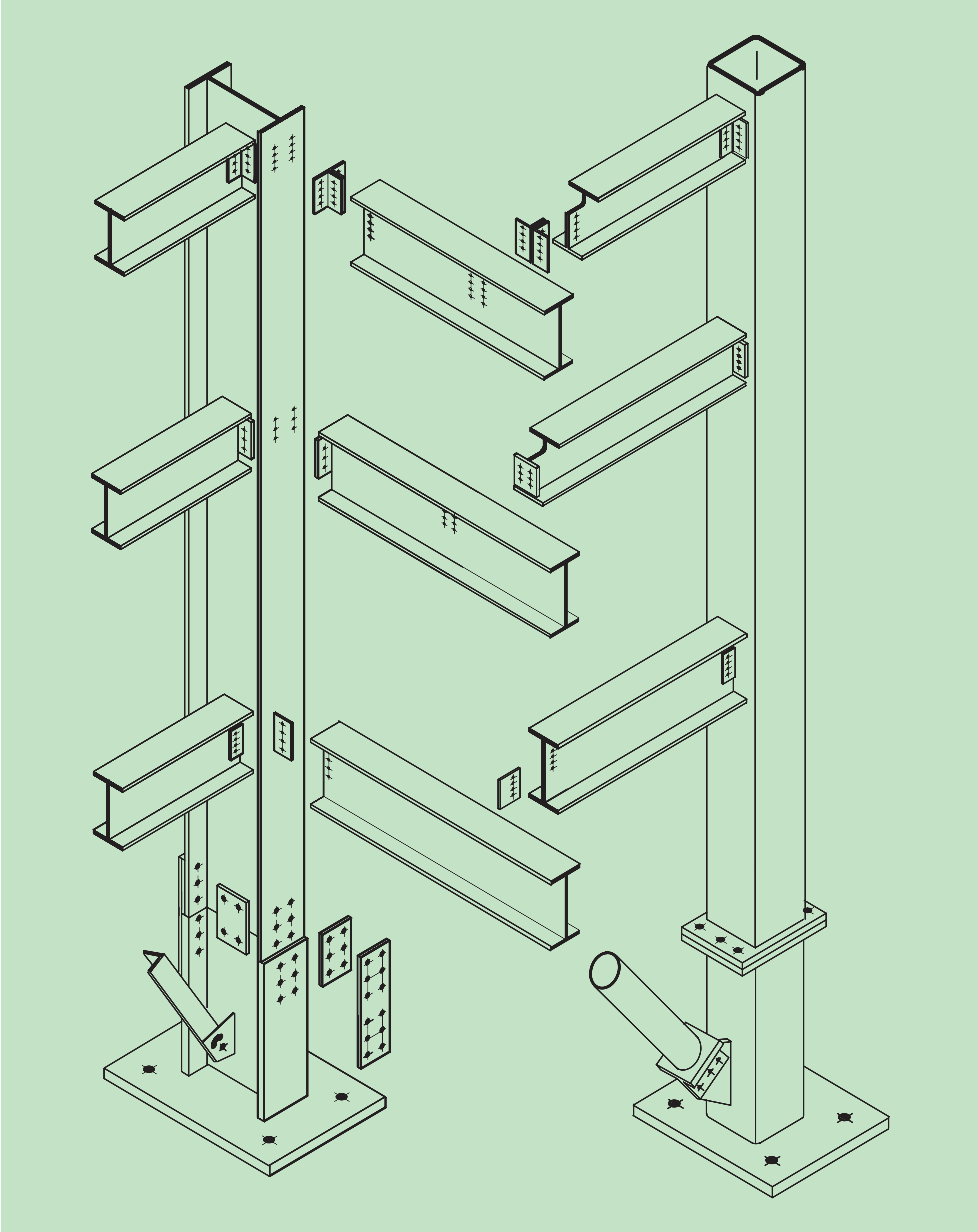

Joints in Steel Construction

Simple Connections

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FOREWORD

This publication is one of a series of "Green Books" that cover a range of steelwork connections. It provides design guidance for structural steelwork connections for use in buildings designed by the "Simple Method" i.e. braced frames where connections carry mainly shear and axial loads only.

Other books in the series are:

Joints in steel construction: Moment connections (P207/95), **which provides design guidance for connections which in addition to shear and axial loads, are required to resist bending moments.**

Joints in steel construction: Composite connections (P213/98), **which provides design guidance for moment resisting composite end plate connections.**

Design guidance for a range of simple connections was originally published in two separate volumes entitled *Joints in simple construction: Volume 1: Design Methods (P205) and Volume 2: Practical Applications (P206/92).* Availability of further research has enabled this publication to include design guidance for a wider range of simple joint and, at the same time, the opportunity has been taken to combine the two volumes.

The major additions to the previous publications (Volumes 1 and 2) are:

- **Inclusion of design guidance for bolted connections to hot finished structural hollow sections using Flowdrill or Hollo-Bolts.**
- **Use of fin plates for deep beams.**
- **Inclusion of design procedures for double lines of bolts in double angle web cleats and fin plates.**
- **Improved structural integrity guidance.**
- **Use of fully threaded bolts.**
- **Inclusion of bracing connections.**
- **Inclusion of slotted and kidney shaped holes.**

This publication was produced by the SCI/BCSA Connections Group, which was established in 1987 to bring together academics, consultants and steelwork contractors to work on the development of authoritative design guides for structural steelwork connections.

Reprinted edition

September 2002 - The first impression was issued
October 2002 – 2002 impression was issued with Corrigendum 1 (see page 491, (a))
July 2005 - 2002 impression was issued with Corrigendum 1 and Corrigendum 2 (see page 491, (b))
In accordance with corrigenda 1 and 2, the capacity tables for fin plates (H27 to H30) have been amended and included in this edition.

Note: ***Additional supplementary capacity tables for flexible end plates (see page 491 (c)) are not included in this edition. These are available on www.steelbiz.org as part of an advisory desk note AD 291.***

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* Editorial Committee members.

The editorial committee acknowledges the original work leading to *Joints in simple construction Vol. I (P205)* and *Vol.2 (P206/92)* which this publication has incorporated.

Assistance in preparing this combined and extended edition was provided by Peter Allen (formerly BCSA). The text and graphics were produced by Richard Stainsby. The connection capacity tables were produced by Nina Knudsen and Jiri Mares of SCI.

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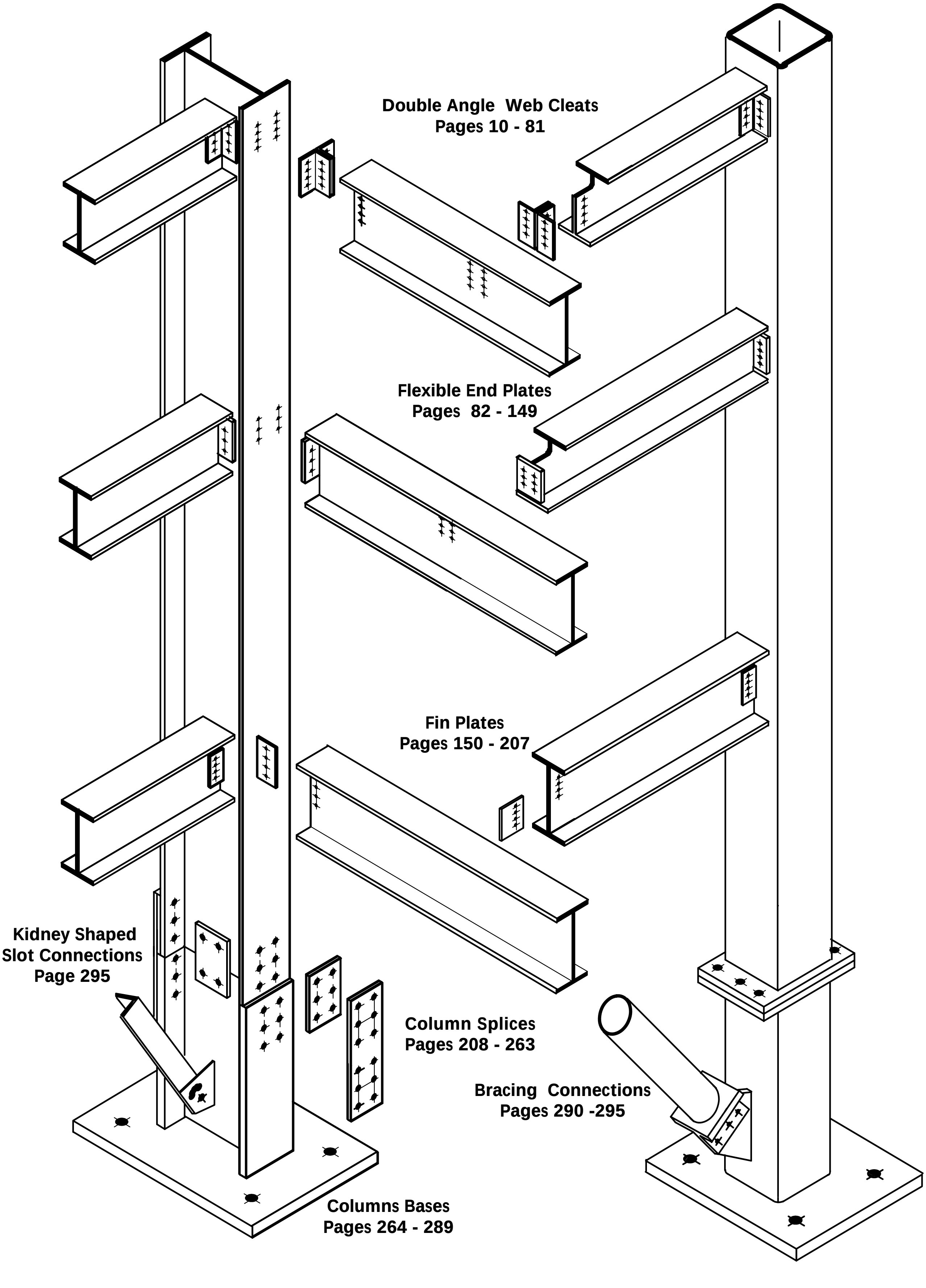
References to BS 5950-1:2000 have been made with the permission of British Standards Institution, BSI Customer Services, 389 Chiswick High Road, London, W4 4AL

PICTORIAL INDEX



CONTENTS

CONTENTS *(Continued)*

CONTENTS *(Continued)*

1. INTRODUCTION

1.1 ABOUT THIS DESIGN GUIDE

This guide provides procedures for designing connections in steel-framed structures in accordance with the recommendations of BS 5950–1:2000[1]. It uses the Simple design method described in clause 2.1.2.2. of the Code. Connections between universal beams and universal columns are included and between universal beams and hot finished structural hollow section columns using the Flowdrill and Hollo-Bolt systems. Designs are provided for:

(a) Beam-to-beam and beam-to-column connections

- Double angle web cleats (DAC)
- Flexible end plates (FEP)
- Fin plates (FP)

(b) Column splices

Bolted splices which may be cover plate or end plate type.

(c) Column bases

Slab bases welded to the column shaft.

(d) Bracing connections

Angles and hollow sections connected to other main members via a gusset plate. Types of suitable connections to facilitate site bolted assembly are provided with design considerations.

(e) Special connections

Special connections for cases when members do not align on a common centreline, at different levels or at a skew angle. Some guidance is given on these special connections.

Steel grades

Steel grades are designated by European classifications currently used by the steel construction industry in BS EN Specifications relating to steel and now adopted in BS 5950–1:2000[1]. Reference is only made to the strength requirements to which, for completeness, should be added the impact (or sub grade) designation. Table 1.1 shows the basic material grade now used and the designation previously employed.

Table 1.1 Steel grades

BS 5950–1:2000	Previous editions of BS 5950
S275	Design Grade 43
S355	Design Grade 50

Design procedures

Individual design procedures are included to check all relevant elements, welds and bolts within the connection. The procedures start with the detailing requirements (joint geometry) then presents the checks for each stage of the load transition through the complete connection including welds, plates, bolts and the section webs or flanges as appropriate. The capacity checks on sections, welds and bolts are all based on BS 5950–1:2000[1].

The design guidance for hollow section columns is at present restricted to hot finished structural hollow sections.

Capacity tables

Without access to suitable software, designing simple connections can be a long and tedious process. To help overcome this problem, capacity tables for standard connections are provided in the yellow pages of this guide.

The capacity tables have been arranged so that the designer can simply select a connection and, with the minimum of calculation, check whether it has sufficient capacity. Tables can also aid designers with initial member selection. The tables use rolled sections to BS 4[2], BS EN 10210[3] and BS EN 10219[4] but other profiles may be used, referring to the design procedures where necessary.

A key aim in producing capacity tables is the standardisation of bolts and fittings, a concept that is widely recognised as improving the efficiency of the steelwork industry.

Design examples

Worked examples illustrating the design procedures and the use of capacity tables are also included.

1.2 CONNECTION CONSIDERATIONS

Classification

In keeping with the design methodology of the overall structure, simple connections are those which comply with the following five conditions:

(i) are assumed to transfer the design shear reaction only between members

(ii) are incapable of transmitting significant moments which might adversely affect members of the structure

(iii) are capable of accepting the resulting rotation

(iv) provide the directional restraint to members which has been assumed in the member design

(v) have sufficient robustness to satisfy the structural integrity requirements.

Structural integrity

It is a recommendation of BS 5950–1:2000[1], clause 2.4.5.2 that all building frames be effectively held together at each principal floor and roof level. Tests carried out at the Building Research Establishment[5] and described in Appendix A show that connections designed and detailed in accordance with this guide, will generally be capable of carrying the basic 75 kN tying force requirement.

Clause 2.4.5.3 recommends that where Building Regulations require that certain buildings should be specially designed to avoid disproportionate collapse, the tying force should be increased above the minimum 75kN. The increased tying force is normally equal to the end reaction, but can sometimes be less depending on how the floor beams are arranged.

Appendices A to D give information on the behaviour and methodology for designing connections to resist tying forces.

Stability during erection

The Steelwork Contractor is usually responsible for stability of the structure during erection.

In simple construction, stability against lateral loading is generally provided by the floor and roof acting as stiff diaphragms to transmit loads horizontally to points of sway resistance such as vertical bracings, shear walls or lift cores. However, during erection there will invariably be stages when the floors or shear walls are incomplete and it is vital that stability is maintained at all times.

In assessing these temporary cases, consideration must be given to many factors such as:

- the wind loads on the unclad structure[6]
- the stiffness of the vertical frames arising from any connection and base fixity
- the ability of partially finished floors to provide diaphragm action.

As most of the connections in this guide have limited rotational stiffness, it may be necessary to arrange the construction so that the floors are decked out in sequence and temporary bracing provided. This will provide both stability to the framework and allow any concrete placement to keep up with the steelwork erection, thus preventing delays to the construction programme. Ideally, any temporary bracing will be in the form of wires with turnbuckles or other tensioning systems rather than steel angles or flats. Tensioned wires have the added benefit of helping to plumb the frame as well as being easier to remove on completion.

Connections types

The selection of beam end connections can often be quite involved. The relative merits of the three connection types included in this guide are summarised in Table 1.2.

Bearing connections

When designing column splices and bases it should be noted that the bearing connections are suitable in most situations. They are less expensive to fabricate and quicker to erect than the non-bearing types. A Steelwork Contractor with reasonable quality sawing equipment can achieve the necessary level of accuracy for a bearing connection.

Composite floors

It is recognised that interaction with a composite floor will affect the behaviour of a simple connection. Common practice is to design such connections without utilising the benefits of the continuity of reinforcement through the concrete slab. However, *Joints in steel construction: Composite connections*[7], enables reinforcement continuity to be allowed for in providing relatively simple flush end-plate connections with substantial moment capacity.

1.3 EXCHANGE OF INFORMATION

The design of the frame and its connections is usually carried out in one of the following ways:

(i) The frame is designed by the Consulting Engineer and the connections are designed by the Steelwork Contractor

(ii) The frame and the connections are designed by the Steelwork Contractor.

(iii) The frame and its principle connections are designed by the Consulting Engineer.

Where method (i) is in operation, care must be taken to ensure that design requirements for the connections are clearly defined in the contract and on the design drawings.

The National Structural Steelwork Specification for Building Construction[8] gives guidance on the transfer of information and there will be great benefits if this is observed. The following items should be considered a minimum:

- a statement describing the design concept
- drawings, or equivalent electronic data, showing the size, grade and position of all members
- the design standards to be used
- the forces required to be transmitted by each connection
- whether the loads shown are factored or unfactored
- requirements for any particular type of fabrication detail and/or restriction on the type of connection to be used.

Table 1.2 Relative merits of beam end connection types

	Double angle web cleats (DAC)	**Flexible end plate (FEP)**	**Fin plate (FP)**
Design			
Shear capacity - percentage of beam capacity	Up to 50% *Up to 75% with two vertical lines of bolts*	Up to 75% *100% with full depth plates*	Up to 50% *Up to 75% with two vertical lines of bolts*
Tying capacity	Good	Fair, but non-standard plates may be required	Good
Special considerations			
Skewed Joints	Unsuitable	Fair	Good
Beams eccentric to columns	Poor	Fair	Good
Connection to column webs	Fair *Standard cleats unsuitable for 152 or 203 UC's*	Good	Fair *To facilitate erection, flange stripping may be required. Sfiffening may be required for long fin plates (Figure 6.7)*
Fabrication and treatment			
Fabrication	Good *No welding required*	Good	Good *Stiffening may be required for long fin plates (Figure 6.8)*
Surface treatment	Fair *Components treated separately before assembly*	Good	Good
Erection			
Ease of erection	Fair *Difficult for two-sided connections*	Fair *Difficult for two-sided connections*	Good
Site adjustment	Good	Fair	Fair
Temporary stability	Fair	Fair	Fair

1.4 COSTS

Simple connections are invariably cheaper to fabricate than moment resisting connections, because they provide a significant degree of simplicity and standardisation.

Giving specific guidance on costs is difficult, as Steelwork Contractor's workmanship rates can vary considerably and are dependant upon the level of investment in plant and machinery. However, the designer's and detailer's main objective must be to reduce the work content. The material cost for fittings and bolts are small compared with workmanship costs. In a typical fabrication workshop the cost of fabrication of connections may be 30% to 50% of the total fabrication cost.

The real costs come from the time taken to design the connection, detail it, make the fittings, mark out the geometry, drill the holes and complete the welding and testing. These costs can be minimised by adopting standard connections, as discussed in Section 2, and by early consultation with the appointed Steelwork Contractor. For further information see *Design for Manufacture Guidelines* [9]

1.5 MAJOR SYMBOLS

The major symbols used in this Guide are listed below for reference purposes. Other are described where used.

A_s	Shear area of bolt
a	Effective throat of weld
B	Width of section (Subscript c or b refers to column or beam)
D	Depth of section (Subscript c or b refers to column or beam)
D_h	Hole diameter
d	Depth of web between fillets *or* diameter of a bolt
e	End distance
F_s	Shear Force in bolt
F_v	Vertical reaction
g	Gauge (Transverse distance between bolt centrelines)
P	Capacity (Subscripts c - compression, b - bearing, s - shear)
p	Bolt spacing ('pitch')
p_y	Design strength of steel
r	Root radius of section
s	Fillet weld leg length
T	Thickness of flange
t	Thickness of web
t_p	Thickness of plate
U_s	Ultimate tensile strength
Z	Elastic modulus (Subscript b refers to bolt group modulus)

Lengths and thicknesses stated without units are in millimetres.

2. STANDARDISED CONNECTIONS

2.1 THE BENEFITS OF STANDARDISATION

In a typical braced multi-storey frame the connections may account for less than 5% of the frame weight, and yet probably 30% or more of the total cost. Efficient connections will therefore have the lowest detailing, fabrication and erection labour content - they will not necessarily be the lightest.

A problem facing the connection designer today is the bewildering range of options he has in selecting:

- the type of connection
- grades and sizes of fittings
- bolt grades, sizes and lengths
- weld types and sizes
- the geometry to adopt.

In preparing this Guide, one of the main objectives has been to carry out a rationalisation and to recommend standards in each of the above areas. This procedure leads to a standard connection where the fittings, bolts, welds and geometry are fully defined.

The benefits of this approach are as follows:

- A reduction in the number of connection types which:
  - leads to a better understanding of their cost and performance by all sides of the industry
  - encourages the development of design aids and computer software.
- The use of a few standard flats or angles for fittings which:
  - improves availability
  - leads to a reduction in material costs
  - reduces buying, storage, and handling time.
- The use of one grade and diameter of bolt in a limited range of lengths which:
  - saves time changing drills or punches in the shop
  - leads to faster erection and fewer errors on site.
- The use of small, single pass fillet welds which:
  - avoids the need for any edge preparation
  - reduces the amount of non-destructive testing required.

In practice, steel structures can be complex and there will be times when the standard connections presented here are not suitable. However, even in these cases it will still be possible to adopt some of the general principles of standardisation such as limiting the range of fittings, sections and bolt sizes.

A summary of the recommended components adopted for this Guide is shown in Table 2.1.

Table 2.1 Recommended components

Component	Preferred Option	Notes
Fittings	Material of grade S275 Limited range of standard flats and angles	– (see Table 2.2)
Bolts	M20 grade 8.8 Bolts	– Some heavily loaded connections may need larger diameter bolts – Foundation bolts M24 grade 4.6
Holes	22mm diameter punched or drilled, or 22mm x 26mm slottted holes made by: – punching in one operation – formed by drilling two holes and completed by cutting – machine operated flame cutting	– 26mm dia for M24 bolts – 6mm oversize for Foundation bolts
Welds	Fillet welds with E35 electrodes 6mm or 8mm leg length	– larger welds may be needed for some column bases

2.2 COMPONENTS

Fittings

With the exception of column bases and some components for heavy column splices that have to be cut from plate, fittings for simple connections are normally manufactured from standard flats or angles.

Table 2.2 lists the recommended range of fitting sizes which have been adopted throughout this Guide.

There can be problems obtaining flats and angles in higher grade steel (S355) and for this reason it is recommended that all fittings material should be of grade S275. The adoption of one grade of steel for fittings also assists the Steelwork Contractor with the control of his stock.

Table 2.2 Recommended sizes of flats and angles

FITTINGS			LOCATION		
Type	**Size mm**	**Thickness mm**	**Web Cleat**	**End Plate**	**Fin Plate**
Flat	100	10			•
	150	8		•	
	150	10			•
	200	10		•	
Angle	90 x 90	10	•		
	150 x 90	10	•		

Bolts

Structural bolting practice for both rolled section members and hollow section members is based predominantly on non-preloaded bolts of strength grades 4.6 and 8.8 used in 2mm clearance holes. Such bolts are termed ordinary bolts and are specified in BS 4190[10] and other standards (see Table 2 in BS 5950-2: 2001[1]).

Grade 8.8 bolts to BS 4190 are commonly available and are recommended for all main structural connections, with the standard bolt being 20mm diameter. Grade 4.6 bolts are generally used only for fixing lighter components such as purlins or sheeting rails, when 12mm or 16mm bolts may be adopted. For holding down bolts, see Section 8.2.

There may be situations, for example a column splice subjected to large load reversals in a braced bay, where the engineer feels that joint slip is unacceptable. In these cases the general grade high strength friction grip bolt to BS 4395[11] should be used.

The mixing of different grades of bolt in the same diameter on any one project should be avoided.

In order to avoid the question on site of acceptability of threads in the shear plane.The determination of bolt shear capacity should assume that the threads are in the shear plane. All the capacity tables and examples in this Guide follow this assumption.

Fully threaded bolts

Common practice in the past has been to use bolts with a short thread length, i.e. 1.5d, and to specify them in 5mm length increments. This can result in an enormous number of different bolts, which is both costly to administer and can prevent rapid erection.

It is recommended that fully threaded bolts (technically known as screws) be used as the industry standard. They can be provided longer than necessary for a particular connection and can therefore dramatically reduce the range of bolt lengths specified. For example, the M20 x 60mm long grade 8.8 fully threaded bolt has been shown to be suitable for 90% of the connections in a typical multi-storey frame. Table 2.3 shows a rationalised range of sizes.

Table 2.3 Recommended sizes - fully threaded bolts

Grade	Diameter	Length (mm)		
*4.6	M12	25	-	-
*4.6	M16	30	-	-
8.8	M16	30	45	-
8.8	**M20**	45	**60**	-
8.8	M24	70	85	100

** Recommended only for use in fixing cold rolled purlins and rails.*

Fully threaded fasteners (4.6 and 8.8 Screws) should be specified to BS 4190: 2001

Research has demonstrated that the very marginal increase in deformation with fully threaded bolts in bearing has no significant effect on the performance of a typical joint. In the specific instances where this additional deformation might be of concern, it is normal and recommended practice to use preloaded (HSFG) bolts. These can be used for example in tension and compression splices where the bolts are in shear/bearing or in column splices where the column ends are not in bearing. A paper by Owens[12] gives the background to the use of fully threaded bolts in both tension and shear conditions.

Bolting to Rectangular Hollow Sections

Bolted connections to rectangular hollow sections (RHS) using the Flowdrill or Hollo-Bolt systems may also be standardised using M20 8.8 bolts but the bolt length must be chosen to suit the fastener system.

For further information on Flowdrill system, see Appendix F and Tables H.55a, H.55b and H.60.

For further information on Hollo-Bolt system, see Appendix G and Tables H.56 and H.61.

Holes

Normal practice is for holes to be drilled in main sections and either punched or drilled in fittings. Hole sizes should be as follows:

Ordinary or HSFG Bolts ≤ 24mm dia: bolt dia + 2mm
Ordinary or HSFG Bolts > 24mm dia: bolt dia + 3mm
Holding down bolts: bolt dia + 6mm

With slab bases thicker than 60mm, the normal clearance of 6mm is often insufficient to allow the necessary adjustment and this may need to be increased.

Holes of 22mm or 26mm can be safely punched through grade S275 material up to 12mm thick, which covers all the standard fittings for beam end connections in this Guide. Further guidance on hole punching can be found in the *National Structural Steelwork Specification*[8].

As will be seen in Section 4, which deals with double angle web cleats, short slotted holes 22mm dia x 26mm long may be used in certain instances. This hole can be punched full size in a single operation.

Welds

All the welds used for connections in this Guide are simple fillet welds carried out by a metal arc process in accordance with BS EN 1011–1[13].

The standard leg length is 6mm or 8mm, which can easily be laid in a single run. Fillet welds of this size are generally recognised as being extremely reliable and will normally need little testing beyond the usual visual inspection.

2.3 GEOMETRY

Beam notches

Notching is normally required for beam-to-beam connections to enable the supported beam to frame into the web of the supporting beam (see Figure 2.1).

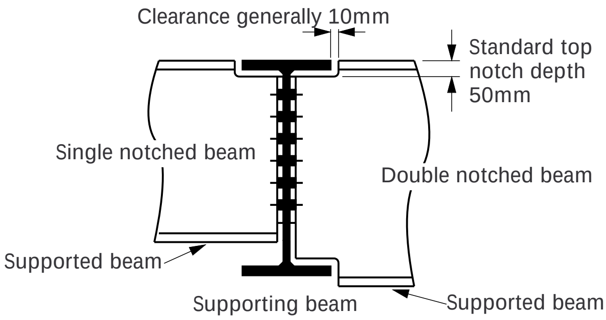


Figure 2.1 Beam notches

The notch length should provide a nominal clearance of 10mm from the edge of the supporting beam flange and will vary depending on the width of the supporting beam, but should be kept to a minimum to avoid local stability problems.

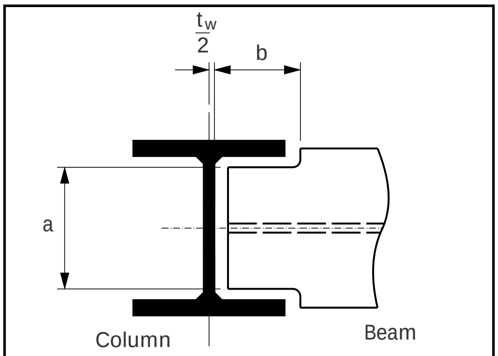
A standard top notch depth of 50mm is recommended and this will be adequate for all but the largest UB and UC sections. Apart from encouraging the use of templates for marking out, the main benefit of a standard notch depth is that it allows the top row of bolts to be positioned a consistent 90mm below the top of the beam - the normal reference point. Specific notch dimensions for a particular rolled section can be found in the dimension tables in the yellow pages.

If a hole is to be drilled at the root to form a radius, then this should be the standard 22mm diameter to avoid changing drills. If the notch is to be cut square, then great care must be taken to avoid overcutting at the corner. Coping machines can usually cut the notch square or with any specified radius.

Flange trims

For some beam to column web connections, the edges of the beam flanges may need to be trimmed as shown in Figure 2.2. The dimensions given make allowance for the beam to swing down into position in the column web, clearing any bolt heads present from beams already connected to the column flanges.

This extra fabrication is obviously costly but can often be avoided by the careful selection of beam and column sizes.



Column	Flange Trim (mm)	
	a	b
356 UC	240	190
305 UC	195	160
254 UC	150	135
203 UC	110	110
152 UC	70	85

Figure 2.2 Flange Trims

Flange chamfers

Occasionally, when beams of different depths connect into opposite sides of a column or beam web, the bottom flange of the smaller beam may clash with the bolts connecting the deeper beam. To avoid using special fittings with non-standard bolt pitches, a bottom flange chamfer can be used as shown in Figure 2.3. This chamfer will give adequate clearance for 20mm bolts at 90mm or greater cross centres.

Vertical bolt layout

The recommended vertical bolt layout using M20 bolts that has been adopted throughout this Guide for all the beam end connections is shown in Figure 2.4.

The tops of all fittings should be placed 50mm below the top of the beam, which for beams with a standard notch, positions them flush with the notch top. This leads to the first bolt row being set down a constant 90mm from the top of the beam, which is generally the setting out point.

For the fittings, a top and bottom edge distance to the bolts of 40mm has been used. This complies with the 2d minimum specified for fin plates in design procedure Check 1(Section 6) and will prevent premature bearing failure with M20 bolts.

Traditionally, the vertical bolt pitch 'p' has varied depending on the type of connection, the section sizes, the material grades and the whim of the detailer! The standard pitch of 70mm which is recommended here has been found to be an optimum solution which will satisfy most conditions. In practice the benefits of using a standard layout will far outweigh any possible savings that might come from varying the pitch and omitting one or two rows of bolts.

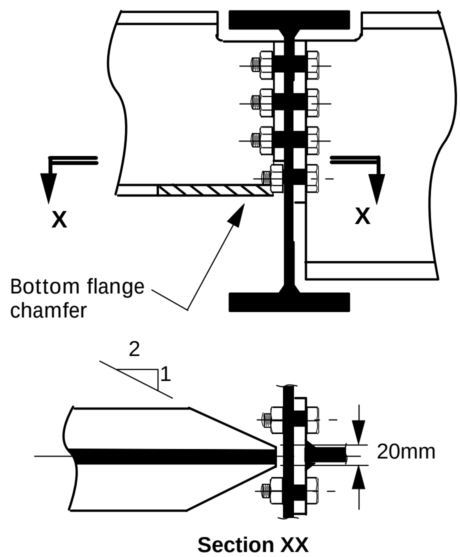


Figure 2.3 Flange chamfers

Bolt gauge

For universal beams and columns the bolt gauge or cross centres has been set at 90mm or 140mm for end plates and 100mm plus the beam web thickness (t_w) for web cleats. These dimensions are designed to provide reasonable clearance for bolt access as well as giving sufficient width for flexibility in end plate or web cleat connections.

The bolt gauge may have to depart from these standard dimensions when connecting to the face of RHS sections because the gauge should be ≥ 0.3 x the RHS face width.

