1 = e_2 = 35\text{mm}$

3) Determine connection parameters

$r_1 = d_c/2 + 2e_1 = 406.4/2 + 2(35) = 273.2$
$r_2 = d_c/2 + e_1 = 406.4/2 + 35 = 238.2$
$r_3 = (d_c - t_c)/2 = (406.4 - 12.5)/2 = 196.95$
$k_1 = \ln(r_2 - r_3) = \ln(238.2/196.95) = 0.190$
$k_3 = k_1 + 2 = 0.190 + 2 = 2.19$

$$f_3 = \frac{1}{2k_1}\left(k_3 + \sqrt{k_3^2 - 4\,k_1}\right) = \frac{1}{2(0.190)}\left(2.19 + \sqrt{2.19^2 - 4(0.19)}\right) = 11.05$$

Plate thickness, $t_p \geq \sqrt{\dfrac{2N^+}{0.9\, f_{p,y}\, \pi\, f_3}} = \sqrt{\dfrac{2(1100)}{0.9(0.275)\pi(11.05)}} = 16.0$

$\therefore$ Use 16 mm plate

Number of bolts, $n \geq \dfrac{N^+}{0.9N_b^*}\left(1 - \dfrac{1}{f_3} + \dfrac{1}{f_3 \ln(r_1/r_2)}\right)$

$$\geq \frac{1100}{0.9(203)}\left(1 - \frac{1}{11.05} + \frac{1}{11.05 \ln(273.2/238.2)}\right) = 9.5 \text{ for M24}$$

$$\geq \frac{1100}{0.9(141)}\left(1 - \frac{1}{11.05} + \frac{1}{11.05 \ln(273.2/238.2)}\right) = 13.6 \text{ for M20}$$

$\therefore$ Use 10 bolts for M24 bolts
or Use 14 bolts for M20 bolts

# 12 List of Symbols and Abbreviations

## 12.1 Abbreviations of organisations

| | |
|---|---|
| AIJ | Architectural Institute of Japan |
| AISC | American Institute of Steel Construction |
| ASTM | American Society for Testing and Materials |
| AWS | American Welding Society |
| CEN | Commission European de Normalisation (Standardisation) |
| CISC | Canadian Institute of Steel Construction |
| CSA | Canadian Standards Association |
| FEMA | Federal Emergency Management Agency (USA) |
| IIW | International Institute of Welding |
| ISO | International Standards Organisation |
| JIS | Japanese Industrial Standard |
| SAC | A partnership between SEAOC, ATC and CUREE |
| SEAOC | Structural Engineers Association of California |

## 12.2 Other abbreviations

| | |
|---|---|
| CHS | circular hollow section |
| RBS | reduced beam section |
| RHS | rectangular or square hollow section |
| CJP | complete joint penetration (weld) |
| CVN | Charpy V-notch |
| HAZ | heat affected zone |
| PJP | partial joint penetration (weld) |

## 12.3 General symbols

| | |
|---|---|
| A | area |
| B* | bearing resistance |
| B | bearing load |
| C | connection rotational stiffness |
| E | modulus of elasticity |
| G | shear modulus of elasticity |
| I | second moment of area |
| J | load ratio between out-of-plane and in-plane beams |
| K | axial stiffness, beam stiffness |
| L | length |
| M* | moment or flexural resistance |
| M | applied moment acting on connection |
| N* | axial load resistance |
| N | axial force acting on connection |
| P | axial force acting on an element of the connection |
| R | resistance in general, reduction factor or rotation capacity |
| S | stiffness generally |
| V* | shear resistance |
| V | shear force |
| W | section modulus |
| | |
| a | weld throat thickness or general dimension |
| b | width dimension |