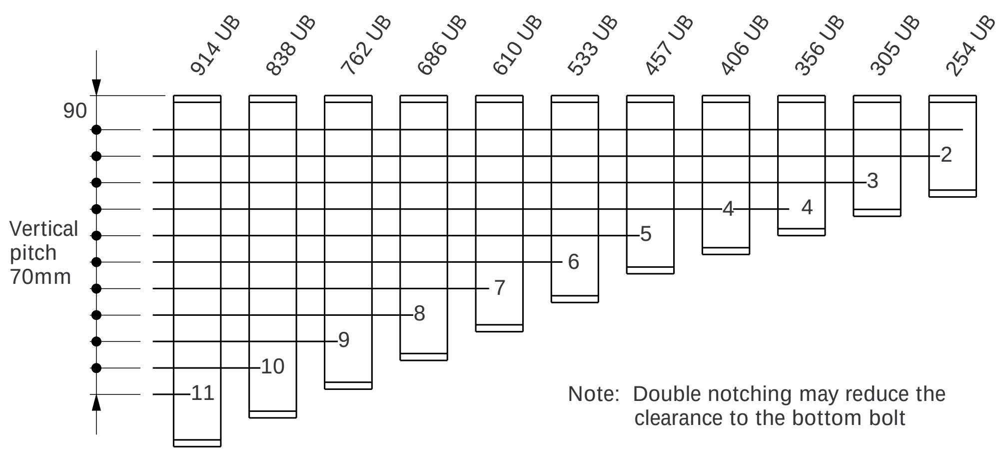


Figure 2.4 Vertical bolt layout (beam end connections using M20 bolts)

3. BEAM-to-BEAM and BEAM-to-COLUMN CONNECTIONS

3.1 INTRODUCTION

The design procedures which follow are suitable for either hand calculation or for the preparation of computer software.

Designing connections by hand is a laborious process and so a full set of capacity tables has been included in the yellow pages at the back of the guide.

Checking the strength of a simple connection involves three stages:

(1) Ensuring that the connection is detailed so that it develops only nominal moments which do not adversely affect the members or the connection itself.

(2) Identifying the load path through the connection i.e. from the beam to the supporting member.

(3) Checking the strength of each component, ensuring each is capable of transferring the load from one part to the next.

For normal design there are ten design procedures required to check all the parts of a beam-to-beam or beam-to-column connection for vertical shear. A further six checks are required where structural integrity of the joint must be considered, (see Table 3.1).

The final six checks are associated with the structural integrity requirements of BS 5950-1:2000[1], whereby beam/column connections must be able to resist lateral tying forces unless these forces are resisted by other means within the construction framework e.g. the floor slabs.

Table 3.1 summarises the design procedure checks required for Double Angle Cleats (DAC), Flexible End Plates (FEP) and Fin Plates (FP). The design procedures are described in Sections 4.5, 5.5 and 6.5.

Table 3.1 Design procedure for beam connections - summary table

	Design procedure checks	DAC	FEP	FP
	Design model	✔	✔	✔
1	**Recommended detailing practice**	✔	✔	✔
2	**Supported beam** – Bolt group/Welds	✔	✔	✔
3	**Supported beam** – Connecting elements	✔	N/A	✔
4	**Supported beam** – Capacity at the connection	✔	✔	✔
5	**Supported beam** – Capacity at a notch	✔	✔	✔
6	**Supported beam** – Local stability of notched beam	✔	✔	✔
7	**Unrestrained Supported beam** – Overall stability of notched beam	✔	✔	✔
8	**Supporting beam/column** – Bolt group/Welds	✔	✔	✔
9	**Supporting beam/column** – Connecting elements	✔	✔	N/A
10	**Supporting beam/column** – Local capacity	✔	✔	✔
11	**Structural Integrity** – Connecting elements	✔	✔	✔
12	**Structural integrity** – Supported beam	✔	✔	✔
13	**Structural integrity** – Tension bolt group/Welds	✔	✔	N/A
14	**Structural Integrity** – Supporting column web (UC or UB)	✔	✔	✔
15	**Structural integrity** – Supporting column wall (RHS)	✔	✔	✔
16	**Structural integrity** – Supporting column wall (CHS)	N/A	N/A	✔

Notes: (i) The detail check is necessary where there is a tick (✔) in the column and is not appropriate where indicated as N/A.

(ii) Checks on the bending, shear, local and lateral buckling capacity of a notched beam section are included in this table as it is usually at the detailing stage that the requirement for notches is established, following which, a check must be made on the reduced section.

4. DOUBLE ANGLE WEB CLEATS

4.1 INTRODUCTION

The double angle web cleat connection consists of a pair of angle cleats that are usually bolted to the supported beam web in the shop and the beam assembly is then bolted to the supporting member (Beam, **I** column or RHS column) on site. Flowdrill or Hollo-Bolts are used for connections to RHS columns, (see Figure 4.1).

The connection requires no welding and permits minor site adjustment when using untorqued bolts in clearance holes. However, it lacks flexibility in accommodating beam skews and there is difficulty in connecting into shallow depth columns. It is also not as strong as an equivalent end plate connection.

The rotational capacity of the connection is governed principally by the deformation capacity of the angles and, to a lesser extent, by the slip between the connected parts, (see Figure 4.2).

To minimise rotational resistance, the thickness of the angle cleats should be kept to a minimum and the bolt cross-centres (or gauge) should be relatively large.

The connection moment is indeterminate but small and can be neglected.

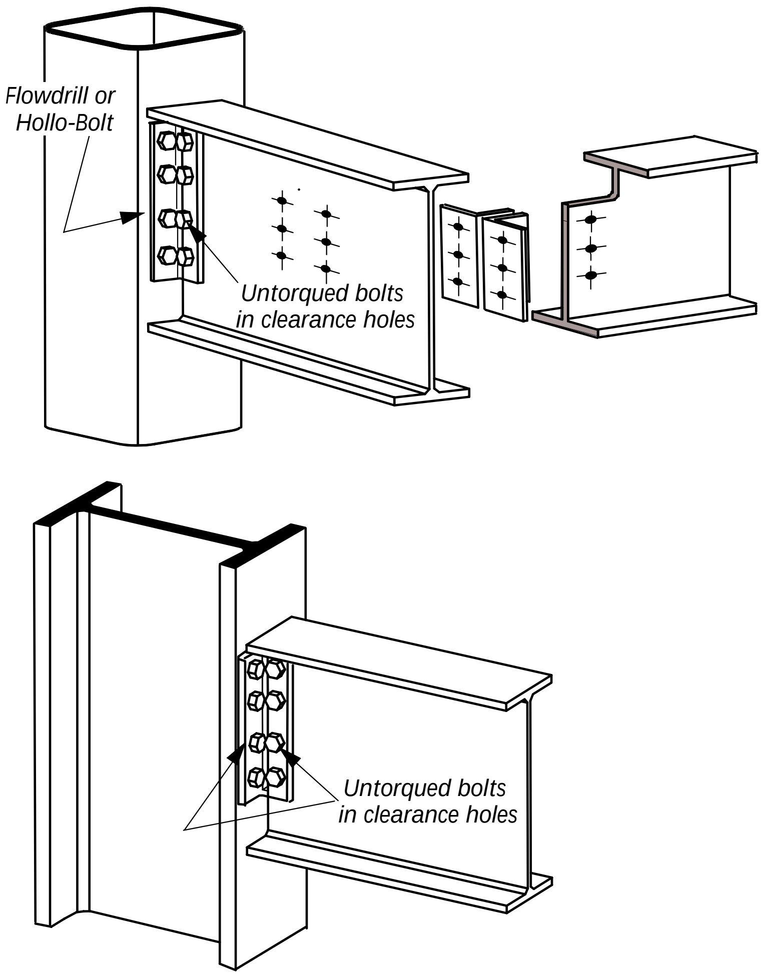


Figure 4.1 Double angle cleat. Beam-to-column and beam-to-beam connections

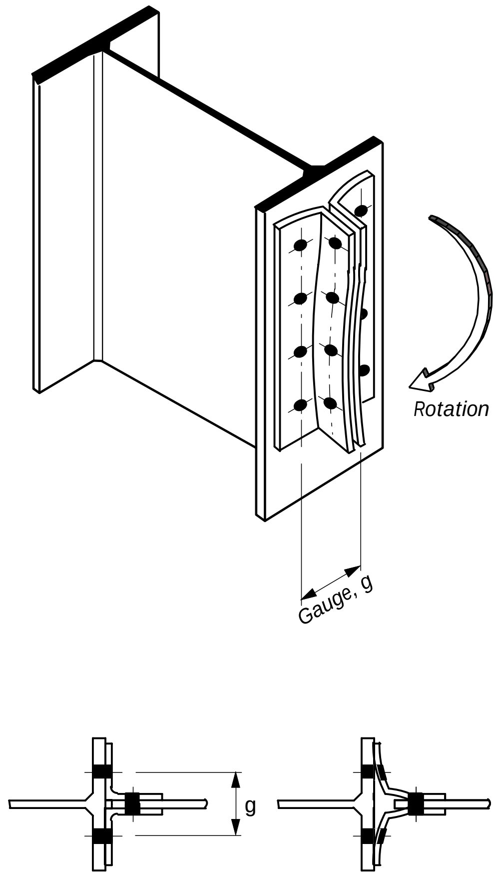


Figure 4.2 Deformation of angle cleats

4.2 PRACTICAL CONSIDERATIONS

Angle cleats are normally made by punching and cropping, with the holes in the beam and supporting members usually being drilled.

One problem of detailing arises when beams with different web thicknesses frame into opposite sides of a supporting beam or column web. For minor variations in the web thickness the normal bolt clearance holes will allow the bolt gauge to be set as normal. However, once the difference is more than 3mm the gauge will usually be set at 100mm + t_w based on the larger beam, and then cleats with an increased back-mark made for the smaller beam. This obviously has its drawbacks and can lead to difficulties with identification, especially if the back-mark in one leg only varies by, say, 2 or 3mm from the standard. A solution to this problem is to use a short slotted hole, as described below.

The slotted hole connection

A short slotted hole 22mm x 26mm long can be used in the outstanding leg of the cleat as shown in Figure 4.3.

The bolt gauge in the support is set constant at 120mm and, by using cleats with a 55mm back-mark to the centre of the slot hole, beams with web thickness from 6mm to 14mm can be accommodated. This caters for all the main UBs with the exception of some sections at the top end of the range.

Although the bearing capacity of the cleat around the slotted hole is reduced (see BS 5950-1[1], clause 6.3.3.3) this check will rarely be critical.

When the connection is bolted up on site, washers should be used over the slotted holes and the bolts firmly tightened. However, care must be taken not to use slotted holes in situations where vibration or dynamic loads are present which might lead to the possibility of joint slip.

Erection

The double angle web cleat is a good connection in terms of its facility for site adjustment. The two sets of bolts are both placed in clearance holes allowing slight adjustment in two directions before the bolts are tightened. In addition to this, packs can be placed between the cleats and the supported member if required.

With two sided connections that share a common set of bolts, the shop bolts should be placed with heads in opposite directions in the connecting pair of beams and the site bolts placed as shown in Figure 4.4. In some cases, it may be necessary to place the nut over the hole and turn the bolt into the nut.

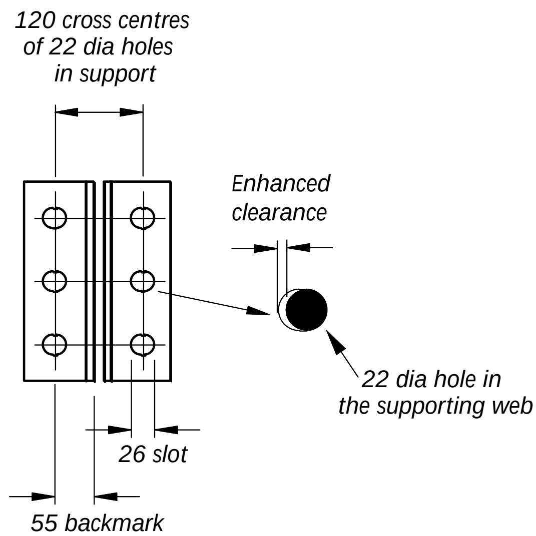


Figure 4.3 Short slotted holes in Double Angle Cleats

To facilitate the erection of a pair of larger beams it may be necessary to provide some form of support during erection. (See Figure 5.3 in Section 5)

4.3 RECOMMENDED GEOMETRY

The design procedures which follow set down a number of recommended details that are intended to achieve the required connection ductility.

When detailing the joint, the main requirements are as follows:-

(i) the cleats are positioned close to the top flange in order to provide positional restraint;

(ii) the cleats are at least 0.6 x the supported beam depth in order to provide the beam with adequate torsional restraint;

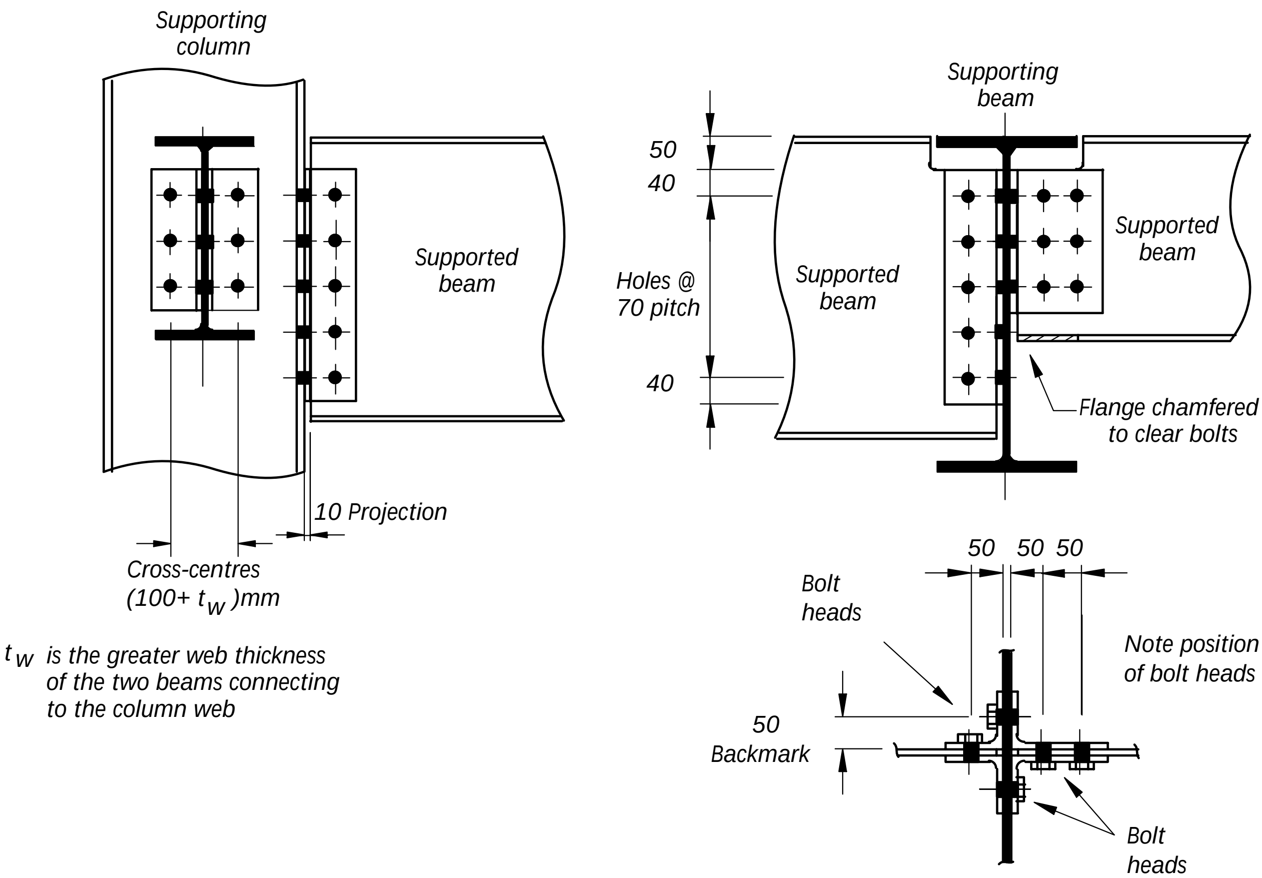
(iii) the cleats are relatively thin (8mm or 10mm);

(iv) the bolts in the supporting member are at reasonable cross centres (100mm + beam web thickness).

The first two requirements ensure that in those cases where the beam is laterally unrestrained, it can be designed with an effective length of 1.0L. (BS 5950–1[1]: Table 13)

The last two requirements ensure adequate ductility to classify as "simple connections".

These requirements, together with the standard geometry presented in Section 2, have been used to create the ‘standard connection’ shown in Figure 4.4.



Beam-to-column **Beam-to-beam**

Recommended cleat size	Vertical bolt lines
90 x 90 x 10 Angle	1
150 x 90 x 10 Angle	2

Bolts: M20 grade 8.8 in 22mm diameter holes
Cleats: Steel grade S 275, minimum length 0.6D where D is depth of supported beam

Note: For the smaller UBs, up to 457mm deep, a 90 x 90 x 8mm angle cleat is sometimes used, and a case can also be made for using 90 x 90 x 12mm angles for the heavier beams when shear in the cleat leg becomes critical. The use of 10mm thick cleats is recommended for standard connections.

Figure 4.4 Standard double angle web cleat connections

4.4 DESIGN

The full design procedure is presented in Section 4.5.

With a single vertical line of bolts in the beam web, the best that can be achieved from a web cleat connection is a connection shear capacity of around 50% of that of the beam in shear. Two vertical lines of bolts will increase this value up to a maximum of 75%, although it will be found that due to the increased eccentricity of the design load, the benefit is disproportionate to the additional bolts used.

For a configuration such as that shown in Figure 4.4, it will be found that shear capacity in a connection with one vertical line of bolts is generally governed by bolt bearing on the web of the supported beam (Check 4). Block shear failure of the beam web may also be critical (Check 4).

Check 3, dealing with the shear capacity of the angle cleat leg, generally becomes the critical mode for larger size beams when a connection has to be made with two vertical rows of bolts. If extra capacity is needed in these cases, then 150 x 90 x 12mm thick cleats may be the solution. This will increase the rotational stiffness of the connection, but it will not be detrimental.

Structural integrity

All floor beam-to-column connections must be designed to resist a tying force of at least 75kN - a force which can be carried by the simplest of cleats. *(See Appendix A).* For certain tall, multi-storey buildings it will be necessary to check the connection for large tying forces to satisfy the structural integrity requirements of BS 5950–1[1].

Generally it will be found that the tying capacity of a web cleat connection is adequate – mainly because of its ability to accept large deformations before failure. Checks 11 to 15 give quick and reasonable results, although reference should be made to Appendix B if a more rigorous approach is necessary.

If a standard connection is unable to carry large tying forces, then extra capacity can be achieved by increasing the cleat thickness and/or reducing the bolt gauge in the support. However, in these cases consideration should be given to the increase in the rotational stiffness of the connection.

The tying force should be considered in isolation and not coexistent with other loads.

Worked examples

Five worked examples are provided in Section 4.6 to illustrate the full set of design checks of Section 4.5.

CONNECTION CAPACITY TABLES

Capacity tables for double angle web cleat connections using ordinary or Flowdrill bolts are given in Tables H.9 to H.12 and in Tables H.13 to H.16 for Hollo-Bolt connections in the yellow pages. The tables include connections with one or two lines of bolts, using both S275 or S355 beams and are detailed in accordance with the standard geometry presented in Figure 4.4 and Table H.7.

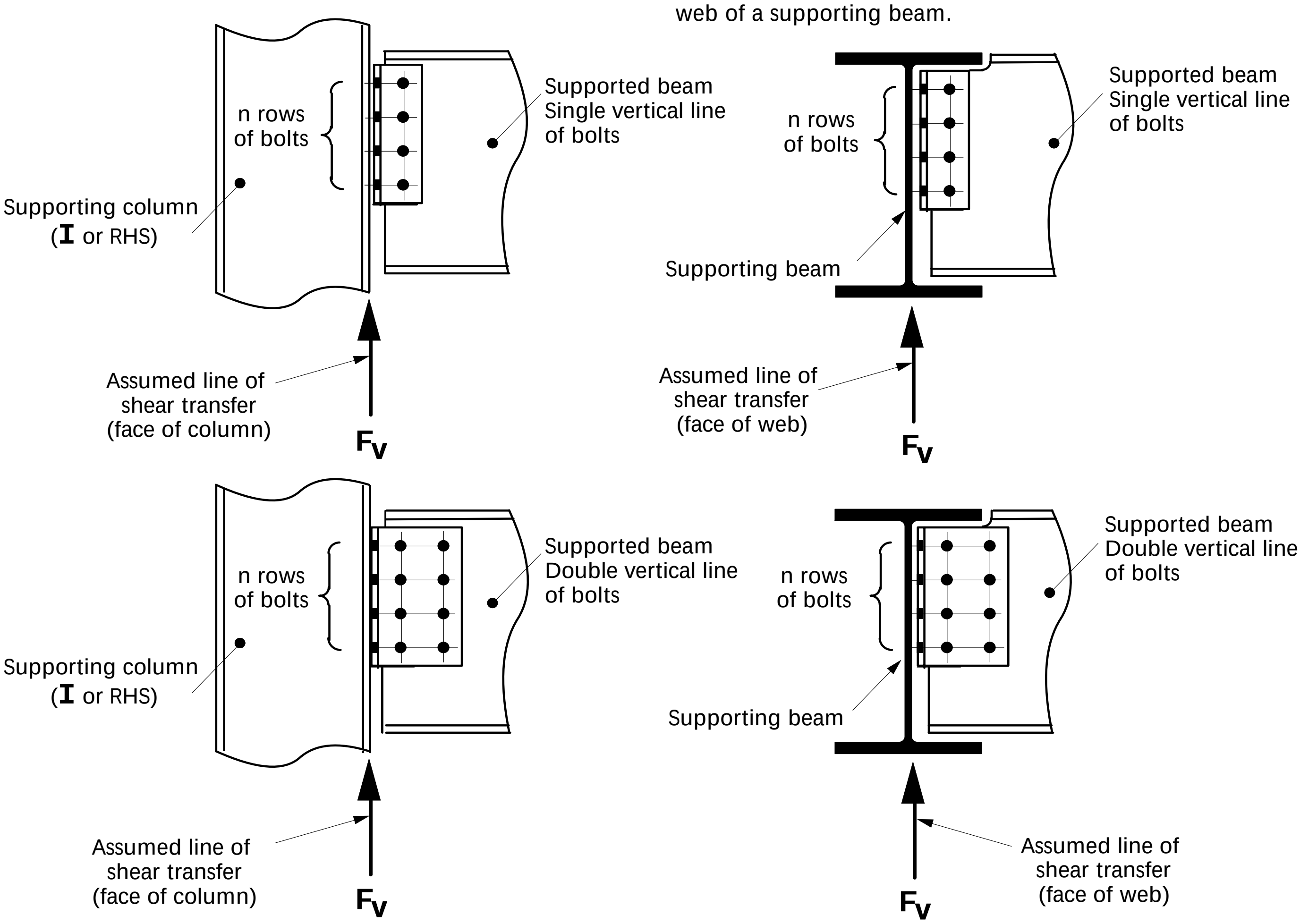
Values of the connection shear and tying capacities are tabulated, together with simple aids to check the supporting member and the beam notch (if applicable). The tying capacities are based on the rigorous method outlined in Appendix B.

4.5 DESIGN PROCEDURES

Recommended design model

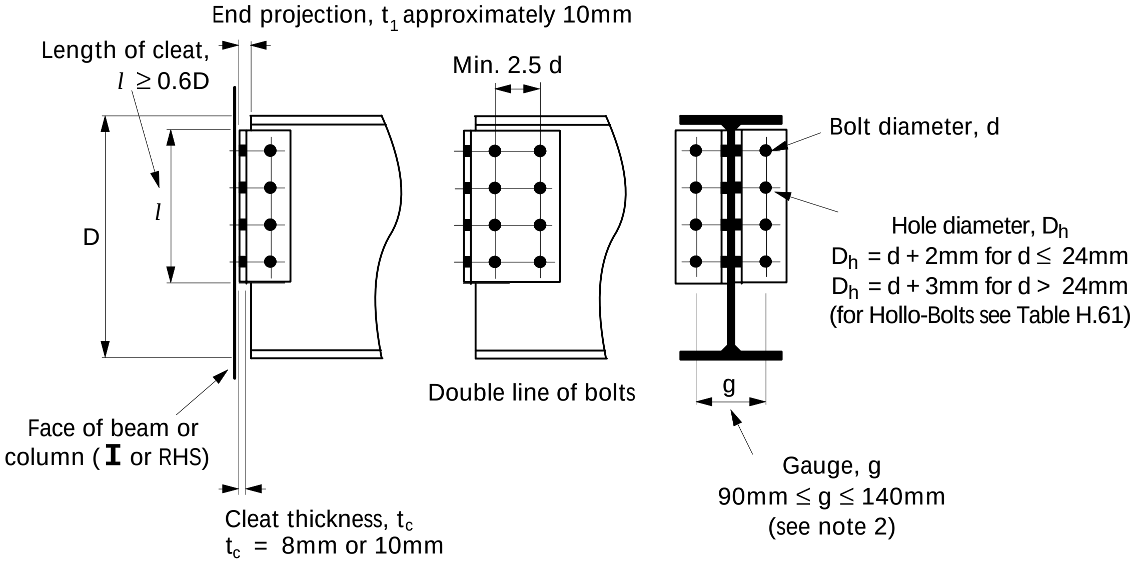
Any simple equilibrium analysis is suitable for design. The one used in this guide is in accordance with traditional UK design practice and does not imply any out of plane bending of the cleat legs.

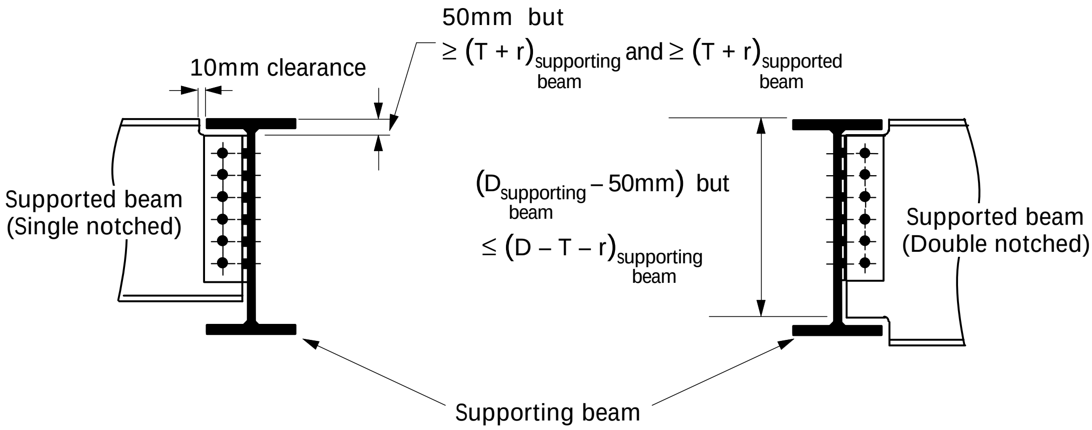
The design model assumes that the angles will deform as the beam ends rotates and the eccentricity moment is resisted by the bolt group on the supported beam. The design procedure applies to beams connected to the flange or web of a column, the wall of a RHS column or the web of a supporting beam.



Check 1	**Recommended detailing practice**		
Check 2	**Supported beam**	–	Bolt group
Check 3	**Supported beam**	–	Connecting elements
Check 4	**Supported beam**	–	Capacity at the connection
Check 5	**Supported beam**	–	Capacity at a notch
Check 6	**Supported beam**	–	Local stability of notched beam
Check 7	**Unrestrained supported beam**	–	Overall stability of notched beam
Check 8	**Supporting beam/column**	–	Bolt group
Check 9	**Supporting beam/column**	–	Connecting elements
Check 10	**Supporting beam/column**	–	Local capacity
Check 11	**Structural integrity**	–	Connecting elements
Check 12	**Structural integrity**	–	Supported beam
Check 13	**Structural integrity**	–	Tension bolt group
Check 14	**Structural integrity**	–	Supporting column web (UC or UB)
Check 15	**Structural integrity**	–	Supporting column wall (RHS)
Check 16	*Not applicable*		

CHECK 1	Recommended detailing practice





Notes

(1) The angle cleats are generally positioned close to the top flange of the beam to provide adequate positional restraint. Cleat length of at least 0.6D is usually adopted to give "nominal torsional restraint" (BS 5950-1[1] Table 13 and clause 4.2.2).

(2) In addition, for connections to RHS columns, g should be at least 0.3 x face width.

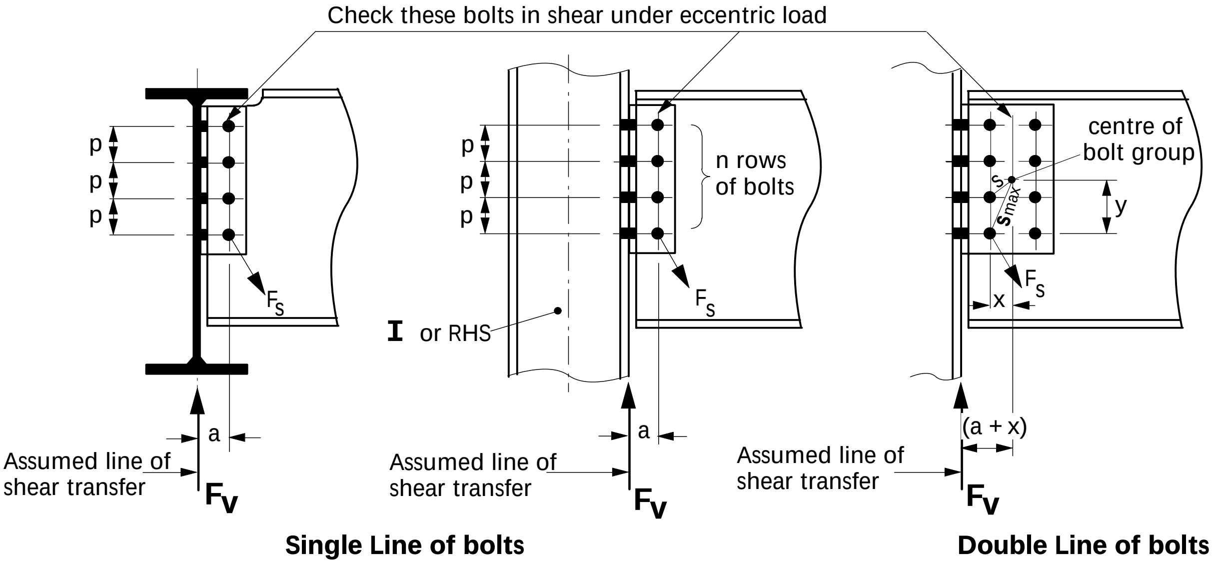
(3) Detail requirements for Flowdrill and Hollo-Bolt connections to RHS columns should also comply with Tables H.60 and H.61 of the yellow pages.

(4) Bolt spacing and edge distances should comply with the recommendations of BS 5950-1: 2000.

In beam-to-**I** section column flange connections, where it is required to comply with structural integrity requirements for a tie force of 75kN, the connection must have at least 2 no. M20, 8.8 bolts in tension with $l \geq 140mm$, $t_c \geq 8mm$ and $g \leq 140mm$.

For greater tie forces and other connections (e.g. to column webs and RHS), checks 11 to 15, as appropriate, should be carried out.

CHECK 2	Supported beam – Bolt group



Shear capacity of bolt group connecting cleats to web of supported beam (taking account of eccentricity 'a' for single line of bolts and (a + x) for double line of bolts.)

Basic requirement:

$F_s \leq 2P_s$

F_s = resultant force on outermost bolt due to direct shear and moment

For single line of bolts

$F_s = \left(F_{sv}^2 + F_{sm}^2\right)^{1/2}$

F_{sv} = vertical force on the bolt due to direct shear $= \dfrac{F_v}{n}$

F_{sm} = force on the outermost bolt due to moment $= \dfrac{F_v\, a}{Z_{bg}}$

Z_{bg} = elastic section modulus of bolt group $= \dfrac{n(n+1)p}{6}$

For double line of bolts

$F_s = \left((F_{sv} + F_{smv})^2 + F_{smh}^2\right)^{1/2}$

F_{sv} = vertical force on the bolt due to direct shear $= \dfrac{F_v}{2n}$

F_{smv} = vertical force on the outermost bolt due to moment $= \dfrac{M\,x}{I_{bg}}$

F_{smh} = horizontal force on the outermost bolt due to moment $= \dfrac{M\,y}{I_{bg}}$

$M = F_v\,(a + x)$

$I_{bg} = \Sigma\, s^2$

where:

p = bolt pitch

s = distance from centre of bolt group to each bolt

s_{max} = distance from centre of bolt group to furthest bolt

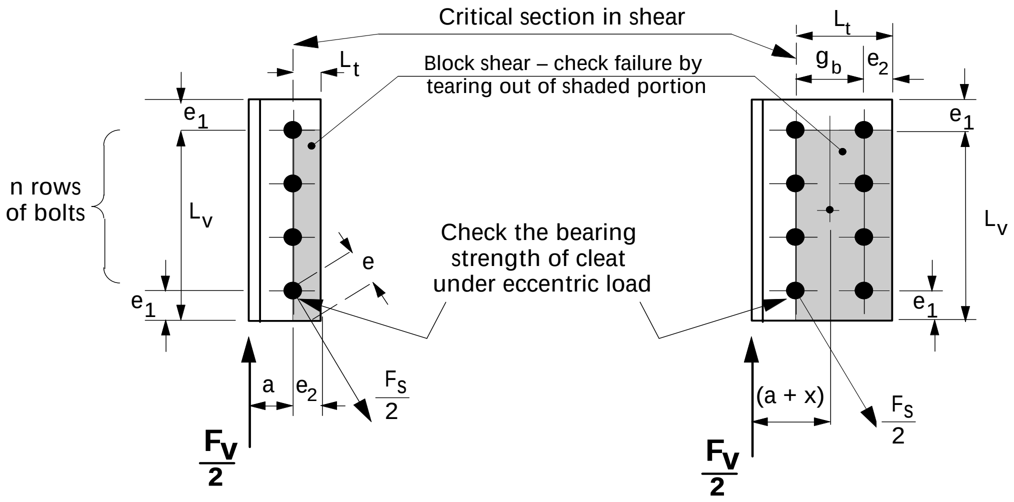
$2P_s$ = shear capacity of a single bolt in double shear
$= 2\, p_s A_s$

p_s = shear strength of a bolt

A_s = shear area of a bolt

I_{bg} = inertia of bolt group

CHECK 3	Supported beam – Connecting elements



Single Line of bolts **Double Line of bolts**

Shear and bearing capacity of cleat connected to supported beam

(i) For shear:

Basic requirement:

$F_v/2 \le P_{v.min}$

$P_{v.min}$ = shear capacity of the leg of the angle cleat

= smaller of Plain shear capacity P_v and Block shear capacity P_r

Plain shear

$P_v = \min\left(0.6\, p_y A_v\,,\ \ 0.7\, p_y K_e A_{v.net}\right)$

$A_v = 0.9\,(2e_1 + (n-1)\,p)\ t_c$

$A_{v.net} = A_v - n\, D_h\, t_c$

Block shear

$P_r = 0.6\, p_y\, t_c\left(L_v + K_e(L_t - kD_h\,)\right)$

$L_v = e_1 + (n-1)p$

$k = 0.5$ and $L_t = e_2$ (for single line of bolts)

$k = 2.5$ and $L_t = e_2 + g_b$ (for double line of bolts)

(ii) For bearing:

Basic requirement:

$F_s/2 \le P_{bs}$

P_{bs} = bearing capacity of the leg of the angle cleat per bolt

$= d\, t_c\, p_{bs}$ but $P_{bs} \le 0.5\, e\, t_c\, p_{bs}$

where:

K_e = 1.2 for S275 steel

= 1.1 for S355 steel

p = bolt pitch

d = diameter of bolt

D_h = diameter of hole

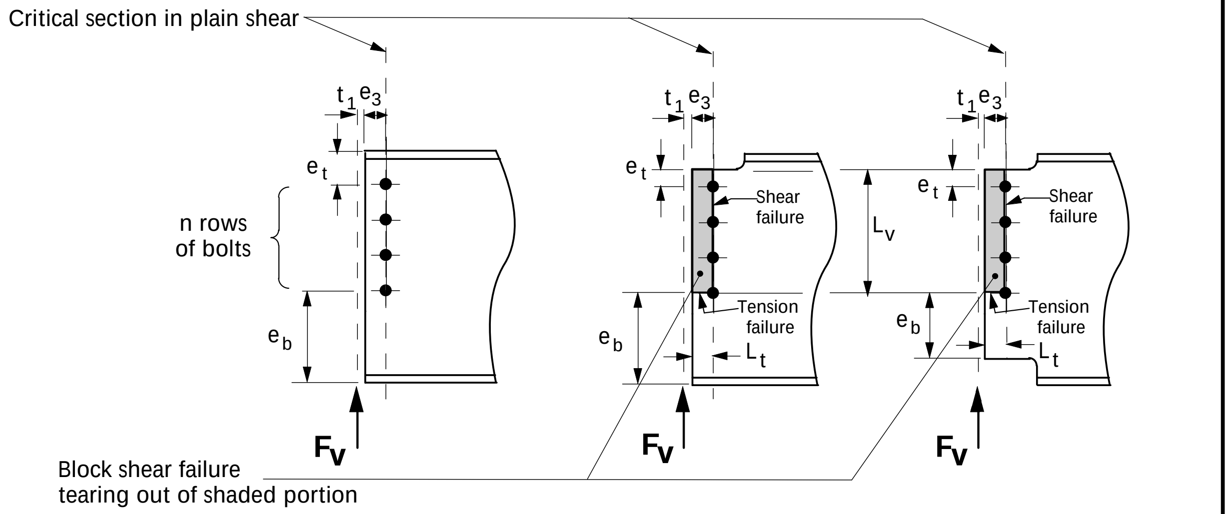
t_c = thickness of cleat

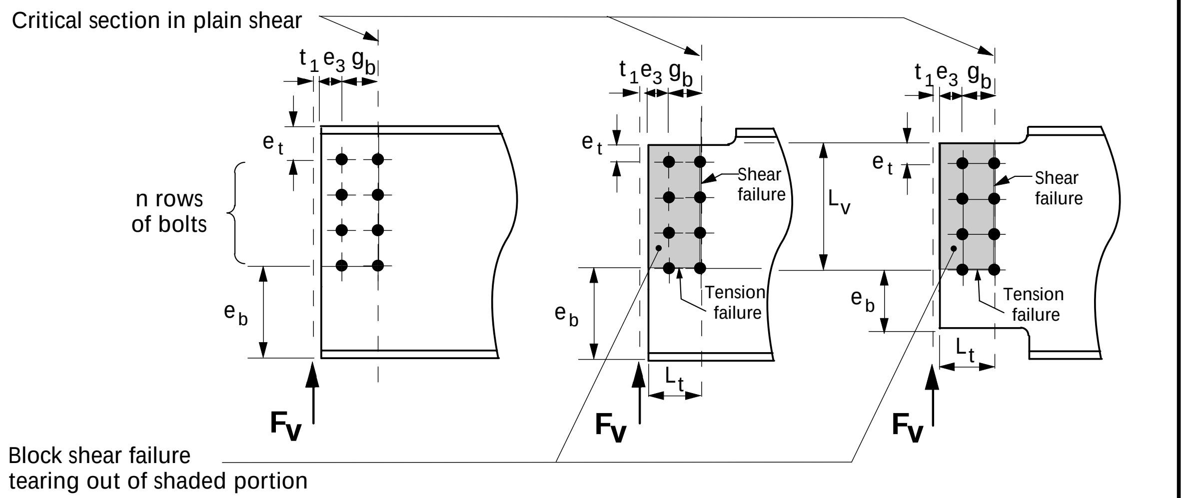
p_{bs} = bearing strength of cleat

e = end distance, and may conservatively be taken as smaller of e_1 and e_2

F_s is defined in CHECK 2

CHECK 4	Supported beam – Capacity at the connection





Shear, bending and bearing capacity of the supported beam:

(i) For shear:

Basic requirement:

$F_v \leq P_{v.min}$

$P_{v.min}$ = shear capacity of the beam at the connection

= smaller of:
Plain shear capacity P_v and
Block shear capacity P_r

Plain shear

$P_v = \min\left(0.6\, p_y A_v\,,\; 0.7\, p_y K_e A_{v.net}\right)$

$A_v = \left(e_t + (n-1)\,p + e_b\right) t_w$
(for un-notched & Single notched beam)

$A_v = 0.9\left(e_t + (n-1)\,p + e_b\right) t_w$
(for double notched beam)

$A_{v.net} = A_v - n\, D_h\, t_w$

Block shear (applicable to notched beams only)

$P_r = 0.6\, p_y\, t_w\left(L_v + K_e(L_t - kD_h)\right)$

$L_v = e_t + (n-1)p$

$k = 0.5$ and $L_t = e_3$ (for single line of bolts)

$k = 2.5$ and $L_t = e_3 + g_b$ (for double line of bolts)

where:

K_e = 1.2 for S275 steel
= 1.1 for S355 steel

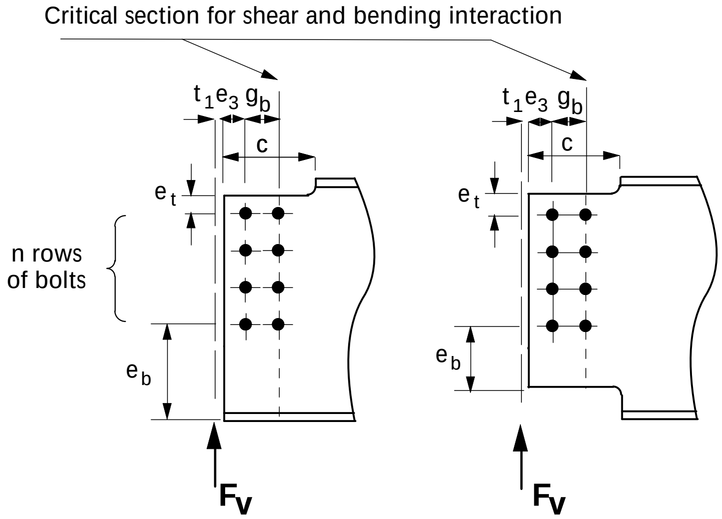
p = bolt pitch

D_h = diameter of hole

t_w = thickness of supported beam web

CHECK 4 continued	Supported beam – Capacity at the connection

Note: *If the notch length c is greater than ($e_3 + g_b$), then shear and bending interaction should be checked at the 2nd line of bolts. The beam at the end of the notch may also be critical - see Check 5.*



(ii) Shear and Bending interaction at the 2nd line of bolts, if the notch length c > ($e_3 + g_b$):

Basic requirement:

$$F_v\,(t_1 + e_3 + g_b) \leq M_{cC}$$

For single notched beam:

For low shear (i.e. $F_v \leq 0.75P_{v.min}$)

$$M_{cC} = p_y\,Z$$

For high shear (i.e. $F_v > 0.75P_{v.min}$)

$$M_{cC} = 1.5\,p_y Z \left(1 - \left(\frac{F_v}{P_{v.min}}\right)^2\right)^{1/2}$$

For double notched beam:

For low shear (i.e. $F_v \leq 0.75P_{v.min}$)

$$M_{cC} = \frac{p_y t_w}{6}\,(e_t + (n\text{-}1)p + e_b)^2$$

For high shear (i.e. $F_v > 0.75P_{v.min}$)

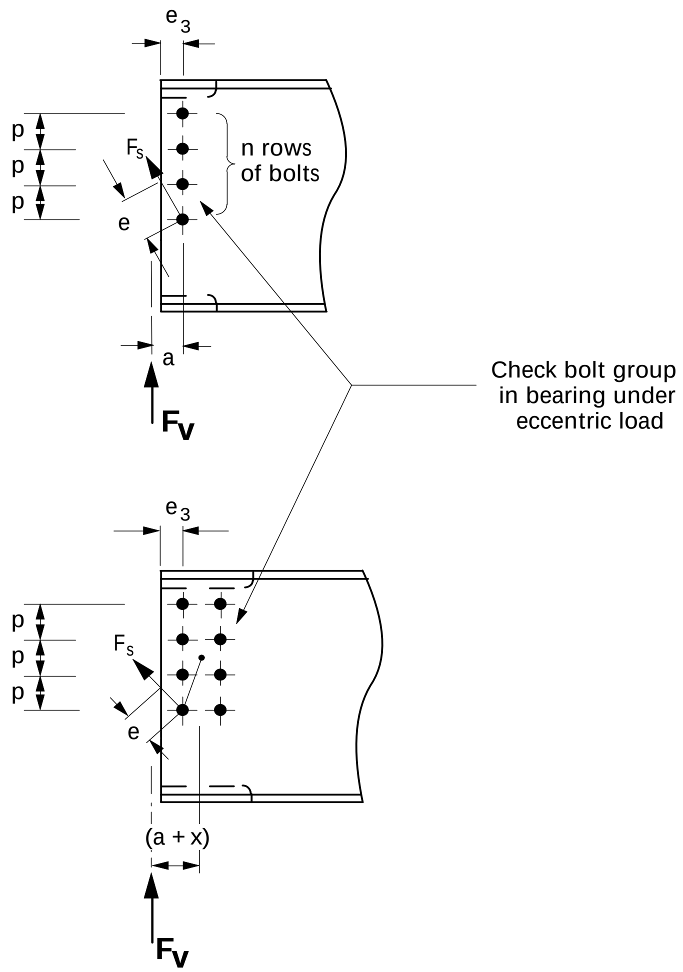
$$M_{cC} = \frac{p_y t_w}{4}\,(e_t + (n-1)p + e_b)^2 \left(1 - \left(\frac{F_v}{P_{v.min}}\right)^2\right)^{1/2}$$

where:

M_{cC} = moment capacity of the notched beam at the connection in the presence of shear.

Z = elastic section modulus of the gross tee section at the bolt line.

CHECK 4 continued	Supported beam – Capacity at the connection



(iii) For bearing:

Basic requirement:

$F_s \quad \leq \quad P_{bs}$

F_s = resultant force as defined in CHECK 2.

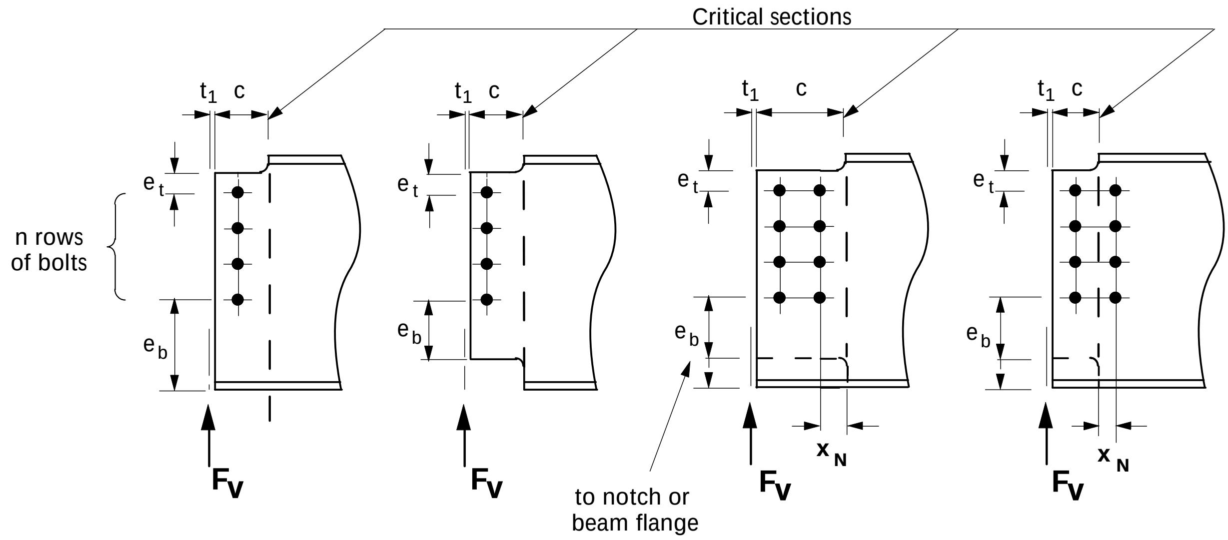
P_{bs} = bearing capacity of the beam web per bolt

= $d\ t_w\ p_{bs}$ but $P_{bs} \leq 0.5\ e\ t_w\ p_{bs}$

p_{bs} = bearing strength of beam web

e = end distance, and may conservatively be taken as e_3

CHECK 5	Supported beam – Capacity at a notch



Shear and bending interaction at the notch:

Basic requirement:

(a) For single bolt line or for double bolt lines, if $x_N \geq 2d$:

$F_v\,(t_1 + c) \quad \leq \quad M_{cN}$

M_{cN} for Single notched beam:

For low shear (i.e. $F_v \leq 0.75P_{vN}$)

$M_{cN} = p_y Z_N$

For high shear (i.e. $F_v > 0.75P_{vN}$)

$$M_{cN} = 1.5 p_y Z_N \left(1 - \left(\frac{F_v}{P_{vN}}\right)^2\right)^{1/2}$$

M_{cN} for Double notched beam:

For low shear (i.e. $F_v \leq 0.75P_{vN}$)

$$M_{cN} = \frac{p_y t_w}{6}\left(e_t + (n-1)p + e_b\right)^2$$

For high shear (i.e. $F_v > 0.75P_{vN}$)

$$M_{cN} = \frac{p_y t_w}{4}\left(e_t + (n-1)p + e_b\right)^2\left(1 - \left(\frac{F_v}{P_{vN}}\right)^2\right)^{1/2}$$

(b) For double bolt lines, if $x_N < 2d$:

$\max\left(F_v(t_1+c),\ F_v(t_1+e_3+g_b)\right) \leq M_{cN}$

$M_{cN} = M_{cC}$ from CHECK 4

where:

M_{cN} = moment capacity of the beam at the notch in the presence of shear

P_{vN} = shear capacity at the notch
= $0.6\, p_y A_{vN}$

A_{vN} = $\left(e_t + (n-1)\,p + e_b\right)t_w$ (for single notched beam)
= $0.9\left(e_t + (n-1)\,p + e_b\right)t_w$ (for double notch beam)

t_1 = end projection

t_w = thickness of supported beam web

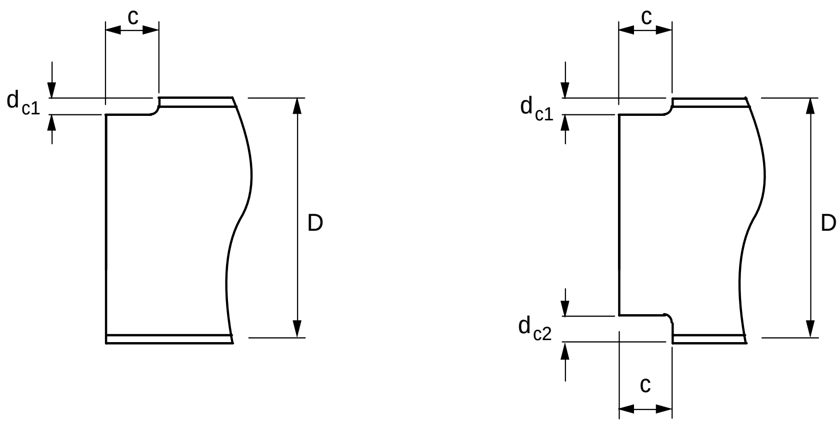
c = length of notch

Z_N = elastic section modulus of the gross tee section at the notch

x_N = +ve

e_3 , g_b as per CHECK 4

CHECK 6	Supported beam - Local stability of notched beam



When the beam is <u>restrained</u> against lateral torsional buckling, no account need be taken of notch stability provided the following conditions are met:

For one flange notched [14],[15]

Basic requirement:

$d_{c1} \leq D/2$ **and:**

c	$\leq$	D	for	$D/t_w \leq 54.3$	(S275 steel)
c	$\leq$	$\dfrac{160000D}{(D/t_w)^3}$	for	$D/t_w > 54.3$	(S275 steel)
c	$\leq$	D	for	$D/t_w \leq 48.0$	(S355 steel)
c	$\leq$	$\dfrac{110000D}{(D/t_w)^3}$	for	$D/t_w > 48.0$	(S355 steel)

For both flanges notched [15]

Basic requirement:

$\text{Max}(d_{c1}, d_{c2}) \leq D/5$ **and:**

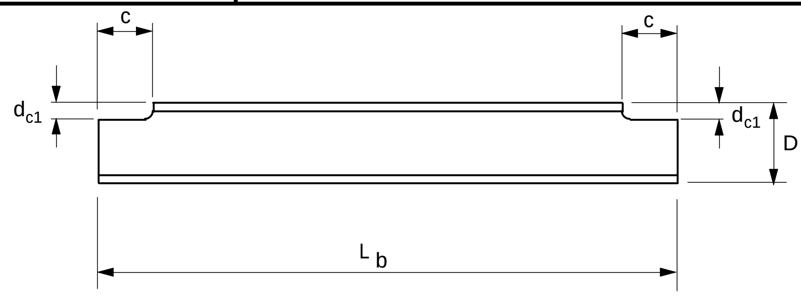
c	$\leq$	D	for	$D/t_w \leq 54.3$	(S275 steel)
c	$\leq$	$\dfrac{160000D}{(D/t_w)^3}$	for	$D/t_w > 54.3$	(S275 steel)
c	$\leq$	D	for	$D/t_w \leq 48.0$	(S355 steel)
c	$\leq$	$\dfrac{110000D}{(D/t_w)^3}$	for	$D/t_w > 48.0$	(S355 steel)

where:

t_w = thickness of supported beam web

Where the notch length c exceeds these limits, either suitable stiffening should be provided or the notch should be checked to references 14, 15 and 16.

CHECK 7	Unrestrained supported beam Overall stability of notched beam



When a notched beam is <u>unrestrained</u> against lateral torsional buckling, the overall stability of the beam should be checked.

Notes:

(1) This check is only applicable for beams with one flange notched. Guidance on double-notched beams is given in Section 5.12 of Reference 17.

(2) If the notch length c and/or notch depth d_{c1} are different at each end, then the larger values for c and d_{c1} should be used.

(3) Beams should be checked for lateral torsional buckling to BS 5950-1[1], clause 4.3 with a modified effective length (L_E) which takes account of notches.

(4) The solution below gives the modified effective length (L_E) based on references 18, 19 and 20. It is only valid for $c/L_b < 0.15$ and $d_{c1}/D < 0.2$ (beams with notches outside these limits should be checked as tee sections, or stiffened).

$$L_E = L_b \left(1 + \frac{2c}{L_b}(K^2 + 2K)\right)^{1/2}$$

$$K = K_o/\lambda_b$$

$$\lambda_b = \frac{u\,v\,L_b}{r_y}$$

where: x, u, v and r_y are for the un-notched **I** beam section and are defined in BS 5950-1
Conservatively, $u = 0.9$ and $v = 1.0$

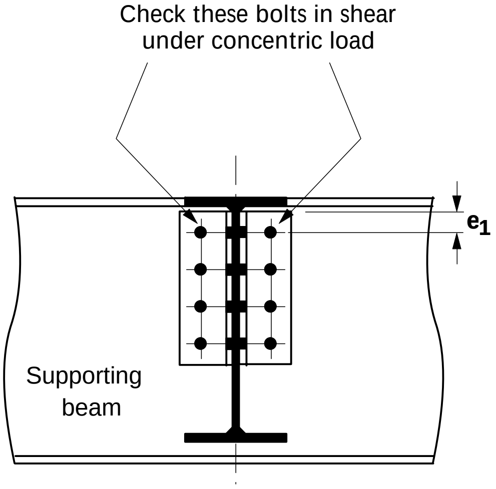
for $\lambda_b < 30$ $\quad K_o = 1.1\,g_o\,x$ but $\leq 1.1\,K_{max}$

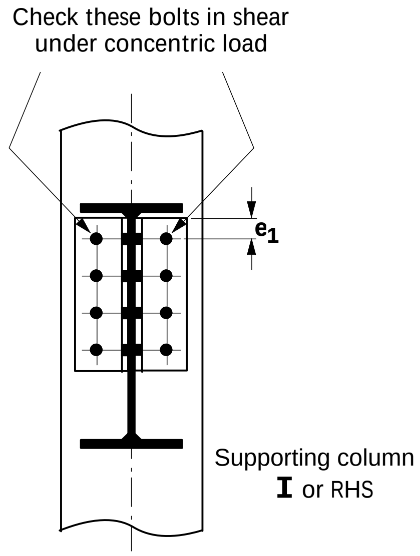
for $\lambda_b \geq 30$ $\quad K_o = g_o\,x$ but $\leq K_{max}$

g_o and K_{max} are tabulated below:

$\frac{c}{L_b}$	g_o	K_{max}	
		UB section	UC section
≤ .025	5.56	260	70
.050	5.88	280	80
.075	6.19	290	90
.100	6.50	300	95
.125	6.81	305	95
.150	7.13	315	100

CHECK 8	Supporting beam/column - Bolt group





Shear capacity of bolt group connecting cleats to supporting beam or column

Basic requirement:

$F_v \leq \Sigma P_s$

P_s = shear capacity of single bolt

= $p_s A_s$*

but for the top pair of bolts, P_s is the smaller of:

$p_s A_s$* or $0.5\, k_{bs}\, e_1\, t_c\, p_{bs}$

where:

p_s = shear strength of a bolt

A_s = shear area of a bolt

t_c = thickness of cleat

p_{bs} = bearing strength of cleat

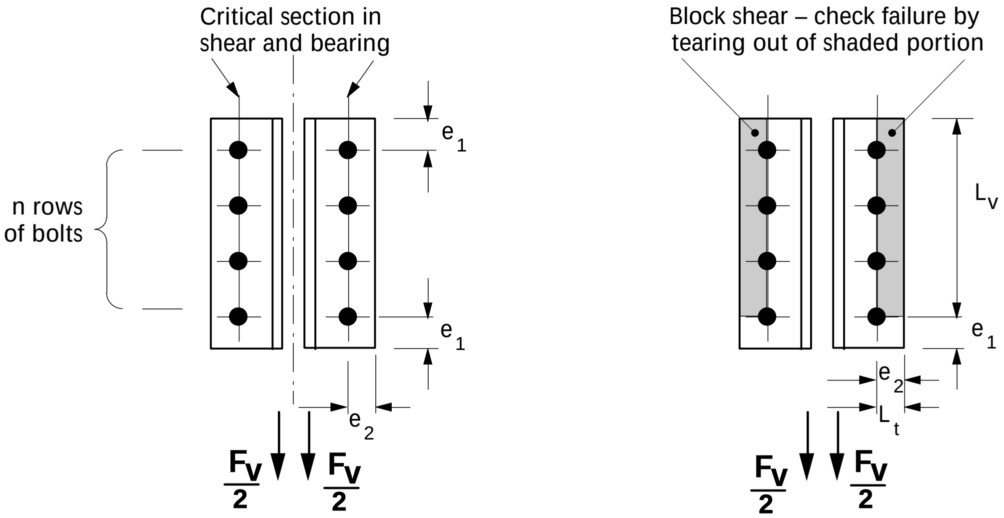
e_1 = end distance

k_{bs} = 1.0 for standard clearance holes, Flowdrill and Hollo-Bolts

= 0.7 for short slotted holes

* For Hollo-Bolts $p_s A_s$ should be taken as the shear capacity given in Table H.56 of the yellow pages

CHECK 9	Supporting beam/column- Connecting elements **(Legs of cleats adjacent to supporting beam or column)**



Shear and bearing capacity of cleats connected to supporting beam or column

(i) For shear:

Basic requirement:

$F_V/2 \leq P_{v.min}$

$P_{v.min}$ = shear capacity of the leg of the angle cleat

= smaller of Plain shear capacity P_v and Block shear capacity P_r

Plain shear

$P_v = \min(0.6\, p_y A_v,\ 0.7\, p_y K_e A_{v.net})$

$A_v = 0.9\,(2e_1 + (n-1)\,p)\, t_c$

$A_{v.net} = A_v - n\, D_h\, t_c$

Block shear

$P_r = 0.6\, p_y\, t_c(L_v + K_e(L_t - kD_h))$

$L_v = e_1 + (n-1)p$

(ii) For bearing:

Basic requirement:

$F_V/2 \leq \Sigma P_{bs}$

ΣP_{bs} = bearing capacity of the leg of the single angle cleat (ie. for 'n' bolts)

P_{bs} = bearing capacity of the leg of the angle cleat per bolt

$= k_{bs} d\, t_c\, p_{bs}$

but for the top bolt, P_{bs} is the smaller of:

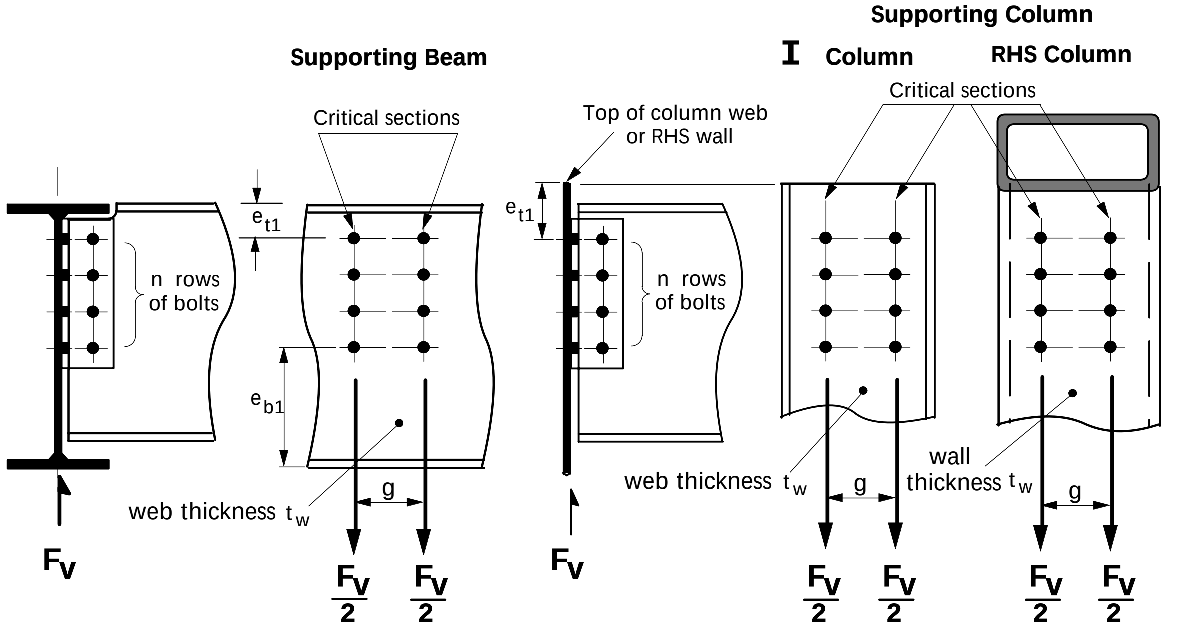
$k_{bs} d\, t_c\, p_{bs}$ or $0.5\, k_{bs}\, e_1\, t_c\, p_{bs}$

where:

p	=	bolt pitch
d	=	diameter of bolt *
D_h	=	diameter of hole *
t_c	=	thickness of cleat
p_{bs}	=	bearing strength of cleat
e_1	=	end distance
k	=	0.5 and $L_t = e_2$ (for single line of bolts in the outstanding leg)
K_e	=	1.2 for S275 steel
	=	1.1 for S355 steel
k_{bs}	=	1.0 for standard clearence holes, Flowdrill and Hollo-Bolts
	=	0.7 for short slotted holes

* For Hollo-Bolts d is the nominal bolt diameter but D_h is the hole diameter in the cleat as given in Table H.61 of the yellow pages.