| | |
|---|---|
| c | general dimension or constant |
| d | diameter |
| e | eccentricity |
| $e_{1,2}$ | distance from tube to bolt centre, distance from bolt centre to edge of plate |
| f | material nominal strength, e.g. $f_u$ = material nominal tensile strength |
| g | bolt spacing across column face |
| h | height dimension or width of column face not connected to beam |
| k | model spring stiffness |
| n | number of fasteners or rows of fasteners or column stress divided by the column yield stress |
| n' | column pre-stress ratio = column "prestress" divided by the column yield stress |
| q | seismic behaviour factor or uniformly distributed load |
| p | bolt pitch or flange plate width per bolt |
| r | radius, radius of gyration |
| s | distance |
| t | thickness |
| w | fillet weld leg length |
| x | distance from neutral axis to inside of compression flange |

| | |
|---|---|
| $\alpha$ | stress reduction factor or angle or over-strength factor |
| $\beta$ | connection width to column width ratio, or a constant |
| $\beta'$ | connection effective width to column width ratio, or a constant |
| $\gamma$ | shear strain or column half width (diameter) to thickness ratio |
| $\gamma_M$ | partial safety factor for capacity $\approx 1/\phi$ |
| $\delta$ | displacement or storey drift |
| $\epsilon$ | strain |
| $\phi$ | resistance (or capacity) factor $\approx 1/\gamma_M$ or rotation |
| $\eta$ | connection (beam) depth to column width ratio or cumulative plastic deformation factor |
| $\theta$ | angle |
| $\sigma$ | actual stress or strength, e.g. $\sigma_y$ = actual yield stress |
| $\tau$ | beam to column thickness ratio |
| $\Delta L$ | elongation |

## 12.4 Subscripts

| | |
|---|---|
| b | beam or bolt or bearing |
| bs | beam splice |
| c | column or concrete, e.g. $A_{c,c}$ = concrete area in concrete filled column |
| cf | column face |
| d | diaphragm |
| e | effective or estimate |
| el | elastic |
| f | flange, e.g. $t_{b,f}$ = beam flange thickness, or face |
| g | gross, e.g. $A_g$ = gross area |
| h | hole or horizontal |
| ip | in-plane |
| j | joint or connection, e.g. $N_j^*$= joint axial load resistance |
| m | mean |
| n | nail or net, e.g. $A_n$ = net area |
| op | out-of-plane |
| p | plate, panel or punching shear |
| pl | plastic |

s splice or distance
sp stiffening plate
u ultimate
un unfactored (service or specified) load
v shear or vertical distance
w web, e.g. $t_{b,w}$ = beam web thickness
y yield

**12.5 Superscripts**

+ positive or tensile, e.g. $N^+$ = tensile force
- negative or compressive, e.g. $N^-$ = compressive force
* capacity or resistance

Symbols not shown here are specifically described at the location where they are used.

In all calculations the nominal mechanical and geometric properties should be used.

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# Annex A: Investigation into through diaphragms

An appraisal of test results obtained from a large-scale collaborative investigation on beam-to-RHS column connections with through diaphragms.

### A.1 Summary of tests

A large-scale investigation into the behaviour of beam-column connections was performed after the Kobe earthquake as a collaboration of 7 universities (AIJ Kinki 1997). Connections selected for the research were those between RHS columns and I-section beams, with the through diaphragms at the positions of beam flanges and with conventional and improved details. In total 86 specimens were tested.

All the specimens were configured to form a T-shaped assembly with a single I-section beam connected to one side of an RHS column. The details of connections can be subdivided into two large groups depending on the design applicable to either shop-welding or field-welding. The beam flanges were groove welded to the through diaphragms, while the beam webs were either welded to the column faces or bolted to the columns via shear tabs. Figure A.1 illustrates these two representative connection details.