CHECK 10	Supporting beam/column - Local capacity **(with one supported beam)**



Local shear and bearing capacity of supporting beam web or column web or RHS wall for one supported beam

(i) For shear:

Basic requirement:

$F_v/2 \leq P_v$

P_v = local shear capacity of supporting beam web or I column web or RHS column wall

$P_v = \min(0.6\, p_y A_v\, ,\ 0.7\, p_y K_e A_{v.net})$

$A_v = (e_t + (n-1)\, p + e_b)\, t_w$

$A_{v.net} = A_v - n\, D_h\, t_w$

(ii) For bearing:

Basic requirement:

$\frac{F_v}{2\,n} \leq P_{bs}$

P_{bs} = bearing capacity of supporting beam or column per bolt

$= k_{bs} d\, t_w\, p_{bs}$

p_{bs} = bearing strength of supporting beam or column

where:

e_t = smaller of e_{t1} and 5d

e_b = smallest of e_{b1}, g/2 and 5d (for supporting beam)

= smaller of g/2 and 5d (for supporting column)

p = bolt pitch

d = diameter of bolt *

D_h = diameter of hole *

t_w = thickness of supporting beam web or column web or RHS wall

k_{bs} = 1.0 for standard clearence holes, Flowdrill and Hollo-Bolts

K_e = 1.2 for S275 steel

1.1 for S355 steel

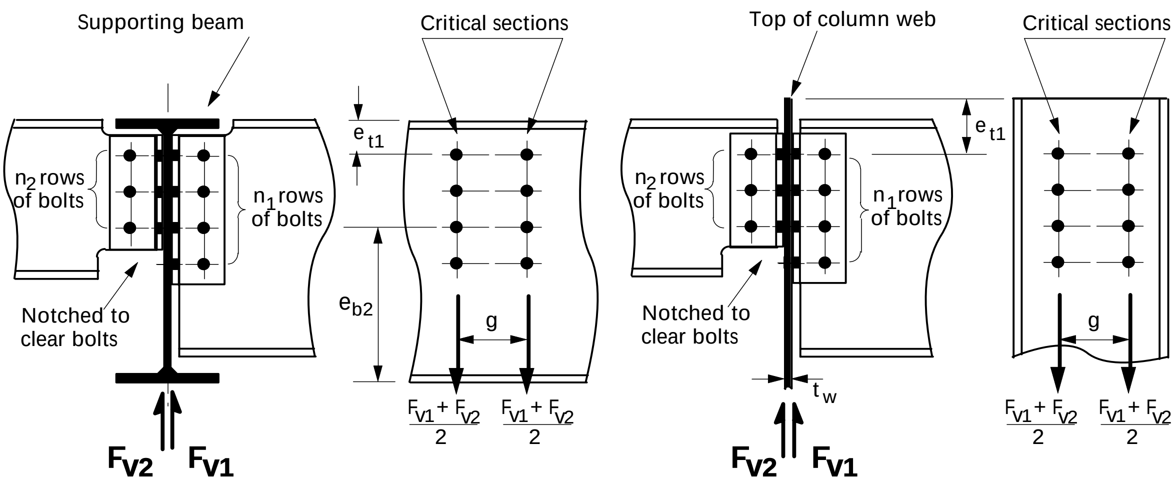
* For Hollo-Bolts d is the nominal bolt diameter but D_h is the hole diameter in the cleat as given in Table H.61 of the yellow pages.

For Flowdrill the diameter of the hole is the bolt diameter.

Note: The above check (i) is for local shear only; the effects of any global shear forces must also be considered.
If the beam is connected to a rolled column flange, and the thickness of the column flange is less than the thickness of the cleat then the bearing capacity of the flange should also be checked.

CHECK 10 (continued)	Supporting beam/column - Local capacity **(with two supported beams)**

Supporting Beam **Supporting I Column**



Local shear and bearing capacity of supporting beam web or column web for two supported beams

(i) For shear:

Basic requirement:

$$\frac{F_{v1A}}{2} + \frac{F_{v2}}{2} \leq P_v$$

$F_{v1A} = F_{v1}\frac{n_2}{n_1}$

P_v = local shear capacity of supporting beam web or column web

$P_v = \min (0.6\, p_y A_v\,,\ 0.7\, p_y K_e A_{v.net})$

$A_v = \left(e_t + (n_2 - 1)\, p + e_b\right) t_w$

$A_{v.net} = A_v - n_2 D_h t_w$

(ii) For bearing:

Basic requirement:

$$\frac{F_{v1}}{2n_1} + \frac{F_{v2}}{2n_2} \leq P_{bs}$$

P_{bs} = bearing capacity of supporting beam or column per bolt

$= d\, t_w\, p_{bs}$

p_{bs} = bearing strength of supporting beam or column

where:

e_t = smaller of e_{t1} and 5d

e_b = smallest of $e_{b2,}$ g/2, p and 5d (for supporting beam)

= smallest of g/2, p and 5d (for supporting column)

p = bolt pitch

d = diameter of bolt

D_h = diameter of hole

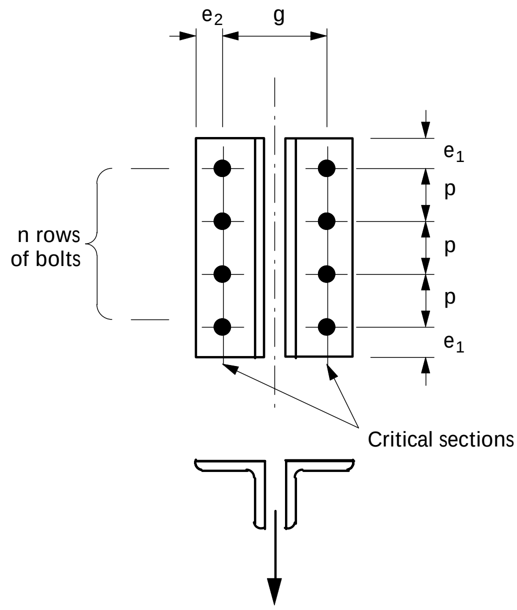
t_w = thickness of supporting beam web or column web

K_e = 1.2 for S275 steel

= 1.1 for S355 steel

Note: The above check (i) is for local shear only; the effects of any global shear forces must also be considered.

CHECK 11	Structural integrity - connecting elements



Note: This check is only needed if it is necessary to comply with structural integrity requirements

Structural integrity – tension capacity of double angle web cleats

Basic requirement:

Tie force $\leq$ Tying capacity of double angle web cleats

Tying capacity of double angle web cleats

$= 0.6\, L_e\, t_c\, p_y$ for S 275 steel (see Appendix B)

$= 0.5\, L_e\, t_c\, p_y$ for S 355 steel (see Appendix B)

Limitations:

$g \leq 140\text{mm}$

$t_c \geq 8\text{mm}$

where:

L_e = effective net length

$= 2e_e + (n - 1)\, p_e - n\, D_h$

$e_e = e_1$ but $\leq e_2$

$p_e = p$ but $\leq 2e_2$

p = bolt pitch

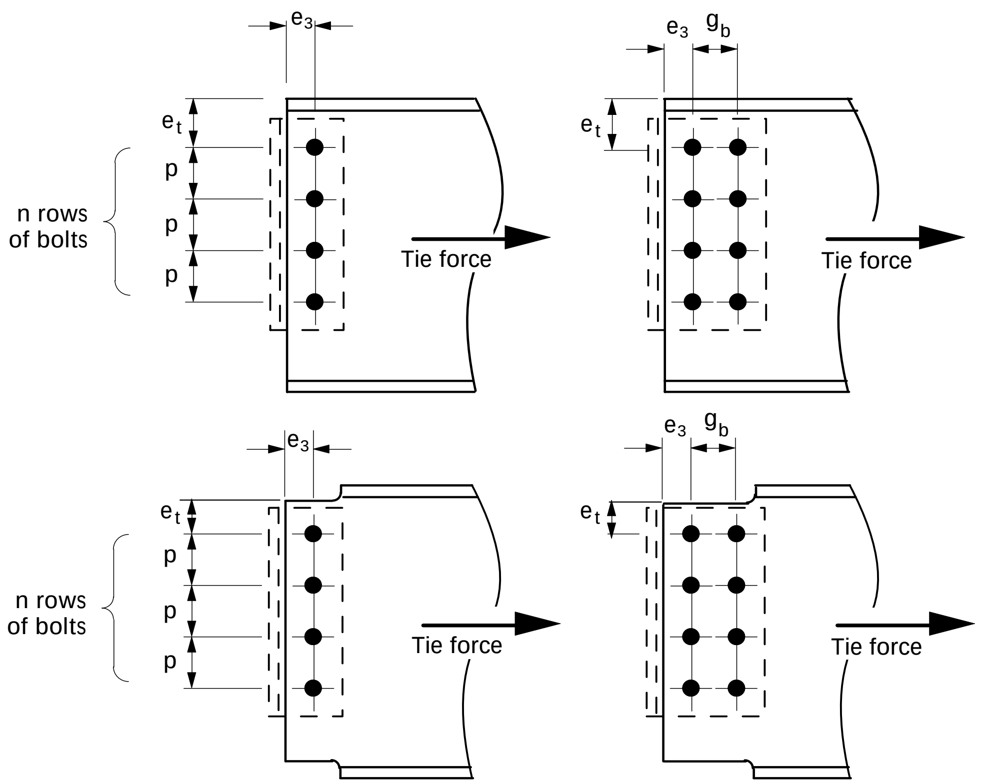
D_h = diameter of hole *

t_c = thickness of cleat

* For Hollo-Bolts D_h is the hole diameter in the cleat as given in Table H.61 of the yellow pages.

Note: Appendix B, gives a rigorous approach for calculating the tension capacity of double angle cleats. The capacity tables on the yellow pages are based on the rigorous approach and not the simplified approach given here.

CHECK 12	Structural integrity - supported beam



Note: This check is only needed if it is necessary to comply with structural integrity requirements

Structural integrity – tension and bearing capacity of beam web

(i) For tension

Basic requirement:

Tie force $\leq$ Net tension capacity of beam web

Net tension capacity of beam web

$= L_e\, t_w\, p_y$

(ii) For bearing

Basic requirement:

Tie force $\leq$ Bearing capacity of beam web

Bearing capacity of beam web

$= 1.5n\, d\, t_w\, p_{bs}$ but

$\leq 0.5ne_3 t_w p_{bs}$

for single line of bolts

$= 3n\, d\, t_w\, p_{bs}$ but

$\leq n\left(1.5d\, t_w\, p_{bs} + 0.5\, e_3\, t_w p_{bs}\right)$

for double line of bolts

where:

L_e = effective net length

$= 2e_e + (n - 1)p_e - nD_h$

$e_e = e_3$ but $\leq e_t$ for single line of bolts

$e_e = e_3 + g_b - D_h$ but $\leq e_t$ for double line of bolts

$p_e = p$ but $\leq 2e_3$ for single line of bolts

$p_e = p$ but $\leq 2(e_3 + g_b - D_h)$ for double line of bolts

t_w = beam web thickness

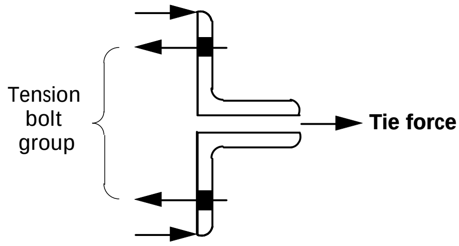
p = bolt pitch

D_h = diameter of hole

d = diameter of bolt

p_{bs} = bearing strength of beam web (BS 5950–1, Table 32)

CHECK 13	Structural integrity – Tension bolt group



Note: This check is only needed if it is necessary to comply with structural integrity requirements

Structural integrity – tension capacity of bolts in presence of extreme prying

Basic requirement:

Tie force $\leq$ Tension capacity of tension bolt group

Tension capacity of tension bolt group = $2n\ A_t\ p_{tr}$ *

where:

n = number of rows of bolts

A_t = tensile stress area of a bolt

p_{tr} = reduced tension strength of a bolt in presence of extreme prying

= 300N/mm^2 for grade 8.8 bolts (see Appendix D)

* See Note (3) for Flowdrill or Hollo-Bolts

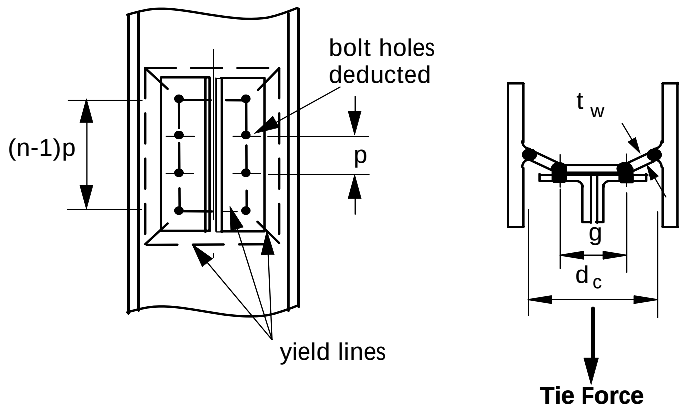
Notes:

(1) The reduced tension strength, (p_{tr}) is only used when double angle web cleat design for structural integrity is based on Appendix B or CHECK 11.

(2) Where a beam is attached to one side of a column web without a beam on the opposite side, or to RHS column, the bolt tensions have to be resisted by local bending of the web or RHS wall. UC webs can resist 75kN but need to be checked if the tying force is higher. UB webs need to be checked for 75kN and higher tying forces. CHECK 14 proposes a design model which could be used for this purpose. CHECK 15 proposes a design model for checking the wall of RHS column.

(3) For Flowdrill or Hollo-Bolt connections the value $A_t\ p_{tr}$ is replaced by the Structural integrity Tensile Capacity (P_{si}) taken from Tables H.55b and H.56 respectively, in the yellow pages.

CHECK 14	Structural integrity – Supporting column web (UC **or** UB)



Angle cleats connecting to column web

Note: This check is only needed if it is necessary to comply with structural integrity requirements

Structural integrity – Tying capacity of rolled column web, in the presence of axial compression in the column

Basic requirement:

Tie force $\leq$ Tying capacity of column web

$$\text{Tying capacity of column web} = \frac{8\,M_u}{1-\beta_1}\left(\eta_1 + 1.5(1-\beta_1)^{0.5}\,(1-\gamma_1)^{0.5}\right)\ *$$

M_u = moment capacity of column web per unit length

$$= \frac{p_u\, t_w^{\,2}}{4}$$

p_u = design tensile strength of the column

= $U_s\,/1.25$ (see inset box)

* Factor 1.5 in the equation includes an allowance for the axial compression in the column.

where:

$$\eta_1 = \frac{(n\text{-}1)\ p - \frac{n}{2}\ D_h}{d_c}$$

$$\beta_1 = \frac{g}{d_c}$$

$$\gamma_1 = \frac{D_h}{d_c}$$

d_c = depth of column between fillets

t_w = thickness of column web

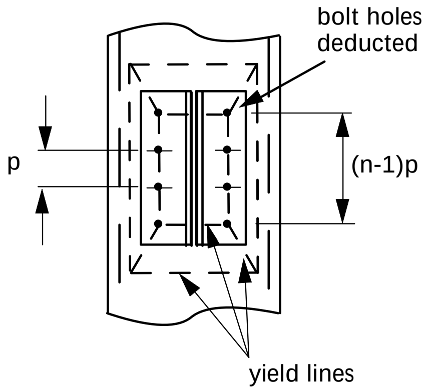
g = gauge (cross centres)

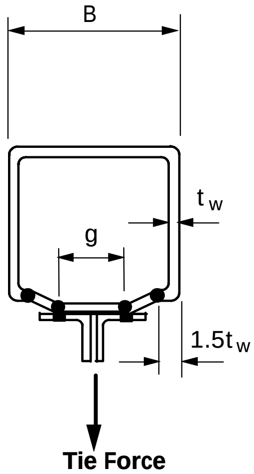
D_h = diameter of hole

Design tensile strength	
	$p_u = U_s\,/1.25$
S275	328 N/mm^2
S355	392 N/mm^2
Values of U_s taken from BS EN 10025: 1993[21]	

Note: The check is required for either single-sided connections to the rolled column web or unequally loaded double-sided connections to the rolled column web.

CHECK 15	Structural integrity – Supporting column wall (RHS)	☐





Angle cleats connecting to RHS column

Note: This check is only needed if it is necessary to comply with structural integrity requirements

Structural integrity – Tying capacity of RHS wall, in the presence of axial compression in the column

Basic requirement:

Tie force $\leq$ Tying capacity of RHS column wall

Tying capacity RHS column wall $= \dfrac{8\,M_u}{1-\beta_1}\left(\eta_1 + 1.5(1-\beta_1)^{0.5}\,(1-\gamma_1)^{0.5}\right)$ *

M_u = moment capacity of RHS column wall per unit length

$= \dfrac{p_u\, t_w^{\,2}}{4}$

p_u = design tensile strength of the RHS column

$= U_s/1.25$ (see inset box)

* Factor 1.5 in the equation includes an allowance for the axial compression in the column.

Design tensile strength	
	$p_u = U_s/1.25$
S275	328 N/mm^2
S355	392 N/mm^2
Values of U_s taken from BS EN 10210 - 1:1994[3]	

where:

$\eta_1 = \dfrac{(n\text{-}1)\;p - \frac{n}{2}\,D_h}{(B - 3t_w)}$

$\beta_1 = \dfrac{g}{(B - 3t_w)}$

$\gamma_1 = \dfrac{D_h}{(B - 3t_w)}$

B = overall width of RHS column wall to which the connection is made

t_w = thickness of RHS column

D_h = diameter of hole in RHS

= bolt diameter for Flowdrill

= hole diameter (in RHS) given in Table H.61 of yellow pages for Hollo-Bolt

g = gauge (cross centres)

n = number of rows of bolts

p = bolt pitch

4.6 WORKED EXAMPLES

The worked examples show design calculations for typical standard connections. Each example demonstrates first the use of the capacity tables (yellow pages) and then full checks according to the procedures in Section 4.5. The full checks will normally only need to be applied to non-standard connections but their application to standard connections demonstrates the validity of the much simpler process when using standard details.

When calculations must be made for non-standard connections, some design checks may be omitted where it is obvious, from inspection of the detail, that a check is not critical. In the case of Example 1, CHECK 3 and CHECK 9 *Block Shear* calculation are not made since they are never a critical factor for well proportioned angle cleats with bolts spaced at reasonable centres in the cleat length. However, if a connection design were made using a long cleat with bolts concentrated at one end of the cleat, it is possible that *Block Shear failure* could occur before *Plain Shear failure* and then the *Block shear* checks should be made.

CHECK 7, dealing with overall stability of an unrestrained beam, should be undertaken by the member designer taking account of any notching required at the ends of the supported beam in order to facilitate the use of a simple connection.

CHECKS 11 to 15 all deal with structural integrity in the presence of an axial tie force required to be developed in some members to ensure the steel frame is sufficiently robust, or in the case of some multi-storey buildings, to localise accidental damage. When tying capacity is not required these checks may be omitted.

Example 1

Example 1 covers design checks for a two sided beam-to-beam connection.

Example 2

Example 2 demonstrates the additional design checks required when a beam-to-column connection must be designed to resist tying forces.

Example 3

Example 3 is a beam connection to an RHS column using normal grade 8.8 bolts in Flowdrill threaded holes to connect double angle web cleats to the column wall. The beam sizes, vertical reactions and tie forces are the same as in Examples 1 and 2, so only checks which are different to the those shown previously are illustrated.

It should be noted that tie forces are ignored in checks for vertical reactions and vertical reactions are ignored in checks for tie forces.

Example 4

Example 4 covers the same beam connections to an RHS column as in Example 3 but uses Hollo-Bolts to connect double angle web cleats to the column wall.

Example 5

Example 5 demonstrates the stability CHECK 7 for a laterally unrestrained beam with a single notch at each end. Although this example is presented here, it is equally applicable to flexible end plates and fin plates.

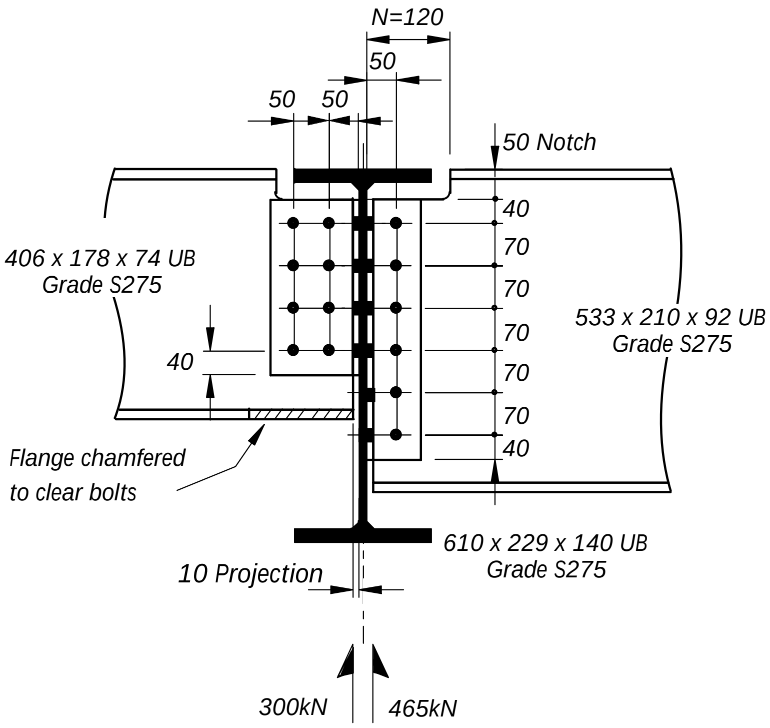
CALCULATION SHEET BCSA	Job No: Joints in Steel Construction - Simple Connections	Sheet: 1 of 18
	Title: Example 1 - Double Angle Cleats - Beam to Beam	
	Client: SCI/BCSA Connections Group	
	Calcs by: RS · Checked by: AM	Date: May 2002

REF.

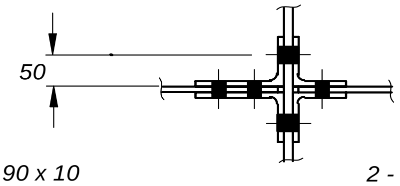
DESIGN EXAMPLE 1

Check the following beam to beam connection for the design forces shown.

Yellow pages used for initial selection of Angle cleats.



See Figure 4.4



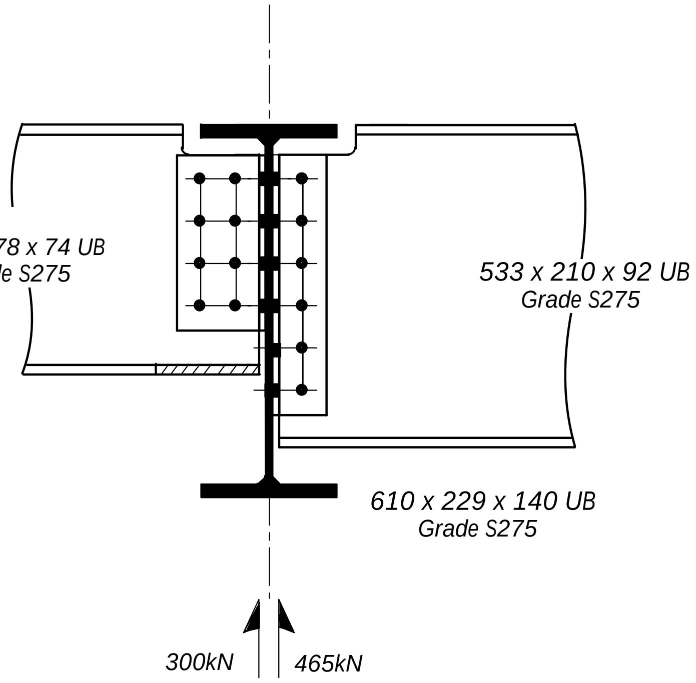
Yellow pages Table H.2

Design Information:

Bolts:	M20 8.8
Material:	All S275

Title	Example 1 - Double Angle Cleats - Beam to Beam	Sheet	2 of 18

CONNECTION DESIGN USING CAPACITY TABLES FROM YELLOW PAGES



406 x 178 x 74 UB Grade S275	533 x 210 x 92 UB Grade S275	
Cleat type CB4 Grade S275	**Cleat type CA6 Grade S275**	
Bolts M20 8.8	**Bolts M20 8.8**	
From capacity table H.10 in Yellow pages	From capacity table H.9 in Yellow pages	Yellow pages Tables H.10 & H.9
Connection shear capacity = 404kN > 300kN	Connection shear capacity = 475kN > 465kN	∴ **O.K.**
Maximum notch length $(c + t_1)$ = 221mm > 120mm	Maximum notch length $(c + t_1)$ = 346mm > 120mm	∴ **O.K.**

Web thickness of supporting beam = 13.1mm — Table H.64

Minimum support thickness = 5.5 + 4.3 = 9.8mm < 13.1mm — ∴ **O.K.**

Connection is adequate

Title	*Example 1 - Double Angle Cleats - Beam to Beam*	Sheet	*3 of 18*

SUMMARY OF FULL DESIGN CHECKS FOR EXAMPLE 1

Note: *Values given are overall capacities unless otherwise noted.*

Sheet Nos	CHECK		406UB (S275)		533UB (S275)		610UB (S275)	
			Capacity	Applied Load	Capacity	Applied Load	Capacity	Applied Load
4	**CHECK 1** - Recommended detailing practice		All recommendations adopted					
4	**CHECK 2** Supported beam - Bolt group shear capacity	(Capacity per bolt, kN)	184	64.9	184	91	Not Applicable	
5 to 7	**CHECK 3** Supported beam - Connecting elements (Strength of cleat)	Shear (kN) Bearing (Capacity per bolt, kN)	400 92	150 32.5	589 92	233 45.5	Not Applicable	
8 to 11	**CHECK 4** Supported Beam - Capacity at connection (notched beam)	Shear (kN) Shear & Bending Interaction (kNm) Bearing (Capacity per bolt, kN)	458 (block shear) 89.1 **87.4**	300 30 **64.9**	708 (block shear) N/A **92.9**	465 N/A **91**	Not Applicable	
			CRITICAL CHECK IN THIS EXAMPLE					
12 & 13	**CHECK 5** Supported beam - Capacity at the notch	Bending capacity (kNm)	89.1	36	164	55.8	Not Applicable	
14	**CHECK 6** Supported beam - Local stablity of notched beam (Beam restrained)	Notch length mm	412.8	110	153.1	110	Not Applicable	
			Notch length (c) < Specified limits					
14	**CHECK 7** LTB of Supported beam (Beam restrained)	-	Not Applicable				Not Applicable	
15	**CHECK 8** Supporting beam - Bolt group shear capacity	(Capacity of bolt group , kN)	735	300	1103	465	Not Applicable	
16 & 17	**CHECK 9** Supporting beam - Connecting elements (Strength of cleat)	Shear (kN) Bearing (Capacity per bolt line, kN)	400 368	150 150	589 552	233 233	Not Applicable	
18	**CHECK 10** Supporting beam - Capacity (Local capacity of beam web)	Shear (kN) Bearing (Capacity per bolt , kN)	Not Applicable				740 121	305 76

Title	Example 1 - Double Angle Cleats - Beam to Beam	Sheet	4 of 18

CHECK 1: Recommended detailing practice

Cleats: 10mm thick

Length, l = 290mm (>0.6D for 406 UB)

= 430mm (>0.6D for 533 UB)

CHECK 2 : Supported beam - Bolt group

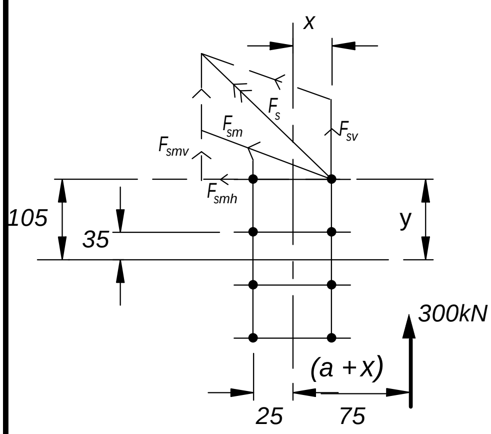
Basic requirement: $F_s \leq 2P_s$

For 406 x 178 x 74 UB

Resultant shear per bolt, $F_s = \left((F_{sv} + F_{smv})^2 + F_{smh}^2\right)^{1/2}$

Direct shear per bolt, $F_{sv} = \dfrac{F_v}{2n} = \dfrac{300}{8} = 37.5\text{kN}$

Eccentric bending moment, $M = F_v(a + x) = 300 \times 0.075 = 22.5\text{kNm}$



Second moment of area of bolt group

$I_{bg} = \Sigma s^2$

$= 4(25^2 + 105^2) + 4(25^2 + 35^2)$

$= 54000\text{mm}^4$

$F_{smh} = \dfrac{M\,y}{I_{bg}} = \dfrac{22.5 \times 105 \times 10^3}{54000} = 43.8\text{kN}$

$F_{smv} = \dfrac{M\,X}{I_{bg}} = \dfrac{22.5 \times 25 \times 10^3}{54000} = 10.4\text{kN}$

$F_s = \left((37.5 + 10.4)^2 + 43.8^2\right)^{1/2} = 64.9\text{kN}$

Double shear capacity of an M20 8.8 bolt, — *Fastener capacities yellow pages Table H.49*

$2P_s = 184\text{kN}$

$\mathbf{F_s = 64.9kN < 184kN}$ — **∴ O.K.**

For 533 x 210 x 92 UB

$F_{sv} = \dfrac{F_v}{n} = \dfrac{465}{6} = 77.5\text{kN}$

$Z_{bg} = \dfrac{n\,(n + l)\,p}{6} = \dfrac{6\,(6 + 1)70}{6} = 490\text{mm}^3$

$M = F_v\,a = 465 \times 0.05 = 23.3\text{kNm}$

$F_{sm} = \dfrac{M}{Z_b} = \dfrac{23.3 \times 10^3}{490} = 47.6\text{kN}$

$F_s = \left(F_{sv}^2 + F_{sm}^2\right)^{1/2} = \left(77.5^2 + 47.6^2\right)^{1/2} = 91\text{kN}$ — *Fastener capacities yellow pages Table H.49*

$2P_s = 184\text{kN}$

$\mathbf{F_s = 91kN < 184kN}$ — **∴ O.K.**

Title	Example 1 - Double Angle Cleats - Beam to Beam	Sheet	5 of 18

CHECK 3 : Supported Beam - Connecting Elements

(i) Basic requirement for shear: **Applied Shear** $\frac{F_v}{2} < P_{v.min}$

For 406 x 178 x 74 UB

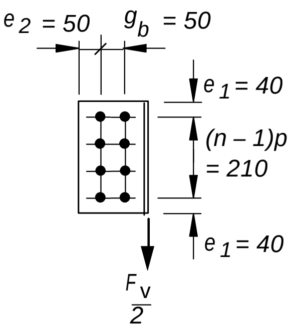
Shear capacity of cleat leg, $P_{v.min}$ is the smaller of Plain shear capacity P_v and Block shear capacity P_r

Plain shear $P_v = \min\left(0.6\, p_y\, A_v \quad \text{or} \quad 0.7\, p_y K_e\, A_{v.net}\right)$

Shear area, $A_v = 0.9\left(2e_1 + (n - l)p\right) t_c$

$= 0.9\left(80 + 210\right) \times 10$

$= 2610\text{mm}^2$



Net area, $A_{v.net} = A_v - n\, D_h\, t_c$

$= 2610 - (4 \times 22 \times 10) = 1730\text{mm}^2$

$0.6 p_y A_v = \dfrac{0.6 \times 275 \times 2610}{10^3} = 431\text{kN}$

$0.7 p_y K_e A_{v.net} = \dfrac{0.7 \times 275 \times 1.2 \times 1730}{10^3} = 400\text{kN}$

$\therefore P_v = 400\text{kN}$

Block shear $P_r = 0.6 p_y\, t_c \left(L_v + K_e \left(L_t - kD_h \right)\right)$

$L_v = e_1 + (n - 1)p = 40 + 210 = 250\text{mm}$

$L_t = e_2 + g_b = 50 + 50 = 100\text{mm}$

$k = 2.5$ (for double line of bolts)

$K_e = 1.2$ (for S275)

$\therefore P_r = \dfrac{0.6 \times 275 \times 10 \left(250 + 1.2(100 - 2.5 \times 22)\right)}{10^3}$

$= 502\text{kN}$

$P_{v.min} = \min (P_v , P_r)$

$= 400\text{kN}$

$F_v/2 = \mathbf{150kN} < \mathbf{400kN}$ **∴ O.K.**

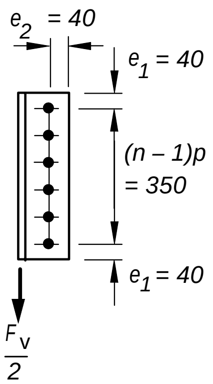
NOTE:

Block shear checks have been shown here, but they are never critical for well proportioned cleats. However, if the bolt spacing is concentrated at one part of a cleat then these checks may be critical.

Title	Example 1 - Double Angle Cleats - Beam to Beam	Sheet	6 of 18

For 533 x 210 x 92 UB

Plain shear



$A_v = 0.9(80 + 350) \times 10 = 3870mm^2$

$A_{v.net} = 3870 - (6 \times 22 \times 10) = 2550mm^2$

$0.6\, p_y.A_v = \frac{0.6 \times 275 \times 3870}{10^3} = 639kN$

$0.7 p_y K_e A_{v.net} = \frac{0.7 \times 275 \times 1.2 \times 2550}{10^3} = 589kN$

$\therefore \; P_v = 589kN$

Block shear $P_r = 0.6p_y\, t_c \left(L_v + K_e \left(L_t - kD_h \right)\right)$

$L_v = e_1 + (n-1)p = 40 + 350 = 390mm$

$L_t = e_2 = 40mm$

$k = 0.5$ (for single line of bolts)

$K_e = 1.2$ (for S275)

$\therefore \; P_r = \frac{0.6 \times 275 \times 10 \left(390 + 1.2(40 - 0.5 \times 22)\right)}{10^3}$

$= 701kN$

$P_{v.min} = \min(P_v , P_r)$

$= 589kN$

$F_v/2$ = **233kN** < **589kN** **∴ O.K.**

NOTE:

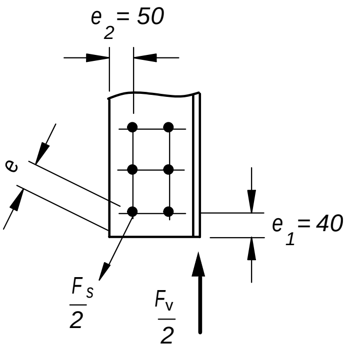
Block shear checks have been shown here, but they are never critical for well proportioned cleats. However, if the bolt spacing is concentrated at one part of a cleat then these checks may be critical.

Title Example 1 - Double Angle Cleats - Beam to Beam	Sheet 7 of 18

(ii) Basic requirement for bearing: $\frac{F_s}{2} \leq P_{bs}$

Bearing capacity, $P_{bs} = d\, t_c\, p_{bs} \leq 0.5\, e\, t_c\, p_{bs}$

For 406 x 178 x 74 UB



e = lesser of e_1 or e_2 (conservatively)

= 40mm

$d\ t_c\, p_{bs} = \dfrac{20 \times 10 \times 460}{10^3} = 92kN$

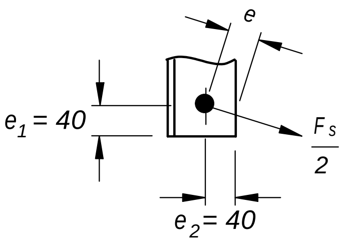
$0.5e\ t_c\, p_{bs} = \dfrac{0.5 \times 40 \times 10 \times 460}{10^3} = 92kN$ — p_{bs} from BS 5950-1 Table 32

$P_{bs} = 92kN$

$\dfrac{F_s}{2} = \dfrac{64.9}{2} = 32.5kN$ — Sheet 4

$\boldsymbol{\dfrac{F_s}{2} = 32.5kN < 92kN}$ — **∴ O.K.**

For 533 x 210 x 92 UB



$P_{bs} = 92kN$ as above

$\dfrac{F_s}{2} = \dfrac{91}{2} = 45.5kN$ — Sheet 4

$\boldsymbol{\dfrac{F_s}{2} = 45.5kN < 92kN}$ — **∴ O.K.**

Title	Sheet
Example 1 - Double Angle Cleats - Beam to Beam	*8 of 18*

CHECK 4: Supported Beam - Capacity at the connection

(i) Basic requirement for shear: F_v $<$ $P_{v.min}$

where $P_{v.min}$ is the smaller of the plain shear capacity, P_v or the block shear capacity, P_r of the supported beam

For 406 x 178 x 74 UB Grade S275

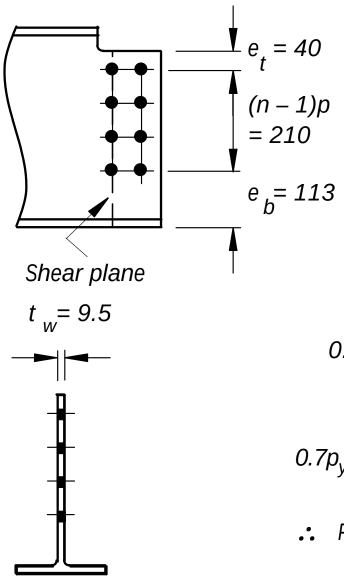
Plain shear capacity

Bolt diameter,

d = 20mm

Hole diameter

D_h = 22mm



$P_v = \min\left(0.6\, p_y A_v\, ,\ 0.7\, p_y K_e A_{v.net}\right)$

Shear area,

$A_v = \left(e_t + (n-1)\,p + e_b\right) t_w$ *(for Single notched beam)*

$= \left(40 + 210 + 113\right) \times 9.5 = 3449mm^2$

Net shear area,

$A_{v.net} = A_v - n\, D_h\, t_w$

$= 3449 - (4 \times 22 \times 9.5) = 2613mm^2$

$0.6\, p_y\, A_v = \dfrac{0.6 \times 275 \times 3449}{10^3} = 569kN$

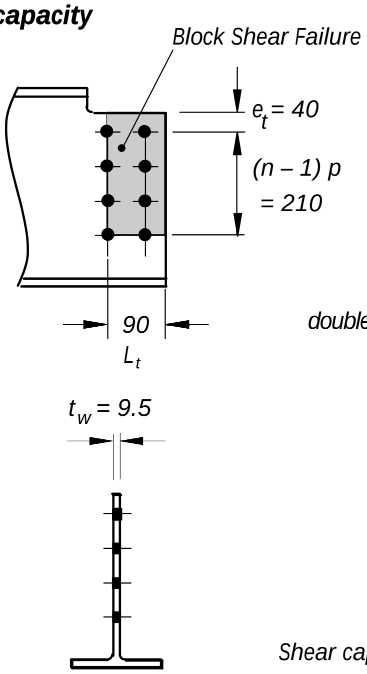
$0.7 p_y K_e A_{v.net} = \dfrac{0.7 \times 275 \times 1.2 \times 2613}{10^3} = 604kN$

$\therefore\ P_v = 569kN$

Block shear capacity

Bolt diameter
d = 20mm

Hole diameter,
D_h = 22mm



$P_r = 0.6\, p_y\, t_w\left(L_v + K_e(L_t - kD_h)\right)$

$L_v = e_t + (n-1)\,p$

$= 40 + 210$

$= 250mm$

double line of bolts

$\therefore\ k = 2.5$

$kD_h = 2.5 \times 22 = 55mm$

$\therefore\ P_r = \dfrac{0.6 \times 275 \times 9.5\left(250 + 1.2(90 - 55)\right)}{10^3}$

$= 458kN$

Shear capacity of supported beam, $P_{v.min}$

$P_{v.min} = \min\left(P_v\, ,\ P_r\right)$

$\therefore P_{v.min} = P_r = 458kN$

Applied shear, F_v **= 300kN < 458kN** **∴ O.K.**

Title Example 1 - Double Angle Cleats - Beam to Beam	Sheet 9 of 18

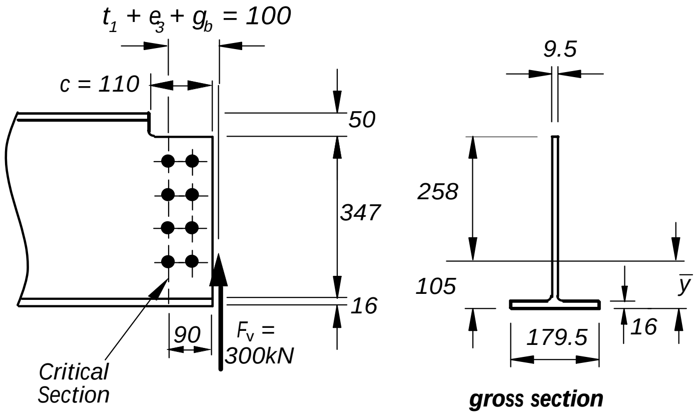
(ii) Shear and bending interaction at the second bolt line is required if $c > e_3 + g_b$

For 406 x 178 x 74UB Grade S275

$c = 110mm > 90mm$

$\therefore$ interaction check required.

For low shear condition: $F_v \leq 0.75\, P_{v.min}$



$0.75\, P_{v.min} = 0.75 \times 458 = 344kN$

$300kN < 344kN$ $\therefore$ low shear condition applies

$P_{v.min}$ from sheet 8

Basic requirement:

$$F_v\,(t_1 + e_3 + g_b) \leq M_{cC}$$

$$F_v\,(t_1 + e_3 + g_b) = \frac{300 \times 100}{10^3} = 30kNm$$

For gross tee section:

Taking moments of area about bottom flange:

$$(179.5 \times 16 \times 8) + (347 \times 9.5 \times (16 + 347/2)) = ((179.5 \times 16) + (347 \times 9.5)) \times \bar{y}$$

$$22{,}976 + 624{,}687 = 6{,}169 \times \bar{y}$$

$$\bar{y} = 105mm$$

Second moment of area about neutral axis:

$$I_{xx} = \frac{1}{10^4}\left(\frac{179.5 \times 16^3}{12} + (179.5 \times 16 \times 97^2)\right) + \frac{1}{10^4}\left(\frac{9.5 \times 347^3}{12} + (347 \times 9.5 \times 84.5^2)\right)$$

$$= 8370cm^4$$

$$Z = \frac{I_{xx}}{y_{max}} = \frac{8370}{25.8} = 324cm^3$$

Moment capacity $M_{cC} = p_y Z = \dfrac{275 \times 324}{10^3} = 89.1kNm$

$$\boldsymbol{F_v\,(t_1 + e_3 + g_b) = 30kNm < 89.1kNm}$$

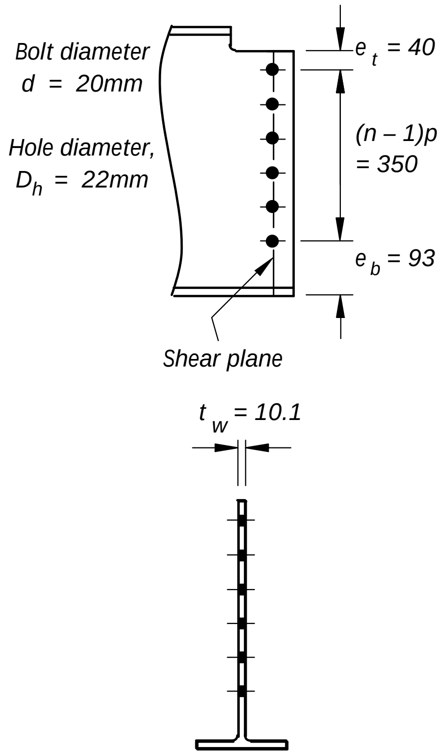
$\therefore$ O.K.

Title *Example 1 - Double Angle Cleats - Beam to Beam*	Sheet *10 of 18*

For 533 x 210 x 92 UB Grade S275

Plain shear capacity

$$P_v = \min(0.6\,p_y A_v,\ 0.7\,p_y K_e A_{v.net})$$



Shear area, $A_v = (e_t + (n-1)\,p + e_b)\,t_w$

(for Single notched beam)

$$= (40 \times (6-1)\,70 + 93) \times 10.1$$

$$= 4878\text{mm}^2$$

Net shear area, $A_{v.net} = A_v - n\,D_h\,t_w$

$$= 4878 - (6 \times 22 \times 10.1)$$

$$= 3545\text{mm}^2$$

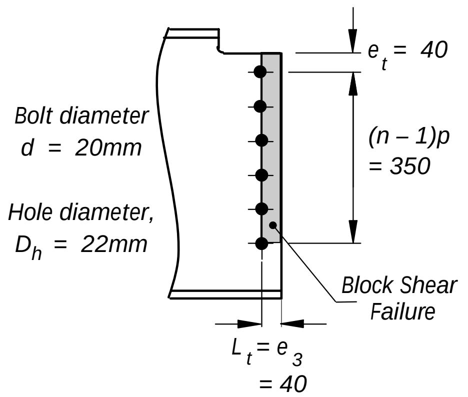
$$0.6\,p_y A_v = \frac{0.6 \times 275 \times 4878}{10^3} = 805\text{kN}$$

$$0.7 p_y K_e A_{v.net} = \frac{0.7 \times 275 \times 1.2 \times 3545}{10^3} = 819\text{kN}$$

$$\therefore\ P_v = 805\text{kN}$$

Block shear capacity

$$P_r = 0.6\,p_y\,t_w(L_v + K_e(L_t - kD_h))$$



$$L_v = e_t + (n-1)\,p$$

$$= 40 + 350 = 390\text{mm}$$

single line of bolts

$$k = 0.5 \text{ and}$$

$$kD_h = 0.5 \times 22 = 11\text{mm}$$

$$\therefore\ P_r = \frac{0.6 \times 275 \times 10.1\,(390 + 1.2(40 - 11))}{10^3}$$

$$= 708\text{kN}$$

Shear capacity of supported beam, $P_{v.min}$

$$P_{v.min} = \min(P_v,\ P_r)$$

$$\therefore\ P_{v.min} = P_r = 708\text{kN}$$

Applied shear, F_v = 465kN < 708kN **∴ O.K.**

Title	Example 1 - Double Angle Cleats - Beam to Beam	Sheet	11 of 18

(iii) Bearing

Basic requirement: $F_s \leq P_{bs}$

For 406 x 178 x 74 UB Grade S275

$$P_{bs} = d\ t_w\, p_{bs} \leq 0.5\ e_3\ t_w\ p_{bs}$$

$$d\ t_w\, p_{bs} = \frac{20 \times 9.5 \times 460}{10^3} = 87.4kN$$

p_{bs} from BS 5950-1 Table 32

$$0.5\ e_3\ t_w\ p_{bs} = \frac{0.5 \times 40 \times 9.5 \times 460}{10^3} = 87.4kN$$

$\therefore\ P_{bs} = 87.4kN$

$F_s = 64.9kN$ — Sheet 4

$\boldsymbol{F_s = 64.9kN < 87.4kN}$ — **∴ O.K.**

Note, the connection capacity is thus: $\dfrac{87.4 \times 300}{64.9} = 404kN$

p_{bs} from BS 5950-1 Table 32

For 533 x 210 x 92 UB Grade S275

$$P_{bs} = d\ t_w\, p_{bs} \leq 0.5\ e_3\ t_w\ p_{bs}$$

$$d\ t_w\, p_{bs} = \frac{20 \times 10.1 \times 460}{10^3} = 92.9kN$$

$$0.5\ e_3\ t_w\ p_{bs} = \frac{0.5 \times 40 \times 10.1 \times 460}{10^3} = 92.9kN$$

$\therefore\ P_{bs} = 92.9kN$

$F_s = 91kN$ — Sheet 4

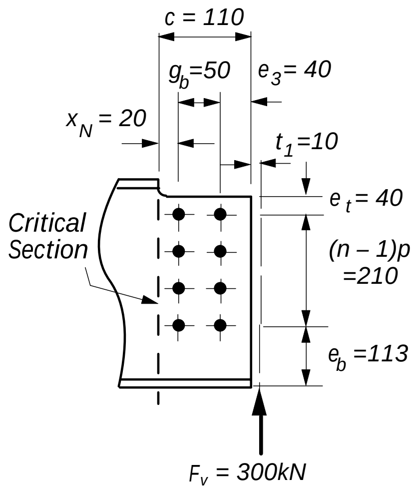
$\boldsymbol{F_s = 91kN < 92.9kN}$ — **∴ O.K.**

Note, the connection capacity is thus: $\dfrac{92.9 \times 465}{91} = 475kN$

Title Example 1 - Double Angle Cleats - Beam to Beam	Sheet 12 of 18

CHECK 5: Supported Beam - Capacity at a notch

Shear and Bending Interaction at the notch



For 406 x 178 x 74 UB Grade S275

Double bolt line. Find whether:

$x_N < 2d$ or $x_N \geq 2d$

$2d = 2 \times 20 = 40mm$

$x_N = 20mm$

since $x_N < 2d$, requirement (b) applies

Basic requirement:

(b) max ($F_v(t_1+c)$, $F_v(t_1+e_3+g_b)$) $\leq$ M_{cN}

$M_{cN} = M_{cC} = 89.1kNm$ — from Check 4 (sheet 9)

$$F_v(t_1+c) = \frac{300 \times (10+110)}{10^3} = 36.0kNm$$

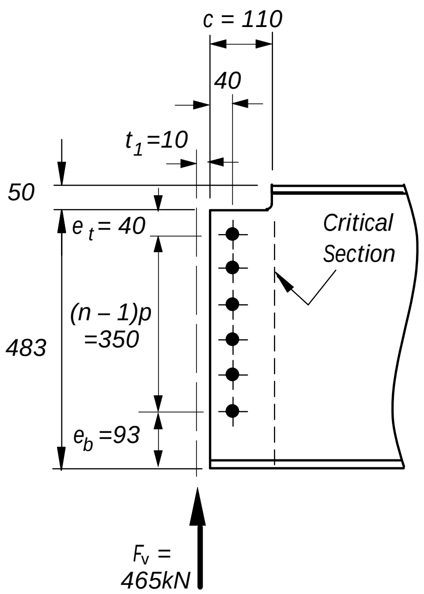
$$F_v(t_1+e_3+g_b) = \frac{300 \times (10+40+50)}{10^3} = 30.0kNm$$

Eccentric moment $F_v\,(t_1 + c)$ = 36kNm < 89.1kNm — **∴ O.K.**

For 533 x 210 x 92 Grade S275

Single bolt line

(a) Basic requirement: $F_v\,(t_1 + c) \leq M_{cN}$



For low shear condition:

$F_v \leq 0.75\,P_{vN}$

$P_{vN} = 0.6\,p_y\,A_{vN}$

$A_{vN} = (e_t + (n-1)\,p + e_b)t_w$ — $A_{vN} = A_v$ From sheet 10

$= 4878mm^2$

$P_{vN} = 0.6\,p_y\,A_{vN}$

$$= \frac{0.6 \times 275 \times 4878}{10^3}$$

$= 805kN$

$0.75P_{vN} = 0.75 \times 805 = 604kN$

$F_v = 465kN \leq 604kN$

∴ low shear criteria for bending applies

$\therefore M_{cN} = P_y\ Z_N$

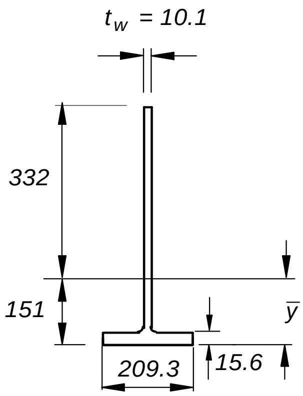
Title *Example 1 - Double Angle Cleats - Beam to Beam*	Sheet *13 of 18*

Taking moments of area about bottom flange:

$$(209.3 \times 15.6 \times 7.8) + (467.4 \times 10.1 \times 249.3) = \left((209.3 \times 15.6) + (467.4 \times 10.1)\right) \times \bar{y}$$

$$\bar{y} = 151\text{mm}$$

Second moment of area about neutral axis:



gross section

$$I_{xx} = \frac{1}{10^4}\left(\frac{209.3 \times 15.6^3}{12} + (209.3 \times 15.6 \times 143.2^2)\right) + \frac{1}{10^4}\left(\frac{10.1 \times 467.4^3}{12} + (467.4 \times 10.1 \times 98.3^2)\right)$$

$$= 19858\text{cm}^4$$

$$Z_N = \frac{I_{xx}}{y_{max}} = \frac{19858}{33.2} = 598\text{cm}^3$$

Moment capacity, $M_{cN} = p_y.Z_N = \dfrac{275 \times 598}{10^3} = 164\text{kNm}$

Eccentric moment, $F_v\ (t_1 + c) = \dfrac{465 \times (10 + 110)}{10^3}$

$$= \mathbf{55.8kNm < 164kNm}$$

∴ O.K.

Title *Example 1 - Double Angle Cleats - Beam to Beam*	Sheet *14 of 18*

CHECK 6: Supported Beam - Local Stablity of notched beam

When the beam is restrained against lateral torsional buckling no account need be taken of notch stability provided the following conditions are met:

For one flange notched beam in S275 steel

Basic requirements:

Notch depth $d_{c1} \leq \frac{D}{2}$

and $c \leq D$ for $\frac{D}{t_w} \leq 54.3$

$c \leq \frac{160000D}{(D/t_w)^3}$ for $\frac{D}{t_w} > 54.3$

c from Sheet 12

For 406 x 178 x 74 UB Grade S275 *(c = 110mm)*

Notch depth $d_{c1} = 50mm < \frac{412.8}{2} = 206.4mm$ ∴ O.K.

$\frac{D}{t_w} = \frac{412.8}{9.5} = 43.5 < 54.3$

$c = 110mm < 412.8mm$ ∴ O.K.

c from Sheet 12

For 533 x 210 x 92 UB*(c =110 mm)*

Notch depth $d_{c1} = 50mm < \frac{533.1}{2} = 266.6mm$ ∴ O.K..

$\frac{D}{t_w} = \frac{533.1}{10.1} = 52.8 < 54.3$

$c = 110mm < 533.1mm$ ∴ O.K.

CHECK 7: Not applicable

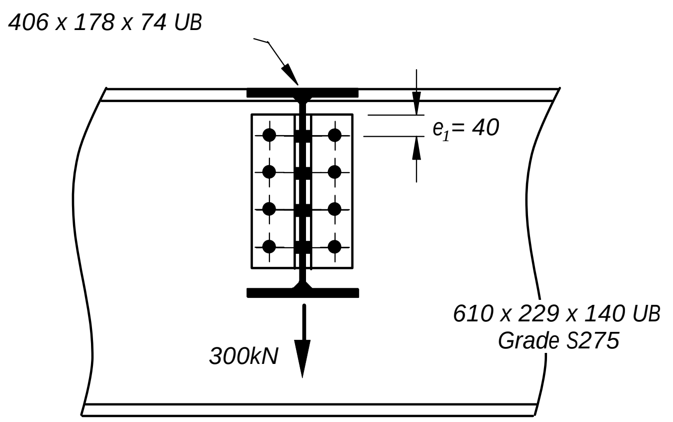
(because the supported beam is being considered as restrained against lateral torsional buckling)

Title	Example 1 - Double Angle Cleats - Beam to Beam	Sheet	15 of 18

CHECK 8: Supporting Beam - Bolt Group

Basic requirement: $F_v \leq \Sigma P_s$

For 406 x 178 x 74 UB



Shear capacity of single bolt, P_s = $p_s A_s$ — p_s from BS 5950-1 Table 30.

For M20 grade 8.8 bolts, $P_s = \dfrac{375 \times 245}{10^3}$ — See Bolt capacities yellow pages Table H.49

$= 91.9\text{kN}$

But for top pair of bolts P_s is the lesser of $p_s A_s$ or $0.5\, k_{bs}\, e_1\, t_c\, p_{bs}$

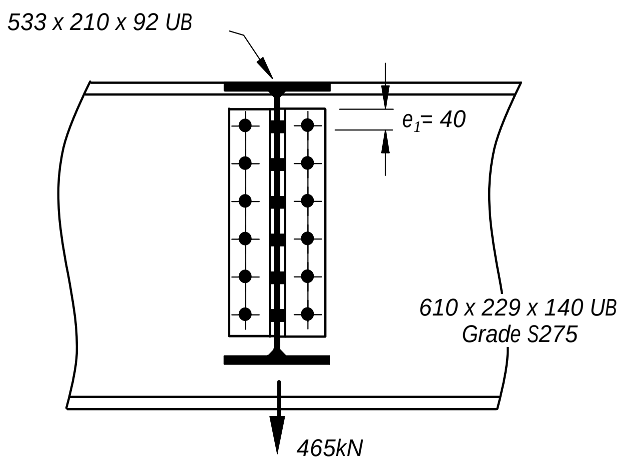
$0.5\, k_{bs}\, e_1\, t_c\, p_{bs} = 0.5 \times \dfrac{1.0 \times 40 \times 10 \times 460}{10^3}$ — p_{bs} from BS 5950-1 Table 32.

$= 92\text{kN}$ $\therefore$ use 91.9kN

$\Sigma P_s = 8 \times 91.9 = 735\text{kN}$

$\mathbf{F_v = 300kN < 735kN}$ **∴ O.K.**

For 533 x 210 x 92 UB



$0.5\, k_{bs}\, e_1\, t_c\, p_{bs} = 92\text{kN}$ $\therefore$ use 91.9kN

$\Sigma P_s = 12 \times 91.9 = 1103\text{kN}$

$\mathbf{F_V = 465kN < 1103kN}$ **∴ O.K.**

Title *Example 1 - Double Angle Cleats - Beam to Beam*	Sheet *16 of 18*

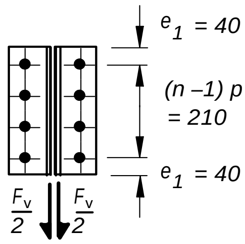
CHECK 9: Supporting Beam - Connecting elements

Shear and bearing of cleats connected to supporting beam

(i) Basic requiremenf for shear: $\frac{F_v}{2} \leq P_{v.min}$

For 406 x 178 x 74 UB

Shear capacity of one angle cleat, $P_{v.min}$ is the smaller of plain shear capacity P_v and block shear capacity P_r



$F_v = 300kN$

Plain shear $P_v = \min(0.6\ p_y\ A_v\ ,\ 0.7\,p_y\,K_e\ A_{v.net})$

Shear area, $A_v = 0.9\,(2e_1 + (n-1)\,p)\,t_c$

$= 0.9\,(80 + 210) \times 10 = 2610mm^2$

Net area, $A_{v.net} = A_v - n\,D_h\,t_c$

$= 2610 - (4 \times 22 \times 10) = 1730mm^2$

$0.6\,p_y\ A_v = \frac{0.6 \times 275 \times 2610}{10^3} = 431kN$

$0.7\,p_y\,K_e\ A_{v.net} = \frac{0.7 \times 275 \times 1.2 \times 1730}{10^3} = 400kN$

$\therefore\ P_v = 400kN$

Block shear $P_r = 0.6p_y\,t_c\,(L_v + K_e\,(L_t - kD_h))$

$L_v = e_1 + (n-1)p = 40 + 210 = 250mm$

$L_t = e_2 = 40mm$

$k = 0.5$ (for single line of bolts)

$K_e = 1.2$ (for S275)

$\therefore\ P_r = \frac{0.6 \times 275 \times 10\,(250 + 1.2(40 - 0.5 \times 22))}{10^3}$

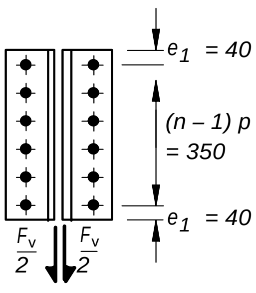
$= 470kN$

$\therefore\ P_{v.min} = \min(P_v\,,\,P_r)$

$= 400kN$

$\mathbf{F_v/2 = 150kN < 400kN}$ — **$\therefore$ O.K.**

For 533 x 210 x 92 UB



$F_v = 465kN$

Plain shear P_v

$A_v = 0.9(80 + 350) \times 10 = 3870mm^2$

$A_{v.net} = 3870 - (6 \times 22 \times 10) = 2550mm^2$

$0.6\,p_y\ A_v = \frac{0.6 \times 275 \times 3870}{10^3} = 639kN$

$0.7\,p_y\,K_e\ A_{v.net} = \frac{0.7 \times 275 \times 1.2 \times 2550}{10^3} = 589kN$

$P_v = 589kN$

Block shear P_r

$P_r = \frac{0.6 \times 275 \times 10\,(390 + 1.2(40 - 0.5 \times 22))}{10^3}$

$= 701kN$

$P_{v.min} = \min(P_v\ ,\ P_r)$

$= 589kN$

$\mathbf{F_v/2 = 233kN < 589kN}$ — **$\therefore$ O.K.**

NOTE: *Block shear checks have been shown here, but they are never critical for well proportioned cleats. However, if the bolt spacing is concentrated at one part of a cleat then these checks may be critical.*

Title	Example 1 - Double Angle Cleats - Beam to Beam	Sheet	17 of 18

(ii) Basic requirement for bearing: $\frac{F_v}{2} \leq \Sigma P_{bs}$

For 406 x 178 x 74 UB

Bearing capacity, $P_{bs} = k_{bs} d\, t_c\, p_{bs}$

but for top bolt $P_{bs} = \min(k_{bs} d\, t_c\, p_{bs},\ 0.5 k_{bs}\, e_1\, t_c\, p_{bs})$

p_{bs} from BS 5950-1 Table 32

$$k_{bs} d\, t_c\, p_{bs} = \frac{1.0 \times 20 \times 10 \times 460}{10^3} = 92kN$$

$$0.5 k_{bs} e_1\, t_c\, p_{bs} = \frac{1.0 \times 20 \times 10 \times 460}{10^3} = 92kN$$

$$\Sigma P_{bs} = 3 \times 92 + 92 = 368kN$$

$$\frac{F_v}{2} = \frac{300}{2} = 150kN < 368kN$$

∴ O.K.

For 533 x 210 x 92 UB

Bearing capacity as above

$$\Sigma P_{bs} = 5 \times 92 + 92 = 552kN$$

$$\frac{F_v}{2} = \frac{465}{2} = 233kN < 552kN$$

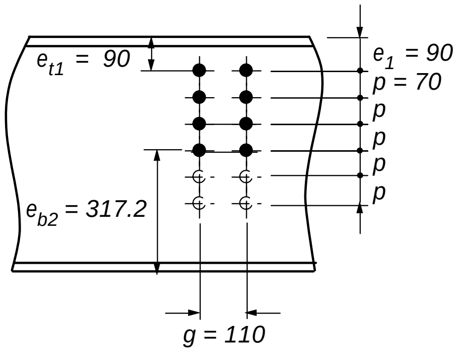
∴ O.K.

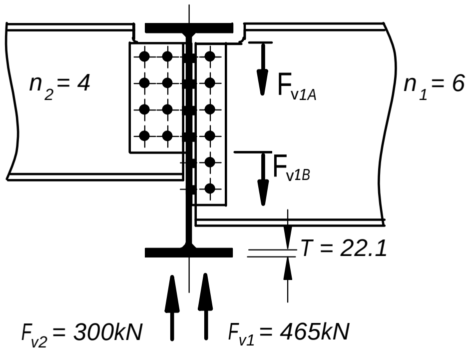
Title	Example 1 - Double Angle Cleats - Beam to Beam	Sheet	18 of 18

CHECK 10 : Supporting Beam - Local Capacity

Local Shear & Bearing Capacity of beam web supporting two beams

(i) Basic requirement for shear: $\frac{F_{v1A}}{2} + \frac{F_{v2}}{2} \leq P_v$





For double sided portion, $\frac{F_{v1A}}{2} + \frac{F_{v2}}{2} \leq P_v$

$$F_{v1A} = F_{v1}\frac{n_2}{n_1} = 465 \times \frac{4}{6} = 310kN$$

Shear capacity, $P_v = min\left(0.6\, p_y\, A_v\ ,\ 0.7\, p_y\, K_e\, A_{v.net}\right)$

Gross shear area, $A_v = \left(e_t + (n_2 - 1)\, p + e_b\right)\, t_w$

$e_t = min\,(e_{t1}\ ,\ 5d) = 90mm$

$e_b = min\,(e_{b2}\ ,\ {}^{g}/_{2}\ ,\ 5d) = 55mm$

$A_v = \left(90 + (4 - 1)\, 70 + 55\right) \times 13.1 = 4651mm^2$

$\therefore\ 0.6\, p_y\, A_v = \frac{0.6 \times 265 \times 4651}{10^3} = 740kN$ — p_y from BS 5950-1 Table 9

Net shear area, $A_{v.net} = A_v - n_2\, D_h\, t_w$

$= 4651 - (4 \times 22 \times 13.1) = 3498mm^2$

$\therefore\ 0.7 p_y K_e A_{v.net} = \frac{0.7 \times 265 \times 1.2 \times 3498}{10^3} = 779kN$

$\therefore\ P_v = 740kN$

$$\frac{F_{v1A}}{2} + \frac{F_{v2}}{2} = \frac{310}{2} + \frac{300}{2} = 305kN$$

305kN < 740kN **∴ O.K**

Note: The above check is for **local** shear only; the effects of any global shear forces must also be considered.