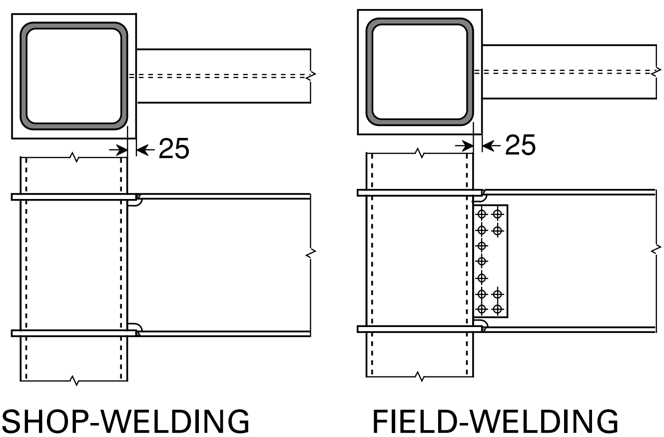


Fig A.1 – Examples of conventional connection designs

Three different grades of steel were used for beams, of which the steel with the designation SS is ordinary low carbon steel for structural use, while the steel with the designation SN is new steel for which both the upper and lower limits of yield stress are specified in the Japanese Industrial Standard. Dimensions and mechanical properties of beam materials are summarised in table A.1.

| Dimensions of I-sections | Steel grade | Specimens extracted from | $\sigma_y$ (MPa) | $\sigma_u$ (MPa) | elong (%) | $_vE_0$ (J) | $_vT_E$ (°C) |
|---|---|---|---|---|---|---|---|
| W600x200x11x17 | SN400B | flange at 1/4 of width | 290 | 446 | 28 | 115 | 4 |
| | | flange at intersection with web | 252 | 436 | 31 | 73 | 27 |
| | | web | 342 | 465 | 29 | 155 | - 44 |
| W500x200x10x16 | SS400 | flange at 1/4 of width | 293 | 448 | 30 | 117 | 8 |
| | | flange at intersection with web | 279 | 446 | 31 | 39 | 16 |
| | | web | 313 | 446 | 31 | - | - |
| W600x250x12x25 | SN490 | flange at 1/4 of width | 371 | 517 | 32 | 197 | - 42 |
| | | flange at intersection with web | 344 | 513 | 28 | 165 | - 30 |
| | | web | 425 | 553 | 23 | - | - |

Note:

Dimensions of I-section are shown as h x $b_f$ x $t_w$ x $t_f$.

elong = elongation; $\sigma_y$ = yield stress; $\sigma_u$ = ultimate tensile strength; $_vE_0$ = absorbed energy at 0 °C; $_vT_E$ = energy transition temperature(°C)

Table A.1 – Dimensions and mechanical properties of beam materials

One of the important variables in connection details was the type of beam copes. The conventional beam cope and improved copes of types A and B are shown in figure 8.2. The conventional cope was manufactured by a cutter with a radius of 35 mm. However, a return with a radius of less than 10 mm was also prepared at the toe where the cope hole contacted the beam flange. The improved cope for field-welded connections was similar in shape to the improved type B cope for shop-welded connections as shown in figure A.2. However, an additional improvement was made by extending a portion of the diaphragm towards the beam flange.

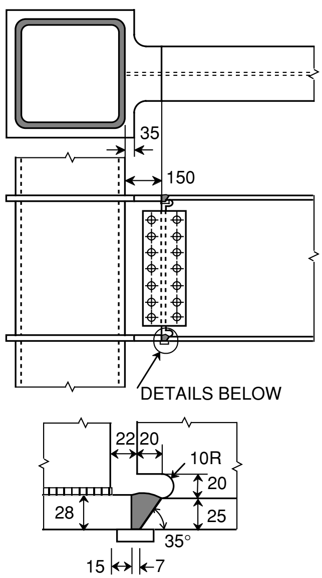


Figure A.2 – Improved connection for field-welding applications

Backing bars were fillet-welded to the beam flanges at points around a quarter of the beam width in accordance with the AIJ Recommendations (AIJ 1995) in all the conventional and improved specimens. This was because past test results as well as damage to connections due to the Kobe earthquake indicated that an existence of fillet welds right in front of the beam cope was harmful and frequently induced brittle fractures starting from the toes of the copes.