(ii) Basic requirement for bearing: $\frac{F_{v1}/2}{n_1} + \frac{F_{v2}/2}{n_2} \leq P_{bs}$

Bearing capacity of beam web, $P_{bs} = d\, t_w\, p_{bs}$ — p_{bs} from BS 5950-1 Table 32

$$= \frac{20 \times 13.1 \times 460}{10^3} = 121kN$$

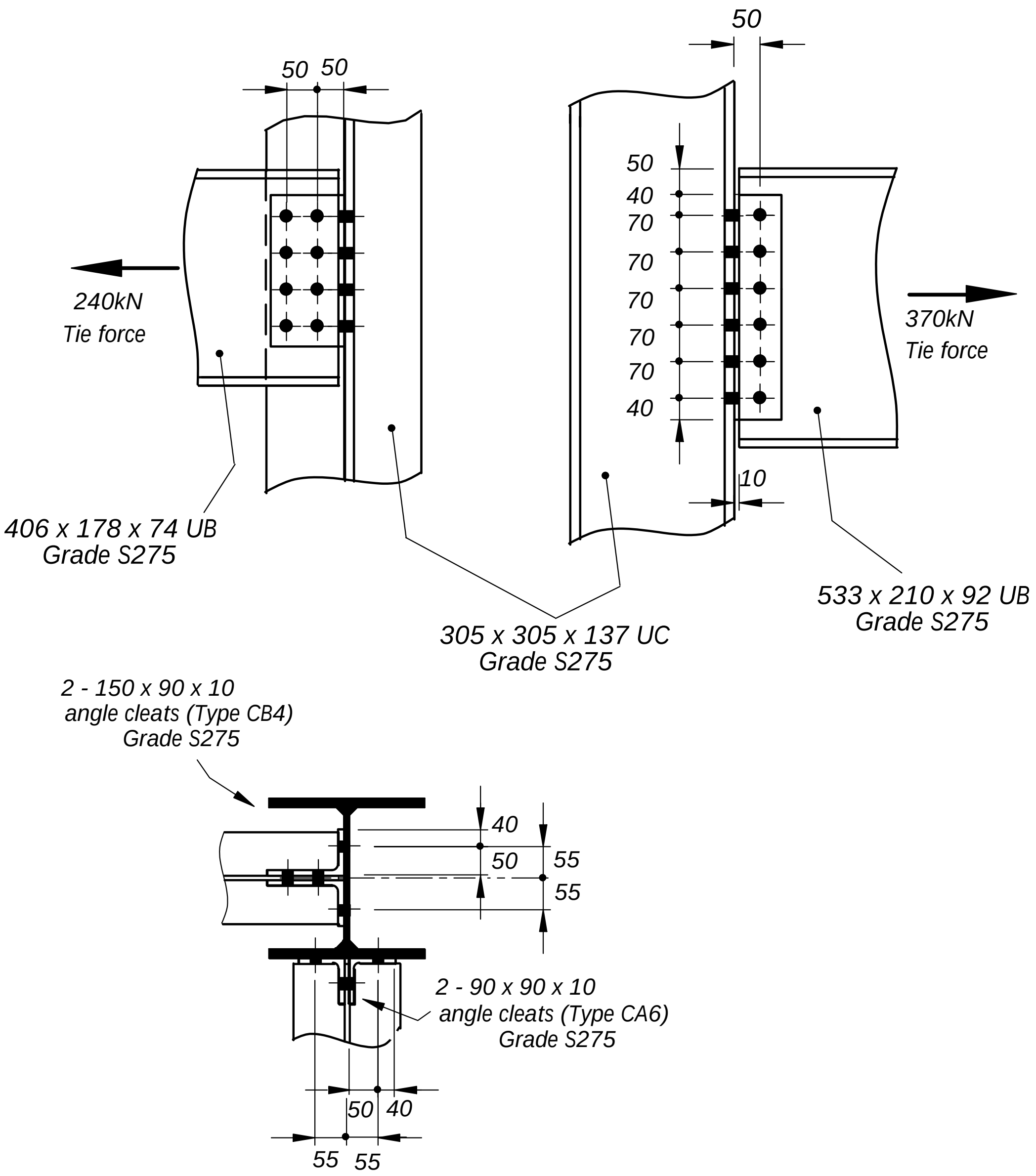
$$\frac{F_{v1}/2}{n_1} + \frac{F_{v2}/2}{n_2} = \frac{465/2}{6} + \frac{300/2}{4} = 76kN$$

76kN < 121kN **∴ O.K**

CALCULATION SHEET BCSA	Job No: Joints in Steel Construction - Simple Connections	Sheet: 1 of 8
	Title: Example 2 - Double Angle Cleats – Beam to UC Column – Structural Integrity	
	Client: SCI/BCSA Connections Group	
	Calcs by: RS · Checked by: AM	Date: May 2002

DESIGN EXAMPLE 2

Check the following beam to column connections for the tie force shown.



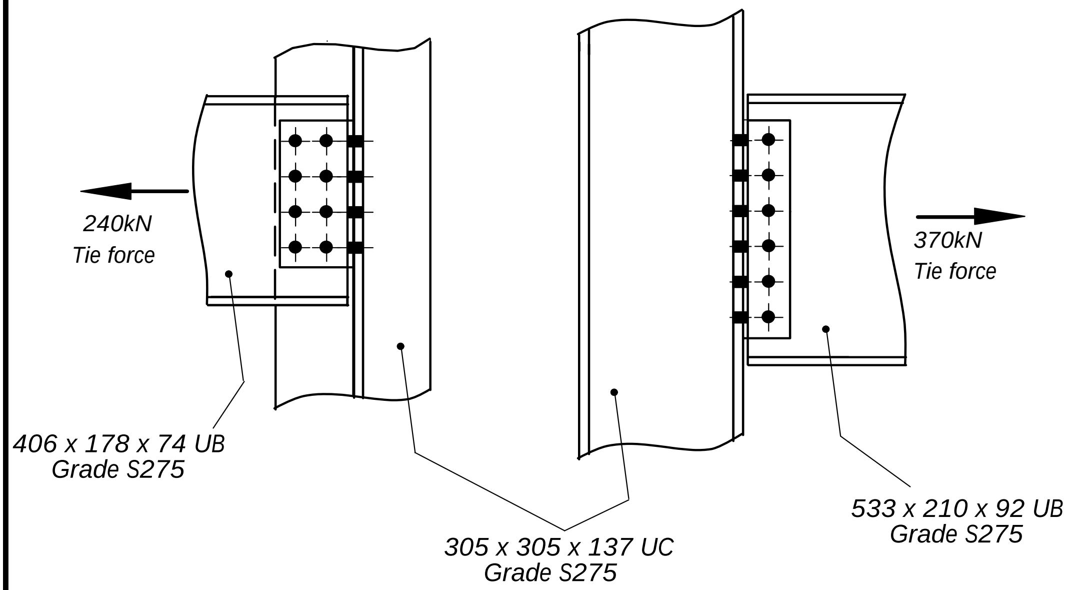
For cleat details see Figure 4.4 and yellow pages Table H.2

Design Information:

Bolts:	M20 8.8
Material:	All S275 steel

Title *Example 2 - Double Angle Cleats – Beam to UC Column – Structural Integrity*	Sheet *2 of 8*

CONNECTION DESIGN USING CAPACITY TABLES FROM THE YELLOW PAGES



406 x 178 x 74 UB Grade S275

Cleat type CB4 Grade S275

Bolts M20 8.8

From capacity table H.10 in yellow pages

Connection tying capacity
= 448kN > 240kN

The beam side of the connection is adequate

533 x 210 x 92 UB Grade S275

Cleat type CA6 Grade S275

Bolts M20 8.8

From capacity table H.9 in yellow pages

Connection tying capacity
= 558kN > 370kN

The beam side of the connection is adequate

Yellow pages Tables H.10 & H.9

∴ O.K.

Note:

(I) *The tying capacity of the connection given in the tables in the yellow pages is the least value obtained from CHECKS 11, 12 and 13. The values for CHECK 11 are based upon the large displacement analysis given in Appendix B.*

(2) *Beams connecting into a column web must also be checked for web bending as shown in CHECK 14 on sheet 7.*

In the case of the 406UB illustrated, CHECK 14 gives the critical tie capacity unless the web is stiffened.

Title	*Example 2 - Double Angle Cleats – Beam to UC Column – Structural Integrity*	Sheet	3 of 8

SUMMARY OF FULL DESIGN CHECKS FOR EXAMPLE 2

Notes: ***(i)*** ** ( ) A greater capacity for angle cleats can be obtained using the large displacement analysis described in Appendix B. It is an iterative procedure more suited to computation by computer.*

(ii) *The tie force values in the capacity tables are based on the large displacement analysis.*

Sheet Nos	***CHECK***		***406UB (S275)***		***533UB (S275)***		***305UC (S275)***	
			Capacity	***Applied Load***	***Capacity***	***Applied Load***	***Capacity***	***Applied Load***
4	***CHECK 11*** *Structural Integrity* *- Connecting Elements - Tension capacity of double angle cleats (kN)*		***333*** *(448)**	***240***	***492*** *(662)**	***370***		
			CRITICAL CHECK BEAM SIDE					
5 & 6	***CHECK 12*** *Structural integrity* *- Supported Beam _ Tension and bearing capacity of beam web*	*Tension (kN)* *Bearing (kN)*	*674* *874*	*240* *240*	*828* *558*	*370* *370*	*Not Applicable*	
6	***CHECK 13*** *Structural integrity* *- Tension capacity of bolts (kN)*		*588*	*240*	*882*	*370*		
7 & 8	***CHECK 14*** *Structural integrity* *- Tying capacity of supporting column web (kN)*		*Not Applicable*				***392***	***240***
							CRITICAL CHECK COLUMN SIDE	

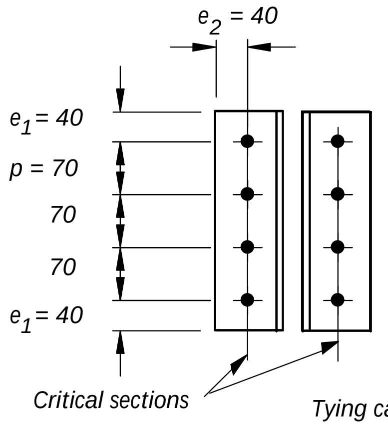
Title *Example 2 - Double Angle Cleats – Beam to UC Column – Structural Integrity*	Sheet *4 of 8*

CHECK 11 Structural Integrity - Connecting Elements

Tension capacity of pair of web angle cleats

Basic requirement: ***Tie force $\leq$ Tying capacity of double angle web cleats***

For 406 x 178 x 74 UB



Tying capacity of double angle web cleats

$= 0.6\, L_e\, t_c\, p_y$ *(For S275 steel)*

Effective net length of cleats is L_e

$L_e = 2\,e_e + (n-1)\,p_e - n\,D_h$

$e_e = e_1$ but $\leq e_2$ = 40mm

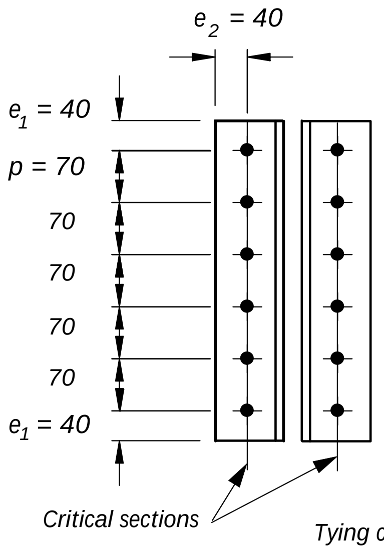
$p_e = p$ but $\leq 2e_2$ = 70mm

$L_e = (2 \times 40) + (3 \times 70) - (4 \times 22)$ = 202mm

Tying capacity $= \dfrac{0.6 \times 202 \times 10 \times 275}{10^3}$ = 333kN

Tie Force = 240kN < 333kN ***∴ O.K.***

For 533 x 210 x 92 UB



Tying capacity of double angle web cleats

$= 0.6\, L_e\, t_c\, p_y$ *(For S275Steel)*

$L_e = 2\,e_e + (n-1)\,p_e - n\,D_h$

$e_e = e_1$ but $\leq e_2$ = 40mm

$p_e = p$ but $\leq 2e_2$ = 70mm

$L_e = (2 \times 40) + (5 \times 70) - (6 \times 22)$ = 298mm

Tying capacity $= \dfrac{0.6 \times 298 \times 10 \times 275}{10^3}$ = 492kN

Tie Force = 370kN < 492kN ***∴ O.K.***

Title *Example 2 - Double Angle Cleats – Beam to UC Column – Structural Integrity*	Sheet *5 of 8*

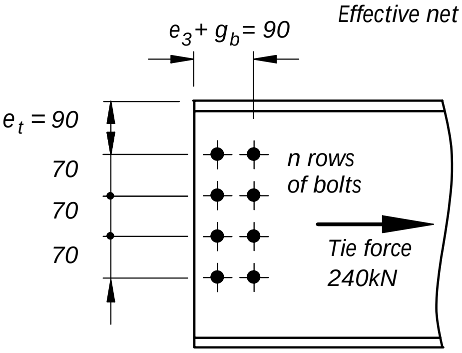
CHECK 12: Structural Integrity - Supported Beam

Tension & Bearing Capacity of Beam Web

(i) For Tension:

Basic requirement: *Tie force* $\leq$ *Net tension capacity of beam web*

For 406 x 178 x 74 UB S275 (double line of bolts)



Net tension capacity of beam web $= L_e\, t_w\, p_y$

Effective net length, $L_e = 2\, e_e + (n-1)\, p_e - n\, D_h$

$e_e = (e_3 + g_b - D_h)$ but $\leq e_t$

$= 68\text{mm} \leq 90$

$p_e = p$ but $\leq 2(e_3 + g_b - D_h)$

$= 70\text{mm} \leq 2(90-22)$

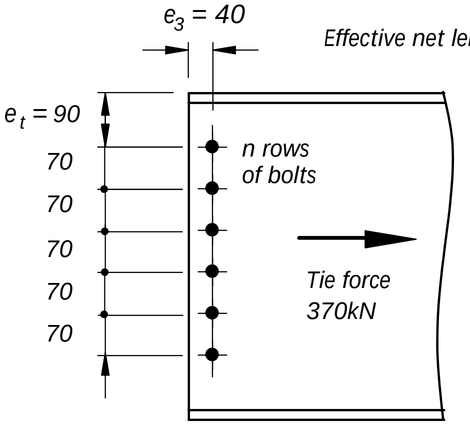
$L_e = (2 \times 68) + (3 \times 70) - (4 \times 22)$

$= 258\text{mm}$

Net tension capacity $= \dfrac{258 \times 9.5 \times 275}{10^3} = 674\text{kN}$ — *See note below*

Tie Force = 240kN < 674kN **$\therefore$ O.K.**

For 533 x 210 x 92 UB Grade S275 (single line of bolts)



Net tension capacity of beam web $= L_e\, t_w\, p_y$

Effective net length, $L_e = 2\, e_e + (n-1)\, p_e - n\, D_h$

$e_e = e_3$ but $\leq e_t$

$= 40\text{mm} \leq 90$

$p_e = p$ but $\leq 2e$

$= 70\text{mm} \leq 2 \times 40$

$L_e = (2 \times 40) + (5 \times 70) - (6 \times 22)$

$= 298\text{mm}$

Net tension capacity $= \dfrac{298 \times 10.1 \times 275}{10^3} = 828\text{kN}$ — *See note below*

Tie Force = 370kN < 828kN **$\therefore$ O.K.**

Note: This value may not be the same as that given in capacity tables, where a double notched beam is assumed (i.e. $e_t = 40$mm).

Title Example 2 - Double Angle Cleats – Beam to UC Column – Structural Integrity	Sheet 6 of 8

(ii) For bearing:

Basic requirement: ***Tie force*** $\leq$ ***Bearing capacity of beam web***

For 406 x 178 x 74 UB Grade S275 (double line of bolts)

Bearing capacity of beam web $= 3\,n\,d\,t_w\,p_{bs}$

but $\leq n(1.5d\,t_w\,p_{bs} + 0.5\,e_3\,t_w\,p_{bs})$

p_{bs} from BS 5950-1 Table 32

$$3\,n\,d\,t_w\,p_{bs} = \frac{3 \times 4 \times 20 \times 9.5 \times 460}{10^3} = 1049\text{kN}$$

$$n(1.5d\,t_w\,p_{bs} + 0.5\,e_3\,t_w\,p_{bs}) = \frac{4(1.5 \times 20 \times 9.5 \times 460 + 0.5 \times 40 \times 9.5 \times 460)}{10^3} = 874\text{kN}$$

$\therefore$ *Bearing capacity of beam web* = 874kN

***Tie Force* = 240kN < 874kN** $\therefore$ **O.K.**

For 533 x 210 x 92 UB Grade S275 (single line of bolts)

Bearing capacity of beam web $= 1.5\,n\,d\,t_w\,p_{bs}$ but $\leq 0.5n\,e_3\,t_w\,p_{bs})$

p_{bs} from BS 5950-1 Table 32

$$1.5\,n\,d\,t_w\,p_{bs} = \frac{1.5 \times 6 \times 20 \times 10.1 \times 460}{10^3} = 836\text{kN}$$

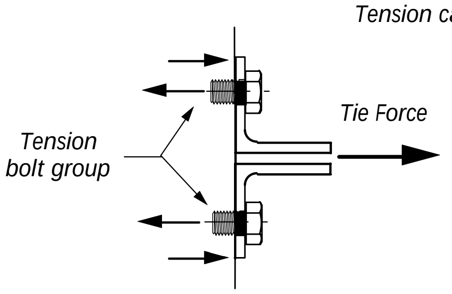
$$0.5n\,e_3\,t_w\,p_{bs} = \frac{0.5 \times 6 \times 40 \times 10.1 \times 460}{10^3} = 558\text{kN}$$

$\therefore$ *Bearing capacity of beam web* = 558kN

***Tie Force* = 370kN < 558kN** $\therefore$ **O.K.**

CHECK 13 : Structural Integrity - Tension Capacity of Bolts

Basic requirement : *Tie Force* $\leq$ *Tension capacity of tension bolt group*



Tension capacity $= 2\,n\,A_t\,p_{tr}$

p_{tr} = *reduced tension strength of bolt in presence of extreme prying force*

= 300N/mm^2 for grade 8.8 bolts — Appendix D

A_t = *tensile stress area of bolt*

= 245mm^2 — Bolt capacity tables Yellow pages Table H.49

For 406 x 178 x 74 UB

$$\text{Tension capacity} = \frac{2 \times 4 \times 245 \times 300}{10^3} = 588\text{kN}$$

***Tie Force* = 240kN < 588kN** $\therefore$ **O.K.**

For 533 x 210 x 92 UB

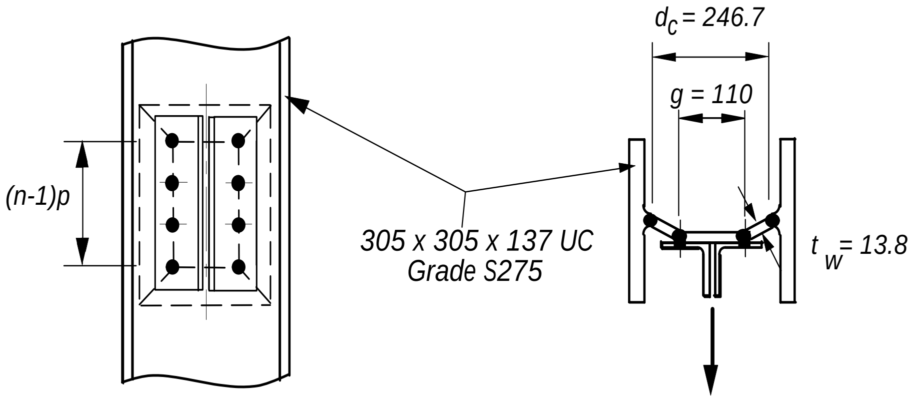
$$\text{Tension capacity} = \frac{2 \times 6 \times 245 \times 300}{10^3} = 882\text{kN}$$

***Tie Force* = 370kN < 882 kN** $\therefore$ **O.K.**

Title Example 2 - Double Angle Cleats – Beam to UC Column – Structural Integrity	Sheet 7 of 8

CHECK 14: Structural Integrity – Capacity of Supporting Column Web

Basic requirement: **Tie Force $\leq$ Tying capacity of column web**



Tying capacity of column web $= \dfrac{8\,M_u}{1-\beta_1}\left[\eta_1 + 1.5(1-\beta_1)^{0.5}(1-\gamma_1)^{0.5}\right]$

M_u = moment capacity of column web per unit length

$= \dfrac{p_u\,t_w^2}{4}$

$= \dfrac{328 \times 13.8^2}{4 \times 10^3} = 15.6\text{kNm/mm}$

$\eta_1 = \dfrac{(n-1)\,p - \frac{n}{2}\,D_h}{d_c}$

$= \dfrac{(4-1) \times 70 - \frac{4}{2} \times 22}{246.7} = 0.673$

$\beta_1 = \dfrac{g}{d_c}$

$= \dfrac{110}{246.7} = 0.446$

$\gamma_1 = \dfrac{D_h}{d_c}$

$= \dfrac{22}{246.7} = 0.089$

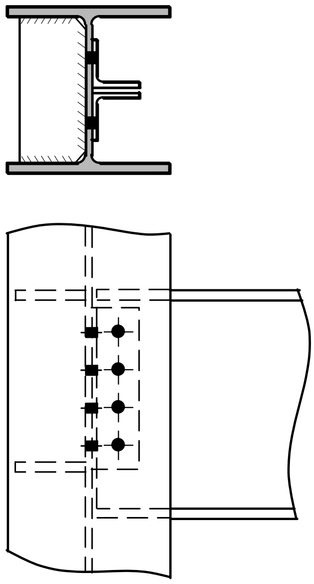
Tying capacity of column web $= \dfrac{8 \times 15.6}{1-0.446}\left[0.673 + 1.5(1-0.446)^{0.5} \times (1-0.089)^{0.5}\right]$

$= 225.3\left[0.673 + (1.116 \times 0.954)\right] = 392\text{kN}$

Tie Force = 240kN < 392kN **∴ O.K.**

Title *Example 2 - Double Angle Cleats – Beam to UC Column – Structural Integrity*	Sheet *8 of 8*

If column web fails to satisfy the criteria shown on sheet 7 then sfiffeners fillet welded on one side to the web and flanges would be required thus:



CALCULATION SHEET BCSA	Job: Joints in Steel Construction - Simple Connections	Sheet: 1 of 9
	Title: Example 3 - Double Angle Cleats - Beam to RHS column using Flowdrill	
	Client: SCI/BCSA Connections Group	
	Calcs by: RS / Checked by: ASM	Date: May 2002

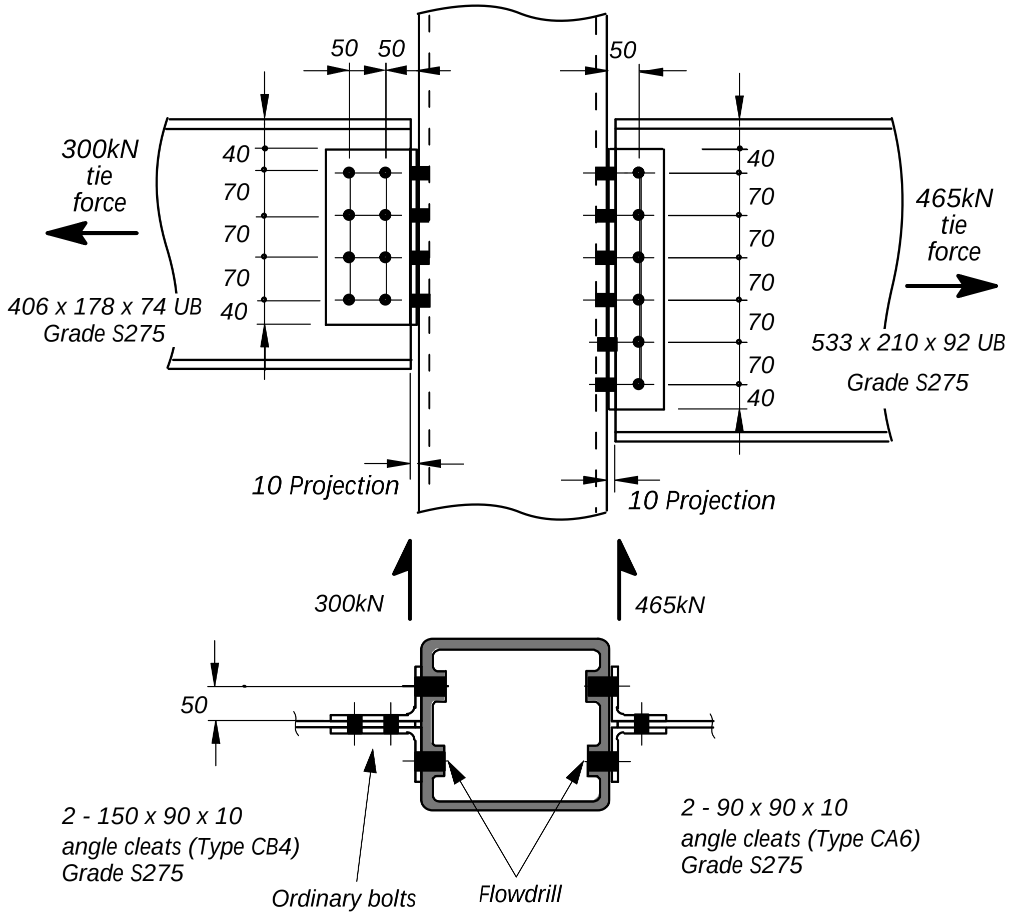
DESIGN EXAMPLE 3

REF.

Check the following beam to RHS column connection for the design forces shown using 8.8 bolts in Flowdrill threaded holes in the column.

In this example it is assumed that the tying force is equal to the end reaction. However, depending on how the floor beams are arranged, the tying force given by the formula in BS 5950-1 clause 2.4.5.3 can sometimes be less.

Note: The connections should be checked independently for (i) shear forces and (ii) Tie forces and NOT for both the forces acting at the same time.
The yellow pages should be used for initial selection of angle cleats.



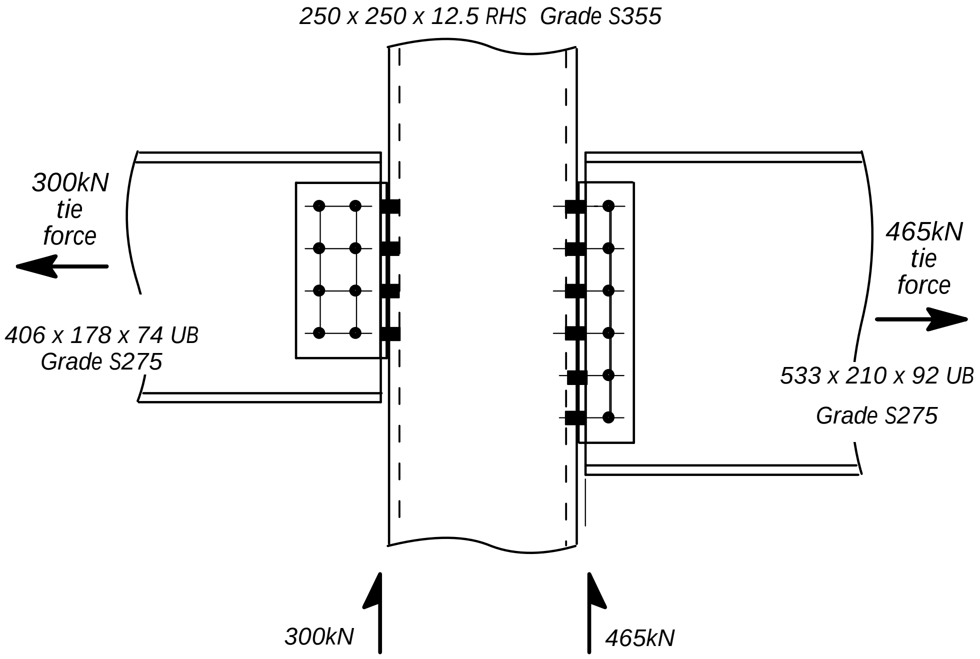
For cleat details see Figure 4.4 & Yellow pages Table H.2

Design Information:

Flowdrill:	M20 8.8
Ordinary bolts:	M20 8.8
Column:	S355
Beams:	S275
Angle cleats:	S275

Title *Example 3 - Double Angle Cleats - Beam to RHS column using Flowdrill*	Sheet *2 of 9*

CONNECTION DESIGN USING CAPACITY TABLES FROM THE YELLOW PAGES



406 x 178 x 74 UB Grade S275	533 x 210 x 92 UB Grade S275	
Cleat type CBD Grade S275	**Cleat type CAS Grade S275**	
Column Bolts Flowdrill M20 8.8	**Column Bolts Flowdrill M20 8.8**	
Beam Bolts M20 8.8	**Beam Bolts M20 8.8**	*Yellow pages Tables H.10 & H.9*
From capacity table H.10 in yellow pages	*From capacity table H.9 in yellow pages*	
Connection vertical shear capacity = 404kN > 300kN	*Connection vertical shear capacity = 475kN > 465kN*	∴ **O.K**
Minimum support thickness = 4.6 mm < 12.5mm	*Minimum support thickness = 3.6mm < 12.5mm*	∴ **O.K.**
Connection tying capacity = 448kN > 300kN	*Connection tying capacity = 558kN > 465kN*	∴ **O.K.**
The beam side of the connection is adequate	***The beam side of the connection is adequate***	

Note:

(I) The tying capacity of the connection given in the tables in the yellow pages is the least value obtained from CHECKS 11, 12 and 13. The values for CHECK 11 are based upon the large displacement analysis given in Appendix B.

(2) Beams connecting into a column web or RHS wall must also be checked for web bending as shown in CHECK 15 on sheets 8 and 9.