Weld tabs used were of two kinds, steel tabs and flux tabs. Although steel tabs are commonly used everywhere, flux tabs are special to Japanese fabricators. Flux tabs are a kind of weld dams made of ceramics but are called flux tabs. When steel tabs were used at the corners of the through diaphragms and beam flanges and left as they were after welding, cracks frequently started at notch roots formed by the unfused regions between the weld tabs and beam flanges. These cracks deteriorated significantly the rotation capacity of beams. Two different welding procedures were specified for fabricating specimens, which were called multi-passes per each layer and single pass per each layer. Cross sections of weld beads are compared between these two in figure A.3. Herein, the former welding procedure is called stringer passes, while the latter one is called weave passes. The weave passes provide higher weld metal deposition rate and greater heat input, which promotes grain growth in the HAZ and attendant low notch toughness.

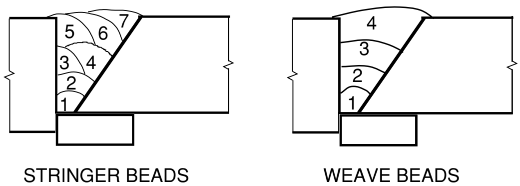


Figure A.3 – Cross section of weld beads

Of 86 specimens, 8 specimens were tested in dynamic loading with the rate varying between 1 Hz and 0.6 Hz while 12 specimens were tested at a temperature of -23 °C. The remainders of the specimens were subjected to slowly applied cyclic loading at room temperatures. All the specimens were tested in principle under the loading sequences predetermined as follows: at least 2 cycles of reversed loading in an elastic region and, subsequently, displacement controlled cyclic loading with the amplitude increased as $2\theta_{pl}$, $4\theta_{pl}$, $6\theta_{pl}$, ….. up to failure, where $\theta_{pl}$ signifies the elastic beam rotation at the full plastic moment $M_{pl}$ (see sect. A.4). Two cycles of loading were applied at each displacement increment.

Of 86 specimens, 70 specimens failed by brittle fracture or ductile tensile tear while the remaining 16 specimens reached the maximum loads owing to local buckling of plate elements at the beam ends. The tensile failure modes can be subdivided into 2 large groups: those that fractured due to cracks starting at the toes of beam copes and the others that fractured from weld metal or heat-affected zones (HAZ) of CJP groove welds at the beam flange ends. Most fractures belonging to the latter failure mode started at the terminations (starting and stopping ends) of groove welds between the beam flanges and through diaphragms.

## A.2 Evaluation of rotation capacity of beams

Among several factors that would deteriorate the performance of connections, these research results revealed that the following 4 factors significantly participated in reducing the rotation capacity of beam-column assemblies. These factors include the conventional beam cope, steel weld tab, weave beads, and unskilled welding operation. One of the other important factors is the material factor, whose effects were unable to be evaluated from these research results. However, it should be noted that the toughness properties of both base and weld materials used in these tests were higher than the specified minimum value of 27 J at 0 °C, which were referred to in section 7.2, according to Charpy V impact test results.

The reduction in the cumulative plastic deformation factor $\eta$ is denoted by R with a subscript showing the cause for the reduction. Each value of R is calculated as the difference in $\eta$ between the two opposite cases, namely the cases with conventional and improved details. The following are the evaluated R values.

If failure was governed by a fracture from the toe of the beam cope,

$$R_{CONVENTIONAL\ COPE} = 56.7 - 40.1 \cong 17 \quad \text{A.1}$$

If failure was governed by a fracture from the welded joints, the following 3 different reductions were found significant.

$$R_{STEEL\ TAB} = 59.5 - 37.8 \cong 22 \quad \text{A.2}$$

The above reduction is due to the difference between flux and steel tabs but is not applicable to the field welded connections with the improved beam copes used in these tests. This is because the improved field-welded connections had straight edges at the terminations of groove welds.

$$R_{WEAVE\ BEADS} = 52.8 - 39.2 \cong 14 \quad \text{A.3}$$

$$R_{UNSKILLED\ AT\ FLUX\ TAB} = 59.5 - 43.2 \cong 16 \quad \text{A.4}$$

The last reduction (equation A.4) is applicable only to field-welded connections so far as the present test results are concerned. This is because the welders who specialise in field-welding were found to be unaccustomed to using flux tabs.