Title	Example 3 - Double Angle Cleats - Beam to RHS column using Flowdrill	Sheet 3 of 9

SUMMARY OF FULL DESIGN CHECKS FOR EXAMPLE 3

Notes: ***(i)*** *CHECKS 1 to 9, where applicable, are all as shown in Example 1 and are not repeated in this example but the calculated capacities are summarised below.*

(ii) *CHECK 4, Shear capacities are higher than those shown for example 1, because there are no notched beams in Example 3.*

(iii) *CHECKS 11 and 12 are as shown in Example 2 and are not repeated in this example.*

(iv) *In accordance with BS 5950-1; tie forces are ignored when checking the capacity to resist vertical reactions and vertical reactions are ignored when calculating the capacity to resist tie forces.*

Sheet Nos	CHECK		406UB (S275) Capacity	406UB (S275) Applied Load	533UB (S275) Capacity	533UB (S275) Applied Load	RHS Column (S355) 406UB Side Capacity	RHS Column (S355) 406UB Side Applied Load	RHS Column (S355) 533UB Side Capacity	RHS Column (S355) 533UB Side Applied Load
See Example 1 Sheets 4 to 18	**CHECK 1** - Recommended detailing practice		All recommendations adopted							
	CHECK 2 Supported Beam - Bolt Group Shear Capacity	Shear (per bolt ,kN)	184	64.9	184	91	Not Applicable			
	CHECK 3 Supported Beam - Connecting Elements (Strength of cleat)	Shear (kN) Bearing (per bolt ,kN)	400 92	150 32.5	589 92	233 45.5	Not Applicable			
	CHECK 4 Supported Beam - Capacity at connection See (ii) above	Shear (kN) Bearing (per bolt,kN)	647 (plain shear) **87.4**	300 **64.9**	888 (plain shear) **92.9**	465 **91**	Not Applicable			
			CRITICAL CHECK FOR VERTICAL SHEAR							
	CHECKS 5, 6 & 7		Not Applicable				Not Applicable			
	CHECK 8 Supporting Column - Bolt Group shear capacity	Shear (bolt group,kN)	735	300	1103	465	Not Applicable			
	CHECK 9 Supporting Column - Connecting Element (Strength of cleat)	Shear (kN) Bearing (Capacity per bolt line, kN)	400 368	150 150	589 552	233 233	Not Applicable			
4 & 5	**CHECK 10** Supporting Column - Capacity (Local capacity of column web)	Shear (kN) Bearing (per bolt , kN)	Not Applicable				972 138	150 37.5	1316 138	233 38.8
6	**CHECK 11** Structural Integrity -Connecting Elements Tension capacity of of double angle cleats	Tension (kN)	**333**	**300**	**492**	**465**	Not Applicable			
			CRITICAL CHECK FOR TIE FORCE							
6	**CHECK 12** Structural integrity - Supported Beam Tension and bearing capacity of beam web	Tension (kN) Bearing (kN)	674 874	300 300	828 558	465 465	Not Applicable			
7	**CHECK 13** Structural Integrity - Tension bolt group	Tension (kN)	Not Applicable				584	300	876	465
7	**CHECK 14**		Not Applicable				Not Applicable			
8 & 9	**CHECK 15** Structural Integrity - Supporting column wall	Tension (kN)	Not Applicable				455	300	597	465

Title	Example 3 - Double Angle Cleats - Beam to RHS column using Flowdrill	Sheet	4 of 9

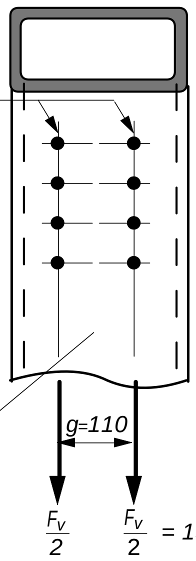
CHECK 10 : Supporting Column - Local Capacity

Shear & Bearing Capacity of column wall

(i) Basic requirement for shear: $\frac{F_v}{2} \leq P_v$

For 406 x 178 x 74 UB side

RHS Column Grade S355



Shear capacity, $P_v = \min(0.6\, p_y\, A_v\, ,\, 0.7\, p_y\, K_e\, A_{v.net})$

Gross shear area, $A_v = (e_t + (n-1)\,p + e_b)\; t_w$

e_b = smaller of $g/2$ and $5\,d$ = 55mm

e_t = smaller of e_{t1} and $5\,d$

since the connection is not near the top of column e_{t1} is not applicable

$e_t = 5d = 5 \times 20 = 100\text{mm}$

$A_v = (100 + (4-1)\,70 + 55) \times 12.5 = 4563\text{mm}^2$

$0.6\, p_y\, A_v = \frac{0.6 \times 355 \times 4563}{10^3} = 972\text{kN}$

Net shear area, $A_{v.net} = A_v - n\, D_h\, t_w$

for FLOWDRILL connections D_h = bolt diameter = 20mm

$A_{v.net} = 4563 - (4 \times 20 \times 12.5) = 3563\text{mm}^2$

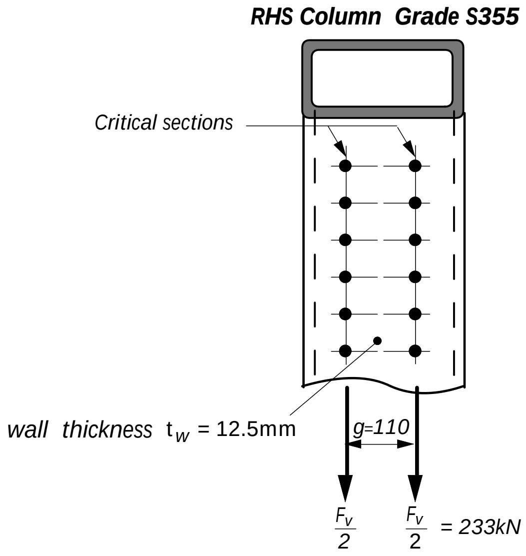
$0.7\, p_y\, K_e\, A_{v.net} = \frac{0.7 \times 355 \times 1.1 \times 3563}{10^3} = 974\text{kN}$

$\therefore\; P_v = 972\text{kN}$

$$\frac{F_v}{2} = 150\text{kN} < 972\text{kN}$$ **∴ O.K.**

Title *Example 3 - Double Angle Cleats - Beam to RHS column using Flowdrill* Sheet *5 of 9*

For 533 x 210 x 92 UB side



	e_b	=	smaller of $g/2$ and 5d	=	55mm
	e_t	=	5d = 5 x 20	=	100mm
	A_v	=	$(100 + (6 - 1)\ 70 + 55) \times 12.5$	=	$6313mm^2$
	$0.6\ p_y\ A_v$	=	$\frac{0.6 \times 355 \times 6313}{10^3}$	=	1345kN
	$A_{v.net}$	=	$6313 - (6 \times 20 \times 12.5)$	=	$4813mm^2$
	$0.7\ p_y\ K_e\ A_{v.net}$	=	$\frac{0.7 \times 355 \times 1.2 \times 4813}{10^3}$	=	1316kN
∴	P_v	=	1316kN		

$$\frac{F_v}{2} = 233kN < 1316kN$$ **∴ O.K.**

Note: *The above check is for **local** shear only; the effects of any global shear forces must also be considered.*

(ii) Basic requirement for bearing: $\frac{F_v}{2n} \leq P_{bs}$

Bearing capacity of column wall per bolt $P_{bs} = k_{bs}\ d\ t_w\ p_{bs}$

$$= \frac{1.0 \times 20 \times 12.5 \times 550}{10^3} = 138kN$$

p_{bs} from BS 5950-1 Table 32

For 406 x 178 x 74 UB side

$$\frac{F_v}{2n} = \frac{300}{2 \times 4} = 37.5kN < 138kN$$ **∴ O.K.**

For 533 x 210 x 92 UB side

$$\frac{F_v}{2n} = \frac{465}{2 \times 6} = 38.8kN < 138kN$$ **∴ O.K.**

Title *Example 3 - Double Angle Cleats - Beam to RHS column using Flowdrill*	Sheet *6 of 9*

CHECK 11 Structural Integrity - Connecting Elements

Tension capacity of pair of web angle cleats

This check is identical to CHECK 11 in Example 2

Basic requirement: Tie force $\leq$ tying capacity of double angle web cleats

For 406 x 178 x 74UB Side

Example 2 DAC sheet 4 of 8

Tying capacity of double angle cleats = 333kN

Tie Force = 300kN < 333kN **∴ O.K.**

For 533 x 210 x 92UB Side

Example 2 DAC sheet 4 of 8

Tying capacity of double angle cleats = 492kN

Tie Force = 465kN < 492kN **∴ O.K.**

CHECK 12 Structural Integrity - Supported Beam

Tension and Bearing Capacity of the Beam Web

This check is identical to CHECK 12 in Example 2

(i) For Tension.

Basic requirement: Tie force $\leq$ net tension capacity of beam webs

Example 2 DAC sheet 5 of 8

For 406 x 178 x 74UB

Net tension capacity of beam web = 674kN

Tie Force = 300kN < 674kN **∴ O.K.**

Example 2 DAC sheet 5 of 8

For 533 x 210 x 92UB

Net tension capacity of beam web = 828kN

Tie Force = 465kN < 828kN **∴ O.K.**

(ii) For Bearing.

Basic requirement: Tie force $\leq$ bearing capacity of beam webs

Example 2 DAC sheet 6 of 8

For 406 x 178 x 74UB

Bearing capacity of beam web = 874kN

Tie Force = 300kN < 874kN **∴ O.K.**

Example 2 DAC sheet 6 of 8

For 533 x 210 x 92UB

Bearing capacity of beam web = 558kN

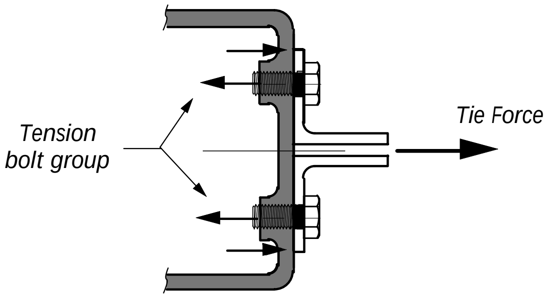
Tie Force = 465kN < 558kN **∴ O.K.**

Title	Example 3 - Double Angle Cleats - Beam to RHS column using Flowdrill	Sheet	7 of 9

CHECK 13 Structural Integrity - Tension Capacity of Bolts

Basic requirement: **Tie Force** $\leq$ **Tension capacity of bolt group**

Tension capacity = $2\ n\ P_{si}$



P_{si} = Flowdrill Structural integrity Tensile Capacity — *Flowdrill Pull-out Capacity*

= 73kN for a grade 8.8 bolt in a 12.5mm RHS column wall — *yellow pages Table H.55b*

For 406 x 178 x 74 UB Side

Tension capacity = 2 x 4 x 73 = 584kN

Tie Force = 300kN < 584kN **∴ O.K.**

For 533 x 210 x 92 UB Side

Tension capacity = 2 x 6 x 73 = 876kN

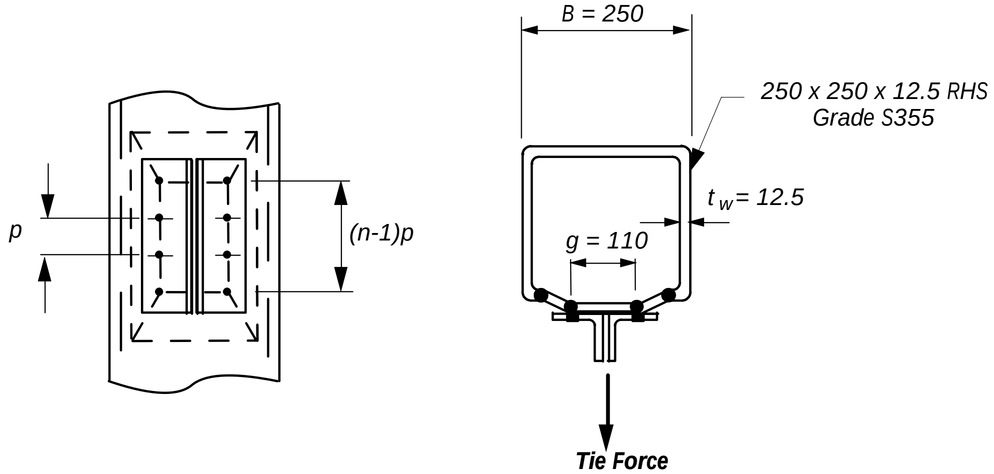
Tie Force = 465kN < 876kN **∴ O.K.**

CHECK 14: not applicable

Title	Example 3 - Double Angle Cleats - Beam to RHS column using Flowdrill	Sheet	8 of 9

CHECK 15: Structural Integrity – Capacity of Supporting Column Wall (RHS)

Basic requirement: **Tie Force $\leq$ Tying capacity of RHS column wall**



For 406 x 178 x 74 UB Side

Tying capacity of column wall $= \dfrac{8\,M_u}{1-\beta_1}\left[\eta_1 + 1.5(1-\beta_1)^{0.5}(1-\gamma_1)^{0.5}\right]$

M_u = moment capacity of RHS column wall per unit length

$= \dfrac{p_u\,t_w^2}{4}$

$= \dfrac{392 \times 12.5^2}{4 \times 10^3} = 15.3\text{kNm/mm}$

D_h = 20mm (Bolt diameter for Flowdrill)

$\eta_1 = \dfrac{(n-1)\,p - \frac{n}{2}\,D_h}{B - 3t_w}$

$= \dfrac{(4-1)\;70 - \frac{4}{2} \times 20}{250 - 3 \times 12.5} = 0.80$

$\beta_1 = \dfrac{g}{B - 3t_w}$

$= \dfrac{110}{212.5} = 0.518$

$\gamma_1 = \dfrac{D_h}{B - 3t_w}$

$= \dfrac{20}{212.5} = 0.094$

Tying capacity of column wall $= \dfrac{8 \times 15.3}{1 - 0.518}\left[0.8 + 1.5(1-0.518)^{0.5} \times (1-0.094)^{0.5}\right]$

$= 253.9\left[0.8 + 1.041 \times 0.952\right] = 455\text{kN}$

Tie Force = 300kN < 455kN **∴ O.K.**

Title *Example 3 - Double Angle Cleats - Beam to RHS column using Flowdrill*	Sheet *9 of 9*

For 533 x 210 x 92 UB Side

$$\eta_1 = \frac{(n-1)\ p - \frac{n}{2}\ D_n}{B - 3t_w}$$

$$= \frac{(6-1)\ 70 - \frac{6}{2} \times 20}{250 - 3 \times 12.5} = 1.36$$

Tying capacity of column wall

$$= \frac{8\ M_u}{1 - \beta_1} \left[\eta_1 + 1.5(1 - \beta_1)^{0.5} (1 - \gamma_1)^{0.5} \right]$$

$$= \frac{8 \times 15.3}{1 - 0.518} \left[1.36 + 1.5(1 - 0.518)^{0.5} \times (1 - 0.094)^{0.5} \right]$$

$$= 253.9 \left[1.36 + 1.041 \times 0.952 \right] = 597kN$$

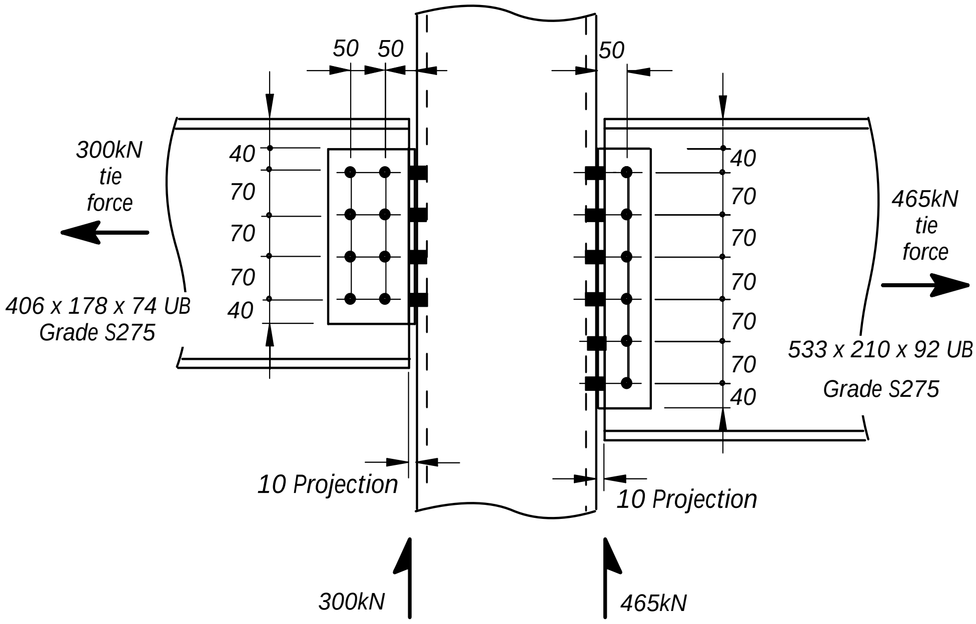
Tie Force = 465kN < 597kN **∴ O.K.**

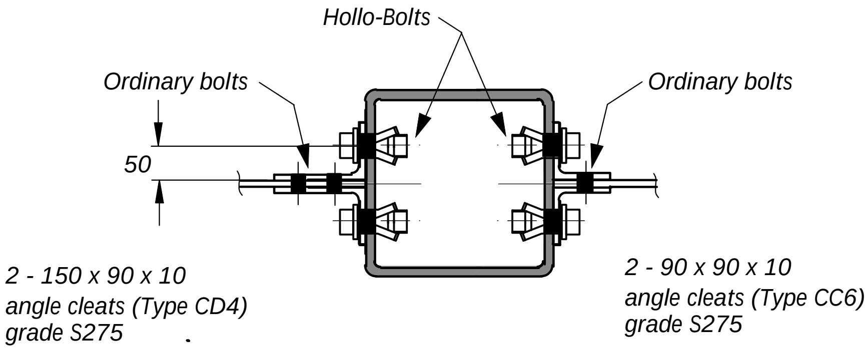
CALCULATION SHEET BCSA	Job: Joints in Steel Construction - Simple Connections		Sheet: 1 of 11
	Title: Example 4 – Double Angle Cleats - Beam to RHS column using Hollo-bolt		
	Client: SCI/BCSA Connections Group		
	Calcs by: RS	Checked by: ASM	Date: May 2002

REF.

DESIGN EXAMPLE 4

Check the following beam to RHS column connection for the design forces shown using Hollo-Bolt connectors to the column.
In this example it is assumed that the tying force is equal to the end reaction. However, depending on how the floor beams are arranged, the tying force given by the formula in BS 5950-1 clause 2.4.5.3 can sometimes be less.
The yellow pages should be used for initial selection of angle cleats.





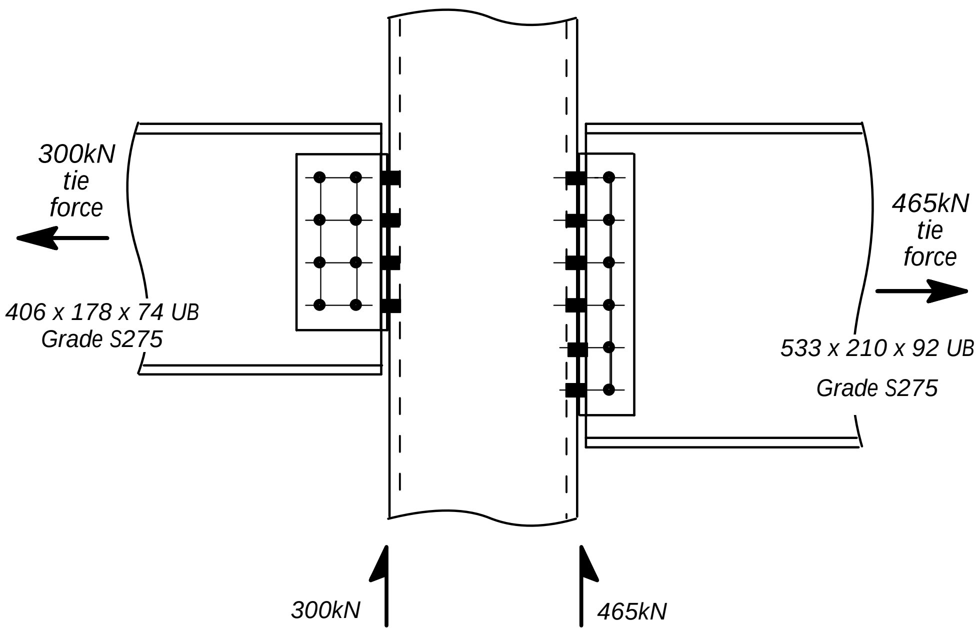
For cleat details see Figure 4.4 & Yellow pages Table H.2

Design Information:

Hollo-bolts:	M20
Ordinary bolts:	M20 8.8
Column:	S355
Beams:	S275
Angle cleats:	S275

Title	*Example 4 – Double Angle Cleats - Beam to RHS column using Hollo-bolt*	Sheet	*2 of 11*

CONNECTION DESIGN USING CAPACITY TABLES FROM THE YELLOW PAGES



406 x 178 x 74 UB	533 x 210 x 92 UB	
Cleat type CD4 Grade S275	**Cleat type CC6 Grade S275**	
Column bolts Hollo-bolts M20	**Column bolts Hollo-bolts M20**	*Yellow pages Tables H.14 & H.13*
Beam bolts M20 8.8	**Beam bolts M20 8.8**	
From capacity table H.14 in yellow pages	*From capacity table H.13 in yellow pages*	
Connection vertical shear capacity = 404kN > 300kN	*Connection vertical shear capacity = 475kN > 465kN*	**∴ O.K.**
Minimum support thickness = 4.6 mm < 12.5mm	*Minimum support thickness = 3.6mm < 12.5mm*	**∴ O.K.**
Connection tying capacity = 393kN > 300kN	*Connection tying capacity = 558kN > 465kN*	**∴ O.K.**
The beam side of the connection is adequate	***The beam side of the connection is adequate***	

Note:

(I) The tying capacity of the connection given in the tables in the yellow pages is the least value obtained from CHECKS 11, 12 and 13. The values for CHECK 11 are based upon the large displacement analysis given in Appendix B.

(2) Beams connecting into a column web or RHS wall must also be checked for web bending as shown in CHECK 15 on sheets 10 and 11.

Example 4 – Double Angle Cleats - Beam to RHS column using Hollo-bolt	Sheet 3 of 11

SUMMARY OF FULL DESIGN CHECKS FOR EXAMPLE 4

Notes (i) CHECKS 1 to 7, where applicable, are all as shown in Example 1 and are not repeated in this example but the calculated capacities are summarised below.

(ii) CHECK 4 shear capacities are higher than those for example1 because there are no notched beams in Example 4.

(iii) CHECK 12 is as shown in Example 2 and is not repeated in this example.

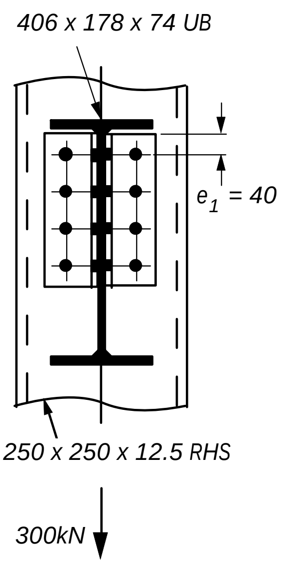
(iv) In accordance with BS 5950-1; tie forces are ignored when checking the capacity to resist vertical reactions and vertical reactions are ignored when calculating the capacity to resist tie forces.

(v) ( )* Values from Yellow pages,Table H.14, using rigorous approach of Appendix B

Sheet Nos	CHECK		406UB (S275) Capacity	406UB (S275) Applied Load	533UB (S275) Capacity	533UB (S275) Applied Load	RHS Column (S355) 406UB Side Capacity	RHS Column (S355) 406UB Side Applied Load	RHS Column (S355) 533UB Side Capacity	RHS Column (S355) 533UB Side Applied Load
See Example 1 Sheets 4 to 14	**CHECK 1** - Recommended detailing practice		All recommendations adopted							
See Example 1 Sheets 4 to 14	**CHECK 2** Supported Beam - Bolt Group Shear Capacity	Shear (per bolt kN)	184	64.9	184	91	Not Applicable			
See Example 1 Sheets 4 to 14	**CHECK 3** Supported Beam - Connecting Elements (Strength of cleat)	Shear (kN) Bearing (per bolt kN)	400 92	150 32.5	589 92	233 45.5	Not Applicable			
See Example 1 Sheets 4 to 14	**CHECK 4** Supported Beam - Capacity at connection See note (ii) above	Shear (kN) Bearing (per bolt, kN)	647 (plain shear) **87.4**	300 **64.9**	888 (plain shear) **92.9**	465 **91**	Not Applicable			
			CRITICAL CHECK FOR VERTICAL SHEAR							
See Example 1 Sheets 4 to 14	**CHECKS 5, 6 & 7**		Not Applicable				Not Applicable			
4	**CHECK 8** Supporting Column - Bolt Group shear capacity	Shear (bolt group, kN)	784	300	1184	465	Not Applicable			
5	**CHECK 9** Supporting Column - Connecting Element (Strength of cleat)	Shear (kN) Bearing (Capacity per bolt line, kN)	280 368	150 150	409 552	233 233	Not Applicable			
7 & 8	**CHECK 10** Supporting Column - Capacity (Local capacity of column web)	Shear (kN) Bearing (per bolt, kN)	Not Applicable				769 138	150 37.5	1008 138	233 38.8
9	**CHECK 11** Structural Integrity -Connecting Elements Tension capacity of of double angle cleats	Tension (kN)	**248 (393)***	**300**	**363 (580)***	**465**	Not Applicable			
			CRITICAL CHECK FOR TIE FORCE							
See Ex. 2 Shts. 5 & 6	**CHECK 12** Structural integrity - Supported Beam Tension and bearing capacity of beam web	Tension (kN) Bearing (kN)	674 874	300 300	828 558	465 465	Not Applicable			
10	**CHECK 13** Structural Integrity - Tension bolt group	Tension (kN)	Not Applicable				584	300	876	465
10	**CHECK 14**		Not Applicable				Not Applicable			
10 & 11	**CHECK 15** Structural Integrity - Supporting column wall	Tension (kN)	Not Applicable				409	300	534	465

Title	Example 4 – Double Angle Cleats - Beam to RHS column using Hollo-bolt	Sheet	4 of 11

CHECK 8: Supporting Column - Bolt Group



Basic requirement: $F_v \leq \Sigma P_s$

Shear capacity of single bolt, $P_s = p_s A_s$

For M20 Hollo-bolts shear capacity $P_s = 100kN$ — *Yellow pages Table H.56*

But for top pair of bolts, $P_s = \min\left(p_s A_s,\ 0.5\, k_{bs}\, e_1\, t_c\, p_{bs}\right)$ *(where* $k_{bs} = 1.0$*)*

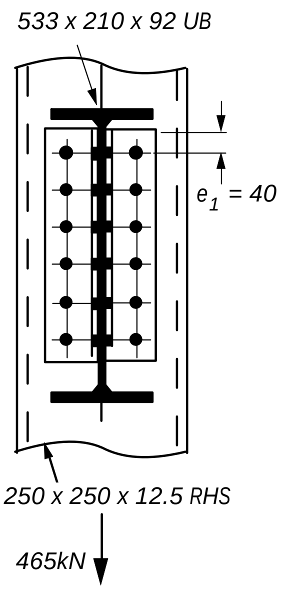
For 406 x 178 x 74 UB

$$0.5\, k_{bs}\, e_1\, t_c\, p_{bs} = \frac{0.5 \times 1.0 \times 40 \times 10 \times 460}{10^3} = 92kN$$

$$\Sigma P_s = 2 \times 92 + 6 \times 100 = 784kN$$

$$\mathbf{F_v = 300kN < 784kN}$$

∴ O.K.



For 533 x 210 x 92 UB

$$\Sigma P_s = 2 \times 92 + 10 \times 100 = 1184kN$$

$$\mathbf{F_v = 465kN < 1184kN}$$

∴ O.K.

Title	Example 4 – Double Angle Cleats - Beam to RHS column using Hollo-bolt	Sheet	5 of 11

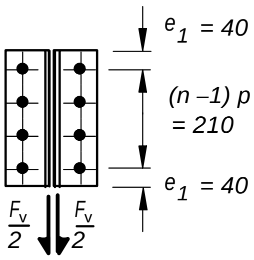
CHECK 9: Supporting Beam - Connecting elements

Shear and bearing of cleats connected to supporting column

(i) Basic requiremenf for shear: $\dfrac{F_v}{2} \leq P_{v.min}$

For 406 x 178 x 74 UB

Shear capacity of one angle cleat, $P_{v.min}$ is the smaller of plain shear capacity P_v and block shear capacity P_r



F_v = 300kN

Plain shear, $P_v = \min\left(0.6\ p_y\ A_v \text{ or } 0.7\ p_y K_e\ A_{v.net}\right)$

Shear area, $A_v = 0.9\left(2e_1 + (n-1)\,p\right) t_c$

$= 0.9\left(80 + 210\right) \times 10 = 2610\text{mm}^2$

Net area, $A_{v.net} = A_v - n\,D_h\,t_c$

for Hollo-Bolt connections $D_h = 35\text{mm}$ — *D_h from Yellow pages Table H.2*

$A_{v.net} = 2610 - (4 \times 35 \times 10) = 1210\text{mm}^2$

$0.6\,p_y\,A_v = \dfrac{0.6 \times 275 \times 2610}{10^3} = 431\text{kN}$

$0.7\,p_y K_e\,A_{v.net} = \dfrac{0.7 \times 275 \times 1.2 \times 1210}{10^3} = 280\text{kN}$

$\therefore\ P_v = 280\text{kN}$

Block shear, $P_r = 0.6\,p_y\,t_c\left(L_v + K_e\,(L_t - k\,D_h)\right)$

$L_v = (n-1)p + e_1 = 210 + 40 = 250\text{mm}$

$L_t = e_2 = 40\text{mm}$

$K_e = 1.2$ (for S275)

$K = 0.5$ (for single line of bolts)

$D_h = 35\text{mm}$ (hole diameter for Hollo-bolt)

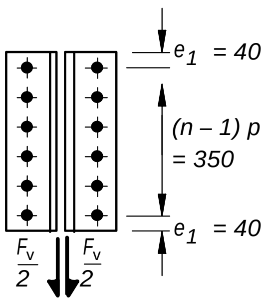
$\therefore\ P_r = \dfrac{0.6 \times 275 \times 10\left(250 + 1.2(40 - 0.5 \times 35)\right)}{10^3}$

$= 457\text{kN}$

$P_{v.min} = \min(P_v\ ,\ P_r) = 280\text{kN}$

$\dfrac{\mathbf{F_v}}{\mathbf{2}}$ **= 150kN < 280kN** — **∴ O.K.**

For 533 x 210 x 92 UB



F_v = 465kN

Plain shear,

$A_v = 0.9(80 + 350) \times 10 = 3870\text{mm}^2$

$A_{v.net} = 3870 - (6 \times 35 \times 10) = 1770\text{mm}^2$

$0.6\,p_y\,A_v = \dfrac{0.6 \times 275 \times 3870}{10^3} = 639\text{kN}$

$0.7\,p_y K_e\,A_{v.net} = \dfrac{0.7 \times 275 \times 1.2 \times 1770}{10^3} = 409\text{kN}$

$\therefore\ P_v = 409\text{kN}$

Block shear, $P_r = \dfrac{0.6 \times 275 \times 10\left(390 + 1.2(40 - 0.5 \times 35)\right)}{10^3}$

$= 688\text{kN}$

$P_{v.min} = \min(P_v\ ,\ P_r) = 409\text{kN}$

$\dfrac{\mathbf{F_v}}{\mathbf{2}}$ **= 233kN < 409kN** — **∴ O.K.**

Title	*Example 4 – Double Angle Cleats - Beam to RHS column using Hollo-bolt*	Sheet	*6 of 11*

(ii) Basic requirement for bearing: $\dfrac{F_v}{2} \leq \Sigma P_{bs}$

For 406 x 178 x 74 UB

Bearing capacity, $P_{bs} = k_{bs}\, d\, t_c\, p_{bs}$

but for top bolt, $P_{bs} = \min\left(k_{bs}\, d\, t_c\, p_{bs}\ ,\ 0.5k_{bs}\, e_1\, t_c\, p_{bs}\right)$

for Hollo-Bolt connections d = *"nominal" bolt diameter* = *20mm* — *Yellow pages Table H.61*

$$k_{bs}\, d\, t_c\, p_{bs} = \frac{1.0 \times 20 \times 10 \times 460}{10^3} = 92kN$$

$$0.5k_{bs}\, e_1\, t_c\, p_{bs} = \frac{0.5 \times 1.0 \times 40 \times 10 \times 460}{10^3} = 92kN$$

p_{bs} from BS 5950-1 Table 32

$$\Sigma P_{bs} = 3 \times 92 + 92 = 368kN$$

$$\frac{F_v}{2} = \mathbf{150kN} < \mathbf{368kN}$$

∴ O.K.

For 533 x 210 x 92 UB

Bearing capacities are as above

$$\Sigma P_{bs} = 5 \times 92 + 92 = 552kN$$

$$\frac{F_v}{2} = \mathbf{233kN} < \mathbf{552kN}$$

∴ O.K.