Of 86 specimens, 24 specimens had improved details in the profile of beam copes and in weld tabs, and were welded using stringer beads by skilled welders. These specimens require no reduction in $\eta$. These specimens showed an average $\eta$ value of 67.3. Thus, the estimate of the cumulative plastic deformation factor $\eta_e$ can be given by the smaller of equations A.5, A.6 and A.7.

If cracks start at the toes of the beam copes,

$$\eta_e = 67 - R_{CONVENTIONAL\ COPE} \quad \text{A.5}$$

If cracks start at the CJP groove welds at the beam ends,

$$\eta_e = 67 - R_{WEAVE\ BEADS} - R_{STEEL\ TAB} \quad \text{A.6}$$

If cracks start at the CJP groove welds at the beam ends and for field-welded connections,

$$\eta_e = 67 - R_{WEAVE\ BEADS} - R_{UNSKILLED\ AT\ FLUX\ TAB} \quad \text{A.7}$$

For connections with improved details, the R values corresponding to the improvement are taken to be null in the above 3 equations.

The cumulative plastic deformation factors are calculated for all the specimens included in the large-scale investigation. The ratios of observed $\eta$ to predicted $\eta$ distribute as shown in figure A.4.

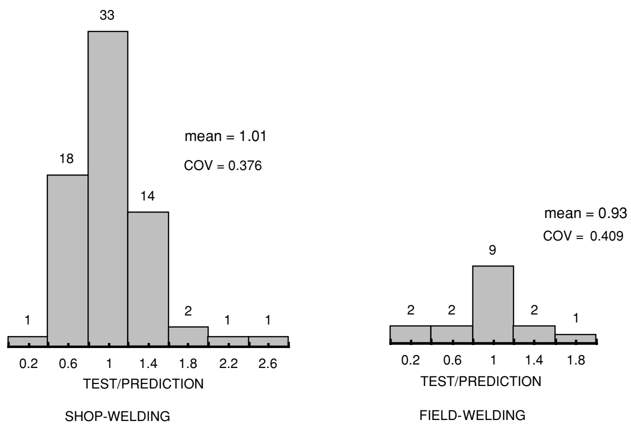


Figure A.4 – Histograms showing distributions of test to predicted ratios

The above equations slightly overestimate the deformation capacity of field-welded connections. Thus, the mean $\eta_m$ and standard deviation $\sigma_h$ of the cumulative plastic deformation factor are given as:

for shop-welded connections

$$\eta_m = 1.01\ \eta_e \text{ and } \sigma_\eta = 15.9 \quad \text{A.8}$$

and for field-welded connections

$$\eta_m = 0.93\ \eta_e \text{ and } \sigma_\eta = 22.3 \quad \text{A.9}$$

## A.3 Flexural strength of beam-column connections

No definite correlation was found in the test results between the maximum moment carried by the beams and the connection details. The maximum moments were greater than the fully plastic moments of the beams. As described in section 8.1, the overstrength factors are plotted against the cumulative plastic deformation factors in figure A.5.

Although scatter is large, the overstrength factor increases linearly with $\eta$ and can be represented by the following equation.

$$\frac{M_{cf,max}}{M_{pl}} = 0.0025\eta + 1.18 \quad \text{A.10}$$

The data for dynamically loaded specimens are omitted for evaluating the above regression equation, because material properties under dynamic loads were not reported.

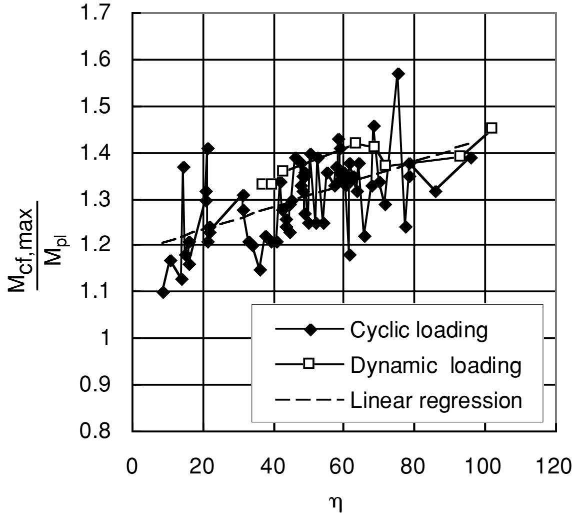


Figure A.5 – Overstrength factor compared with cumulative plastic deformation factor (AIJ Kinki 1997)

## A.4 Definition of cumulative plastic deformation factor

Several parameters have so far been used as the measure representing the performance of beam-column assemblies. The large-scale investigation used the cumulative plastic deformation factor as defined below.

An example of hysteresis loops of the flexural moment at the column face, $M_{cf}$ versus the rotation of the beam segment between the loading point and column face $\Theta_{cf}$ (see figure A.6) is shown in figure A.7.

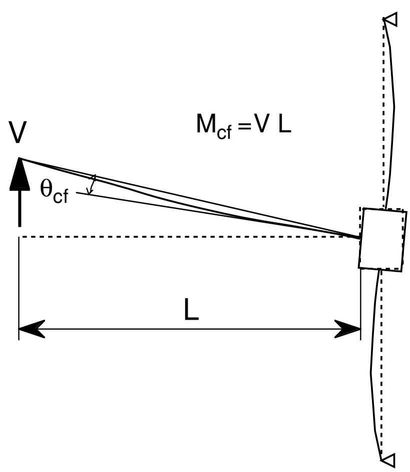


Figure A.6 – Definition of beam rotation and moment at column face

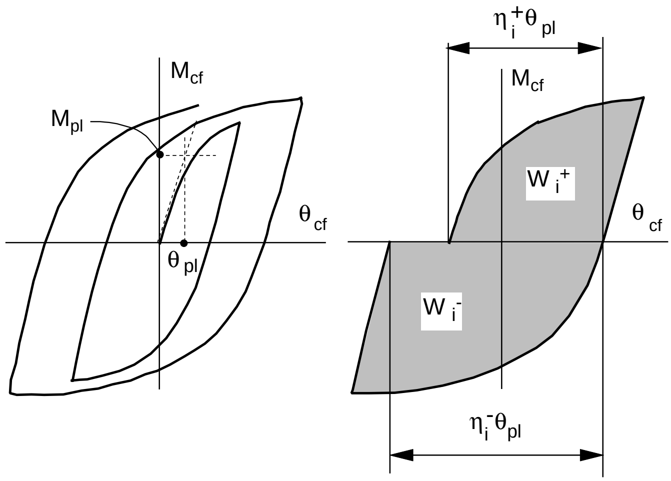


Figure A.7 – Definition of cumulative plastic deformation factor

The elastic beam rotation $\Theta_{pl}$ at the fully-plastic moment $M_{pl}$ is defined as the elastic component of beam rotation at $M_{cf} = M_{pl}$ (see figure A.7). $M_{pl}$ is calculated using the measured yield stresses of beam materials and measured dimensions of beam sections. The plastic components of beam rotation at the i-th half cycle, non-dimensionalised by dividing it by $\Theta_{pl}$, are denoted by $\eta_i^+$ and $\eta_i^-$, in which the + and - symbols distinguish positive and negative moments (see figure A.7). The cumulative plastic deformation factor is defined as the sum of $\eta_i^+$ and $\eta_i^-$ sustained by the specimen until failure occurs and is written as:

$$\eta = \sum_i \left( \eta_i^+ + \left| \eta_i^- \right| \right) \qquad \text{A.11}$$

The alternative definition of the cumulative plastic deformation factor is the sum of plastic energies dissipated during all the cycles, non-dimensionalized by dividing the energy by $M_{pl}\Theta_{pl}$. According to the latter definition $\eta_i^+$ and $\eta_i^-$ are written as:

$$\eta_i^+ = \frac{\sum_i W_i^+}{M_{pl}\Theta_{pl}} \text{ and } \eta_i^- = \frac{\sum_i W_i^-}{M_{pl}\Theta_{pl}} \qquad \text{A.12}$$

where $W_i$ denotes the energy absorbed at the i-th half cycle (see figure A.7).

The FEMA criteria (2000) are using the inter-storey drift angle as a performance parameter. The drift angle can be expressed as a function of $\eta$ and the overstrength factor given by equation A.10 by following the loading sequences adopted by the large-scale investigation. Assuming $\Theta_{pl} = 0.09$, which is equal to the average value of $\Theta_{pl}$ for the specimens used in these tests, the cumulative plastic deformation factor can be converted to the drift angle by the following equation:

$$\Theta = 0.0081\eta^{0.46} \qquad \text{A.13}$$

# Comité International pour le Développement et l'Étude de la Construction Tubulaire

# International Committee for the Development and Study of Tubular Structures

CIDECT, founded in 1962 as an international association, joins together the research resources of the principal hollow steel section manufacturers to create a major force in the research and application of hollow steel sections world-wide.
The CIDECT web site is www.cidect.com

**The objectives of CIDECT are:**

- to increase the knowledge of hollow steel sections and their potential application by initiating and participating in appropriate research and studies.
- to establish and maintain contacts and exchanges between producers of 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 good engineering practice and suitable architecture, in general by disseminating information, organising congresses, etc.
- to co-operate with organisations concerned with specifications, practical design recommendations, regulations or standards at national and international levels.

**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 connections
- Fatigue resistance of welded connections
- Aerodynamic properties
- Bending strength of hollow steel section beams
- Corrosion resistance
- Workshop fabrication, including section bending
- Material properties

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 CIDECT Design Guides, in the series “Construction with Hollow Steel Sections”, already published, is given below. These design guides are available in English, French, German and Spanish.

1. Design guide for circular hollow section (CHS) joints under predominantly static loading (1991)
2. Structural stability of hollow sections (1992, reprinted 1996)
3. Design guide for rectangular hollow section (RHS) joints under predominantly static loading (1992)
4. Design guide for structural hollow section columns exposed to fire (1995, reprinted 1996)
5. Design guide for concrete filled hollow section columns under static and seismic loading (1995)
6. Design guide for structural hollow sections in mechanical applications (1995)
7. Design guide for fabrication, assembly and erection of hollow section structures (1998)
8. Design guide for circular and rectangular hollow section welded joints under fatigue loading (2000)

In addition, as a result of the ever increasing interest in steel hollow sections in internationally acclaimed structures, two books “Tubular Structures in Architecture” (sponsored by the European Community) and “Hollow Sections in Structural Applications” (published by Bouwen met Staal) have been published.

Copies of the design guides, the architectural book and research papers may be obtained from:

The Steel Construction Institute
Silwood Park
Ascot
Berkshire SL5 7QN
England
Tel: +44-(0)1344-23345 Fax: +44-(0)1344-22944
e-mail: publications@steel-sci.com Website: http://www.steel-sci.org

“Hollow Sections in Structural Applications” is available from the publisher:
Bouwen met Staal
PO Box 29075
NL-3001 GB Rotterdam
The Netherlands
Tel: +31-10-411 5070 Fax: +31-10-412 1221
e-mail: info@bouwenmetstaal.nl

# CIDECT organisation (2004)

- President: B. Lugtenberg – United Kingdom
- Treasurer/secretary: R. Murmann – United Kingdom
- A General Assembly of all members meeting once a year and appointing an Executive Committee responsible for administration and execution of established policy
- A Technical Commission and a Promotions Commission meeting at least once a year and directly responsible for the research work and technical promotion work

Present members of CIDECT are:

- Aceralia Transformados, Spain
- Arcelor Tubes, France
- Borusan Muhendislik, Turkey
- Corus Tubes, United Kingdom
- IPSCO Inc., Canada
- Onesteel, Australia
- Rautaruukki Oy, Finland
- Sidenor, Greece
- Smorgon Steel Tube Mills, Australia
- Tata Tubes, India
- Vallourec & Mannesmann Tubes, Germany
- Voest Alpine Krems, Austria

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