Title *Example 4 – Double Angle Cleats - Beam to RHS column using Hollo-bolt*	Sheet *7 of 11*

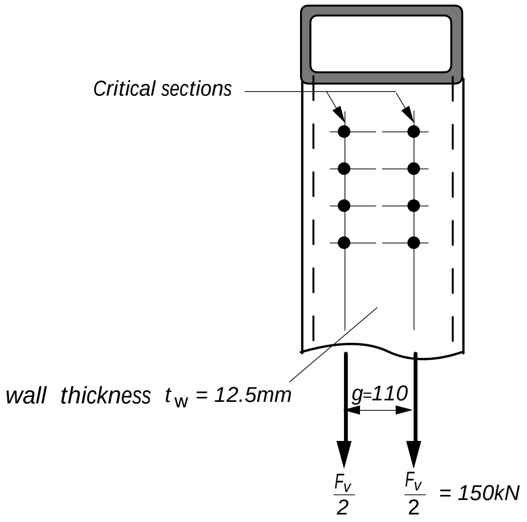
CHECK 10 : Supporting Column - Local Capacity

Shear & Bearing Capacity of column wall

(i) Basic requirement for shear: $\frac{F_v}{2} \leq P_v$

For 406 x 178 x 74 UB side

RHS Column Grade S355



Shear capacity, P_v $= \min\left(0.6\, p_y\, A_v\,,\; 0.7\, p_y K_e\, A_{v.net}\right)$

Gross shear area, A_v $= \left(e_t + (n-1)\,p + e_b\right)\, t_w$

e_b = smaller of $g/2$ and $5\,d$ = 55mm

e_t = smaller of e_{t1} and $5\,d$

since the connection is not near the top of column e_{t1} is not applicable

e_t = $5d$ = 5×20 = 100mm

A_v = $\left(100 + (4-1)\,70 + 55\right) \times 12.5$ = 4563mm^2

$\therefore$ $0.6\, p_y A_v = \frac{0.6 \times 355 \times 4563}{10^3}$ = 972kN

Net shear area, $A_{v.net} = A_v - n\, D_h\, t_w$

D_h from Yellow pages Table H.61

for Hollo-Bolt connections D_h = hole diameter = 35mm

$A_{v.net}$ = $4563 - (4 \times 35 \times 12.5)$ = 2813mm^2

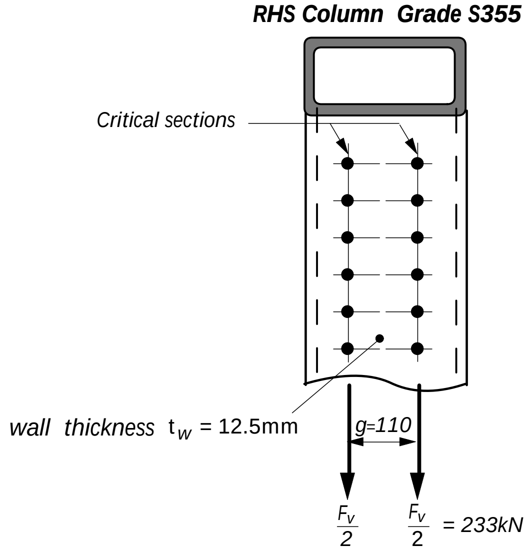
$\therefore$ $0.7\, p_y K_e\, A_{v.net} = \frac{0.7 \times 355 \times 1.1 \times 2813}{10^3}$ = 769kN

$\therefore$ P_v = 769kN

$\frac{F_v}{2}$ = **150kN** < **769kN** **$\therefore$ O.K.**

Title	*Example 4 – Double Angle Cleats - Beam to RHS column using Hollo-bolt*	Sheet *8 of 11*

For 533 x 210 x 92 UB side



e_b = smaller of $g/2$ and $5d$ = 55mm

e_t = $5d$ = 5×20 = 100mm

A_v = $(100 + (6 - 1)\,70 + 55) \times 12.5$ = $6313mm^2$

$\therefore\ 0.6\,p_y\,A_v = \frac{0.6 \times 355 \times 6313}{10^3} = 1345kN$

$A_{v.net} = 6313 - (6 \times 35 \times 12.5) = 3688mm^2$

$\therefore\ 0.7\,p_y\,K_e\,A_{v.net} = \frac{0.7 \times 355 \times 1.1 \times 3688}{10^3} = 1008kN$

$\therefore\ P_v = 1008kN$

$\boldsymbol{\frac{F_v}{2} = 233kN < 1008kN}$ **∴ O.K.**

Note: *The above check is for **local** shear only; the effects of any global shear forces must also be considered.*

(ii) Basic requirement for bearing: $\frac{F_v}{2n} \leq P_{bs}$

Bearing capacity of column wall per bolt $P_{bs} = k_{bs}\,d\,t_w\,p_{bs}$

$= \frac{1.0 \times 20 \times 12.5 \times 550}{10^3} = 138kN$

p_{bs} from BS 5950-1 Table 32

For 406 x 178 x 74 UB side

$\boldsymbol{\frac{F_v}{2n} = \frac{300}{2 \times 4} = 37.5kN < 138kN}$ **∴ O.K.**

For 533 x 210 x 92 UB side

$\boldsymbol{\frac{F_v}{2n} = \frac{465}{2 \times 6} = 38.8kN < 138kN}$ **∴ O.K.**

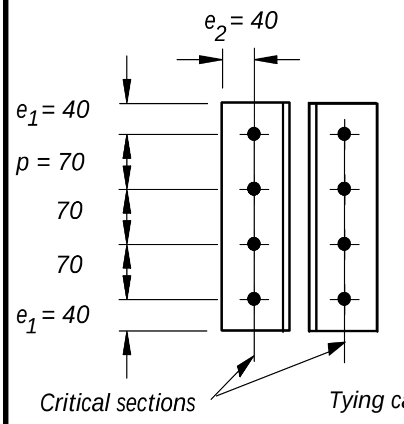
Title	Example 4 – Double Angle Cleats - Beam to RHS column using Hollo-bolt	Sheet	9 of 11

CHECK 11 Structural Integrity - Connecting Elements

Tension capacity of pair of web angle cleats

Basic requirement: **Tie force $\leq$ Tying capacity of double angle web cleats**

For 406 x 178 x 74 UB



Tying capacity of double angle web cleats

$= 0.6\, L_e\, t_c\, p_y$ (For S275 steel)

Effective net length of cleats is L_e

$L_e = 2\,e_e + (n-1)\,p_e - n\,D_h$

$e_e = e_1$ but $\leq e_2$ = 40mm

$p_e = p$ but $\leq 2e_2$ = 70mm

$L_e = (2 \times 40) + (3 \times 70) - (4 \times 35)$ = 150mm

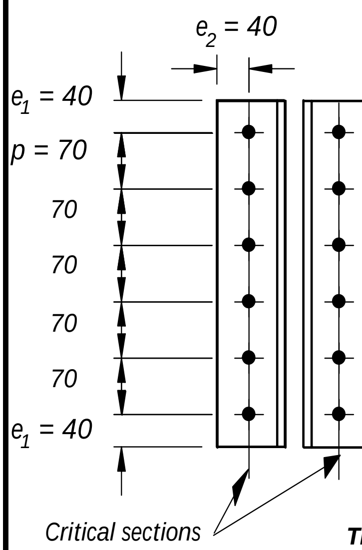
Tying capacity $= \dfrac{0.6 \times 150 \times 10 \times 275}{10^3}$ = 248kN

Tie Force = 300kN $\not<$ 248kN — **∴ Fails**

However, using the rigorous approach of Appendix B, the tying capacity is 393kN.
(For this standard connection, the value is given in the yellow pages) — Yellow pages Table H.14

Tie Force = 300kN < 393kN — **∴ O.K.**

For 533 x 210 x 92 UB



Tying capacity of double angle web cleats

$= 0.6\, L_e\, t_c\, p_y$ (For S275Steel)

$L_e = 2\,e_e + (n-1)\,p_e - n\,D_h$

$e_e = e_1$ but $\leq e_2$ = 40mm

$p_e = p$ but $\leq 2e_2$ = 70mm

$L_e = (2 \times 40) + (5 \times 70) - (6 \times 35)$ = 220mm

Tying capacity $= \dfrac{0.6 \times 220 \times 10 \times 275}{10^3}$ = 363kN

Tie force = 465kN $\not<$ 363kN — **∴ Fails**

However, using the rigorous approach of Appendix B, the tying capacity is 580kN. — Yellow pages Table H.14

Tie Force = 465kN < 580kN — **∴ O.K.**

Note: The tying capacity obtained by CHECK 11 is the same, for similar cleats, with a "single line of bolts" or a "double line of bolts". The standard connection values, obtained from Table H.14 of the yellow pages,are CHECK 11 critical values (whereas the values tabulated in Table H.13 are CHECK 12(ii) values for these particular standard connections).

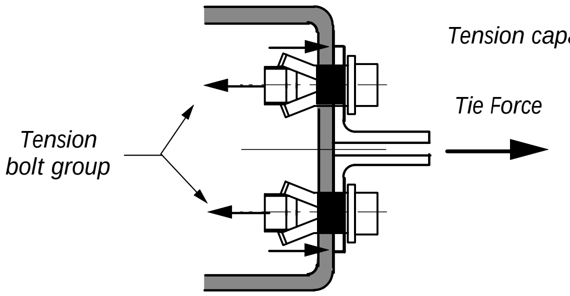
Title	Example 4 – Double Angle Cleats - Beam to RHS column using Hollo-bolt	Sheet	10 of 11

CHECK 12: Structural Integrity - Supported Beam

This check is identical to CHECK 12 in Example 2

CHECK 13 : Structural Integrity - Tension Capacity of Bolts

Basic requirement: **Tie Force** $\leq$ **Tension capacity of tension bolt group**



Tension capacity = $2\ n\ P_{si}$

Tie Force P_{si} = Hollo-Bolt Structural integrity Tensile Capacity — *Yellow pages Table H.56*

= 73kN for Hollo-Bolt

For 406 x 178 x 74 UB Side

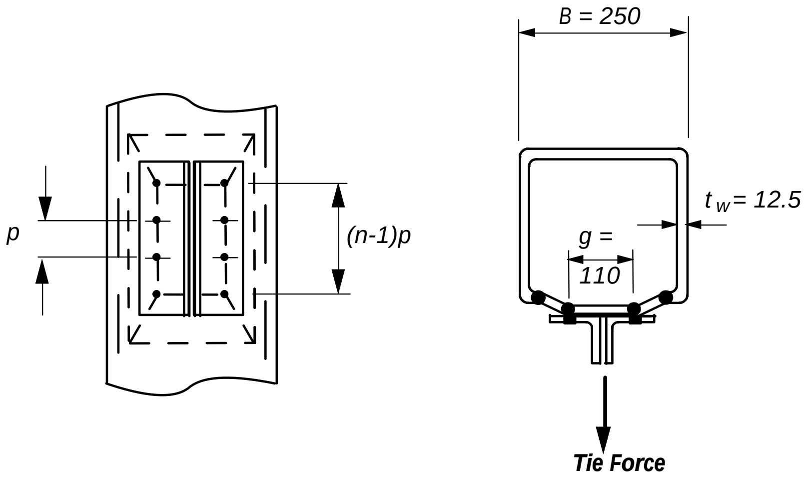
Tension capacity	=	2 x 4 x 73	=	584kN	
Tie Force	**=**	**300kN**	**<**	**584kN**	**∴ O.K.**

For 533 x 210 x 92 UB Side

Tension capacity	=	2 x 6 x 73	=	876KN	
Tie Force	**=**	**465kN**	**<**	**876kN**	**∴ O.K.**

CHECK 14: not applicable

CHECK 15: Structural Integrity – Capacity of Supporting Column Wall (RHS)



Basic requirement: **Tie Force** $\leq$ **Tying capacity of RHS column wall**

For 406 x 178 x 74 UB Side

$$\text{Tying capacity of column wall} = \frac{8\,M_u}{1-\beta_1}\left[\eta_1 + 1.5(1-\beta_1)^{0.5}\,(1-\gamma_1)^{0.5}\right]$$

Title	*Example 4 – Double Angle Cleats - Beam to RHS column using Hollo-bolt*	Sheet	*11 of 11*

M_u = moment capacity of RHS column wall per unit length

$$= \frac{p_u t_w^2}{4} = \frac{392 \times 12.5^2}{4 \times 10^3} = 15.3\text{kNm/mm}$$

D_h = 35mm (Hole diameter for Hollo-Bolt)

$$\eta_1 = \frac{(n-1)\ p - \frac{n}{2}\ D_h}{B - 3t_w}$$

$$= \frac{(4-1) \times 70 - \frac{4}{2} \times 35}{250 - 3 \times 12.5} = 0.659$$

$$\beta_1 = \frac{g}{B - 3t_w}$$

$$= \frac{110}{212.5} = 0.518$$

$$\gamma_1 = \frac{D_h}{B - 3t_w}$$

$$= \frac{35}{212.5} = 0.165$$

Tying capacity of column wall

$$= \frac{8 \times 15.3}{1 - 0.518} \left[0.659 + 1.5(1 - 0.518)^{0.5} \times (1 - 0.165)^{0.5} \right]$$

$$= 253.9 \left[0.659 + (1.041 \times 0.914) \right] = 409\text{kN}$$

Tie Force = 300kN < 409kN **∴ O.K.**

For 533 x 210 x 92 UB Side

$$\eta_1 = \frac{(n-1)\ p - \frac{n}{2}\ D_h}{B - 3t_w}$$

$$= \frac{(6-1)\ 70 - \frac{6}{2} \times 35}{250 - 3 \times 12.5} = 1.152$$

Tying capacity of column wall

$$= \frac{8\ M_u}{1 - \beta_1} \left[\eta_1 + 1.5(1 - \beta_1)^{0.5} (1 - \gamma_1)^{0.5} \right]$$

$$= \frac{8 \times 15.3}{1 - 0.518} \left[1.152 + 1.5(1 - 0.518)^{0.5} \times (1 - 0.165)^{0.5} \right]$$

$$= 253.9 \left[1.152 + (1.041 \times 0.914) \right] = 534\text{kN}$$

Tie Force = 465kN < 534kN **∴ O.K.**

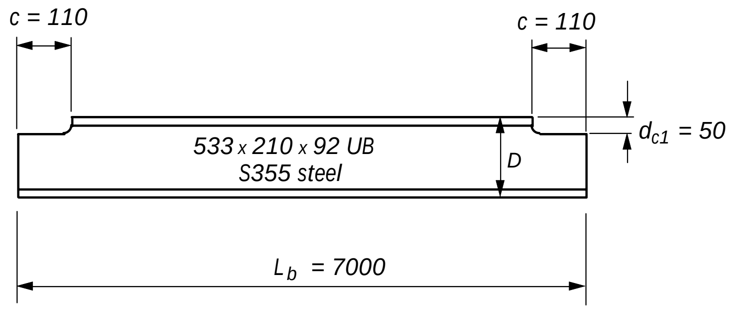
CALCULATION SHEET BCSA	Job: Joints in Steel Construction - Simple Connections		Sheet: 1 of 2
	Title: Example 5 - Double Angle Cleats - Stability of an unrestrained notched beam		
	Client: SCI/BCSA Connections Group		
	Calcs by: RS	Checked by: ASM	Date: May 2002

CHECK 7: Unrestrained Supported Beam - Overall Stability of Notched Beam

(Unrestrained against lateral torsional buckling)

Basic requirement: *Beams should be checked for lateral torsional buckling to BS 5950-1 clause 4.3.6 with a modified effective length (L_E) which takes account of notches.*

The following approach is only valid for $c/L_b < 0.15$ and $d_{c1}/D < 0.2$ (beams with notches outside these limits should be checked as tee sections, or stiffened)



Check the beam shown, which was designed as unrestrained over the span with an effective length of $1.0L_b$, when it has a single notch at each end, and a design moment of 200kNm due to UDL.

Beam 533 x 210 x 92 UB grade S355 $\quad M_x = 200\text{kNm}$

$S_x = 2360\text{cm}^3$

$r_y = 4.51\text{cm}$

$x = 36.4$

$u = 0.873$

} properties obtained from published tables or derived from BS 5950-1 — SCI - P202 'Blue Book'

$v = 1$ } conservative

$$c/L_b = \frac{110}{7000} = 0.016 < 0.15 \quad \therefore \text{OK}$$

$$d_c/D = \frac{50}{533.1} = 0.094 < 0.2 \quad \therefore \text{OK}$$

Modified effective length, L_E

$$L_E = L_b \left[1 + \frac{2c}{L_b}(K^2 + 2K)\right]^{1/2}$$

$$K = K_o/\lambda_b$$

$$\lambda_b = \frac{u\, v\, L_b}{r_y}$$

$$\lambda_b = \frac{0.873 \times 1 \times 7000}{45.1} = 135.5$$

$\lambda_b > 30 \quad \therefore K_o = g_o x \quad \text{but} \leq K_{max}$

Title Example 5 - Double Angle Cleats - Stability of an unrestrained notched beam	Sheet 2 of 2

Note: For values of $^{c}/_{L_b}$ < 0.025 use: g_o = 5.56 and K_{max} = 260

K_o = 5.56 x 36.4 = 202.4 < K_{max}

$\therefore K = \dfrac{202.4}{135.5} = 1.49$

and $L_E = 7000\left[1 + \dfrac{(2 \times 110)}{7000} \times ((1.49^2 + (2 \times 1.49))\right]^{1/2}$

= 7550mm

BS 5950-1 4.3.6.7

$\lambda_{LT} = u\, v\, \lambda \sqrt{\beta_w}$

β_w = 1 for class 1 cross-section

$\lambda = L_E / r_y = \dfrac{7550}{45.1} = 167$

$\lambda_{LT} = 0.873 \times 1 \times 167 = 146$

$p_y = 355\text{N/mm}^2$

$p_b = 74\text{N/mm}^2$

$M_b = p_b S_x$ (for class 1 cross-section)

$= \dfrac{74 \times 2360}{10^3} = 175\text{kNm}$

BS 5950-1 p_b - Table 16 M_b - 4.3.6.4

Basic requirement: $\mathbf{M_x \leq \dfrac{M_b}{m_{lt}}}$ **and** $\mathbf{M_x \leq M_{cx}}$

BS 5950-1 4.3.6.2

M_{cx} = 838kNm

SCI - P202 Page D-59

m_{Lt} = 0.925 for UDL

BS 5950-1 Table 18

$\therefore \dfrac{M_b}{m_{Lt}} = \dfrac{175}{0.925} = 189\text{kNm}$

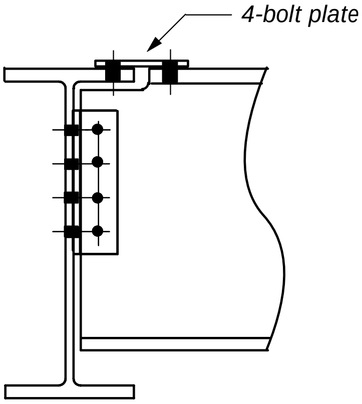
$\mathbf{M_x = 200kNm \nleq 189kNm}$ **∴ Fails**

SOLUTIONS: (1) Increase beam size

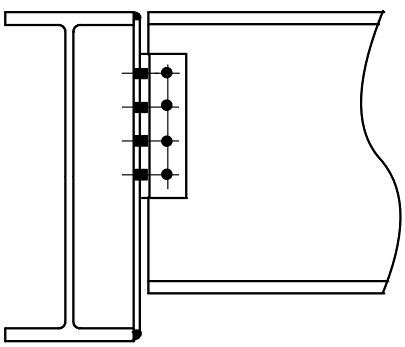
or (2) Provide plan bracing to the beam to reduce unrestrained span.

or (3) Provide restraints between the top flange and the supporting beam flange as shown below:

or (4) Eliminate the need for notches as shown below:



(3)



(4)

5. FLEXIBLE END PLATES

5.1 INTRODUCTION

Typical flexible end plate connections are shown in Figure 5.1. The end plate, which may be full or partial depth, is welded to the supported beam in the workshop. The beam is then bolted to the supporting, beam, column or RHS column on site. Flowdrill or Hollo-Bolts are used for connections to RHS columns.

End plates are probably the most popular of the simple beam connections currently in use in the U.K. They can be used with skewed beams and can tolerate moderate offsets in beam to column joints.

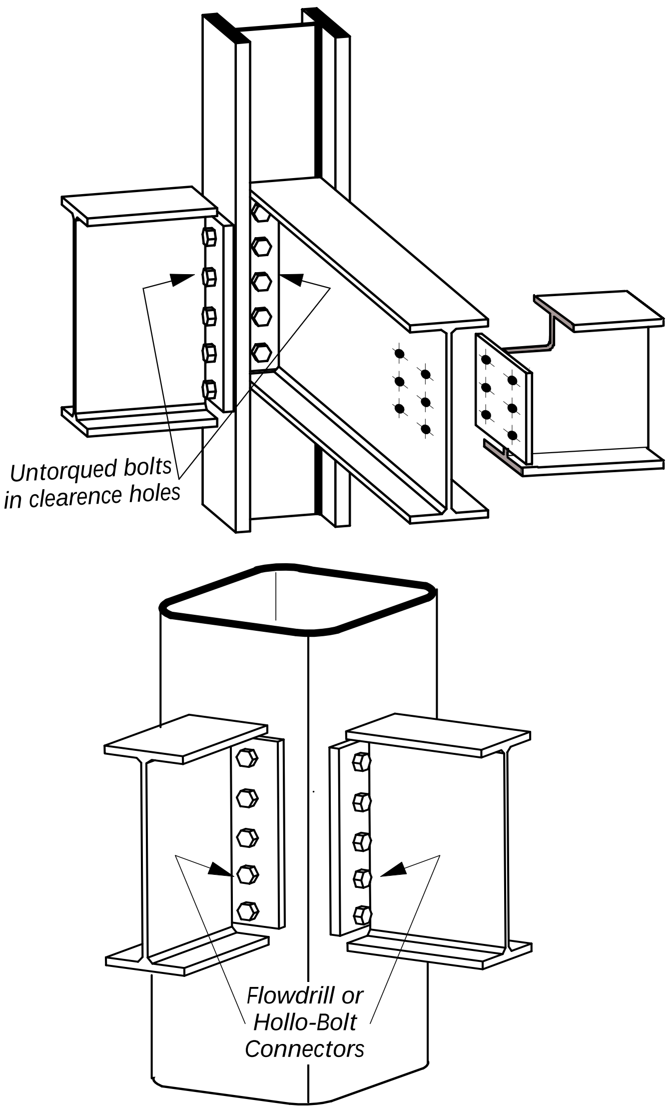


Figure 5.1 Flexible end plate. Beam-to-column and beam-to-beam connections

5.2 PRACTICAL CONSIDERATIONS

Partial depth flexible end plates

Normal practice is for partial depth end plate to be welded to the beam web only, usually with 6mm fillet welds, although these may need to be increased to 8mm or even 10mm, particularly with deeper beams and high grade steels. The weld should not be continued across the top and bottom of the plate. Care must be taken during welding to avoid undercutting the beam web as shown in Figure 5.2.

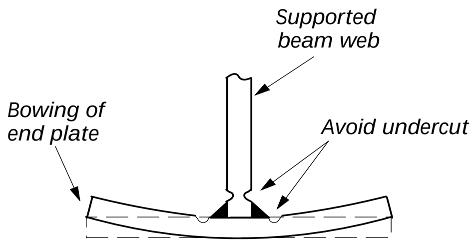


Figure 5.2 Undercutting and weld distortion

It is also quite common, particularly with the thinner plates, to experience bowing of the plate due to weld distortion. Moderate curvature of the plate in this way should not be a problem as the joint will usually be pulled together during erection as the bolts are tightened.

Two sizes of end plate have been recommended for use in this Guide. A 150 x 8mm flat with bolts at 90mm cross centres will generally be adequate for beams up to 457mm deep. For deeper beams a 200 x 10mm flat can be used with a 140mm bolt gauge.

In practice there will be occasions when it is not possible to adhere rigidly to the above guidelines. For example, two sided connections framing into a beam or column web must clearly be detailed with a common bolt gauge and large beams connecting into a 152 UC or 203 UC column web will have to be fitted with a narrower end plate.

Full depth flexible end plates

End plates which extend the full depth of the beam, and welded to the beam flanges, may also be used as shown in Figure 5.3.

For shallower beam and column sections when used as beams a full depth plate with an inside profile weld to the web and flanges can potentially have a capacity equal to the full shear capacity of the beam. Provided that thin plates are used, then this practice should not invalidate the design model.

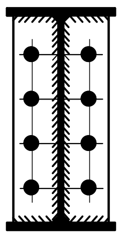


Figure 5.3 Full depth flexible end plate

If the recommended detailing practice given in Check 1 is followed then a full depth end plate has sufficient flexibility to be considered a simple connection. If a stiffer arrangement is provided the connection begins to move into the semi-rigid classification when consideration should be given to both the additional moment that may be transferred into the columns and to the required rotation capacity within the connection.

Research work has indicated that at the ultimate limit state these additional moments are redistributed back into the beams. This phenomenon of moment shedding has been researched at Imperial College, London[22] and at The University of Sheffield[23]. This second study highlights the situations where these moments may safely be ignored.

Using the design rules given in Section 5.5, the connection moment (which is indeterminate but small) can be neglected.

Connections with thicker end plates are discussed in another publication in this series, *Moment Connections.*[24]

Erection

Unlike double angle web cleats, this connection has little facility for site adjustment. To avoid the accumulation of tolerances over a number of beams, a slightly shorter beam with various thicknesses of packs should be detailed at regular intervals, for example every 5th beam of a continuous run. The number of packs should be kept to a minimum (less than 4) and allowance made for the loss in shear capacity of the bolts.

Difficulties can also be encountered on site with two sided connections where a pair of beams either side of a column web share a common set of bolts. For larger beams it may be advisable to provide some form of support during erection as shown in Figure 5.4.

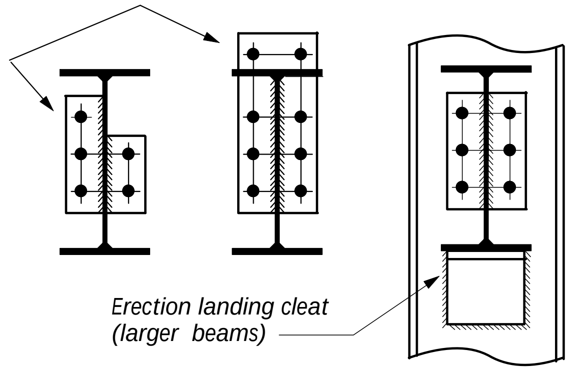


Figure 5.4 Erection aids

5.3 RECOMMENDED GEOMETRY

The design procedures which follow set down a number of recommended details which are intended to achieve the required flexibility.

When detailing the joint, the main requirements are as follows:-

(i) the end plate is positioned close to the top flange in order to provide positional restraint;

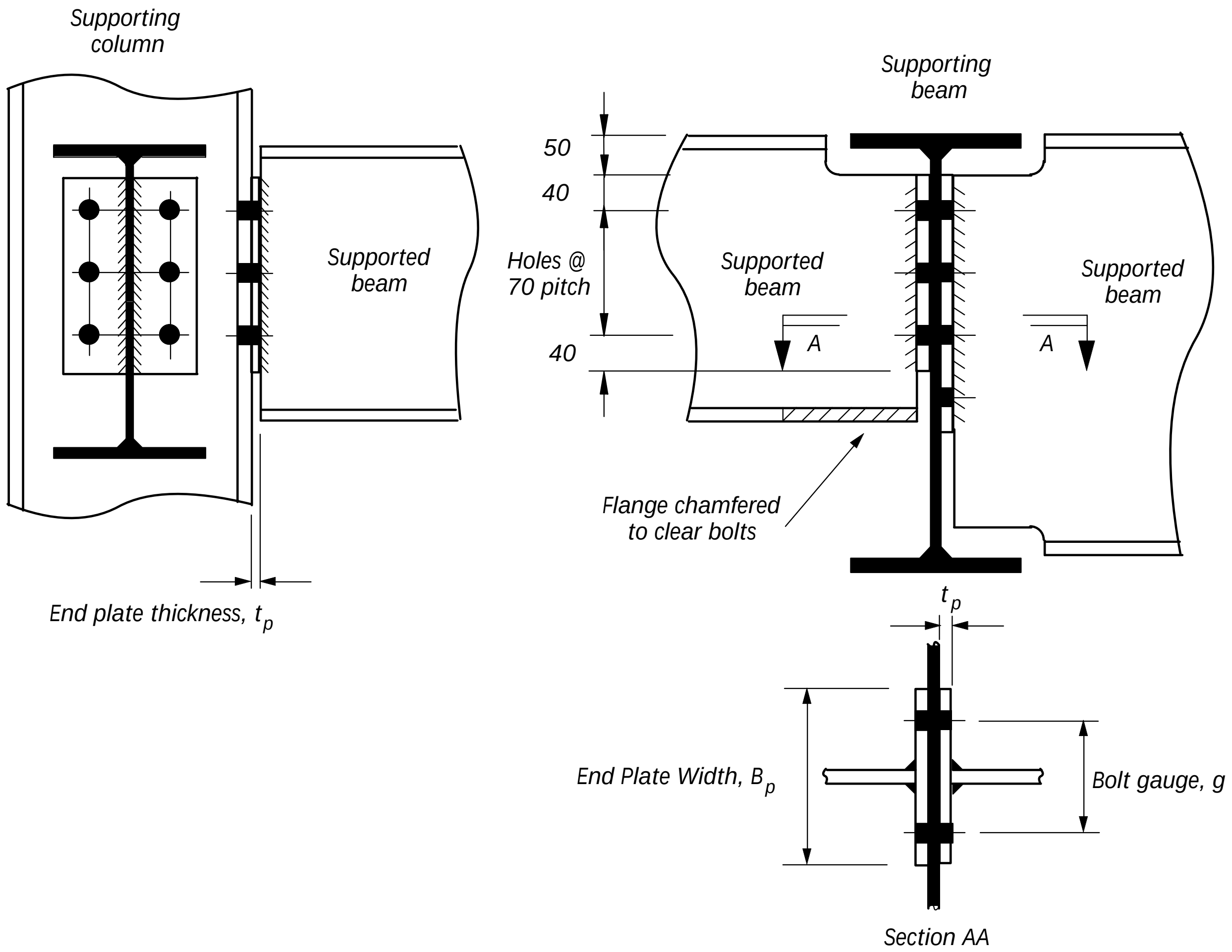
(ii) the end plate depth is least 0.6 x the supported beam depth in order to provide the beam with adequate torsional restraint;

(iii) the end plate is relatively thin (8mm or 10mm);

(iv) the bolts are at reasonable cross centres (90mm or 140mm).

The first two requirements ensure that in those cases where the beam is laterally unrestrained, it can be designed with an effective length of 1.0L. (BS 5950-1: Table 13)[1] The last two requirements ensure adequate flexibility and ductility to classify as "simple connections".

These requirements, together with the standard geometry presented in Section 2, have been used to create the 'standard connection' shown in Figure 5.5.



Beam-to-column **Beam-to-beam**

Supported Beam	Recommended End Plate Size $B_p \times t_p$	Bolt Gauge g
up to 457	150 x 8	90
533 and above	200 x 10	140

Bolts: M20 8.8 in 22mm diameter holes

End Plate: Steel grade S275, minimum length 0.6D where D is depth of supported beam

Figure 5.5 Standard partial depth end plate connections

5.4 DESIGN

The full design procedure is presented in Section 5.5.

The flexible end plate is the strongest of the three beam connections dealt with in the guide, having a vertical shear capacity of around 50% to 70% of the beam for the partial depth end plate and up to 100% for the full depth end plate.

If the standard geometry shown in Figure 5.5 and Table H.18 is adopted, it will be found that the connection shear capacity is generally governed:

(a) for ordinary/Flowdrill bolts: either by shear in the beam web, (Check 4) or shear in the bolts (Check 8),

(b) For Hollo-Bolts: either by shear in the beam web (Check 4) or shear in the end plate (Check 9).

The general behaviour of the connection is shown in Figure 5.6 For partial depth end plates there are basically two stages:

(i) the unhindered rotation of the connection until,

(ii) the lower beam flange bears against the support.

In a beam-to-beam connection where the supporting beam is free to rotate, there will be adequate rotational capacity in the connection even with a thick end plate.

Where the supporting beam is not free to rotate the rotational capacity must be provided by the connection. In this instance, the designer should be aware that thick, full depth end plates may lead to large prying forces and overstressing of the bolts and welds.

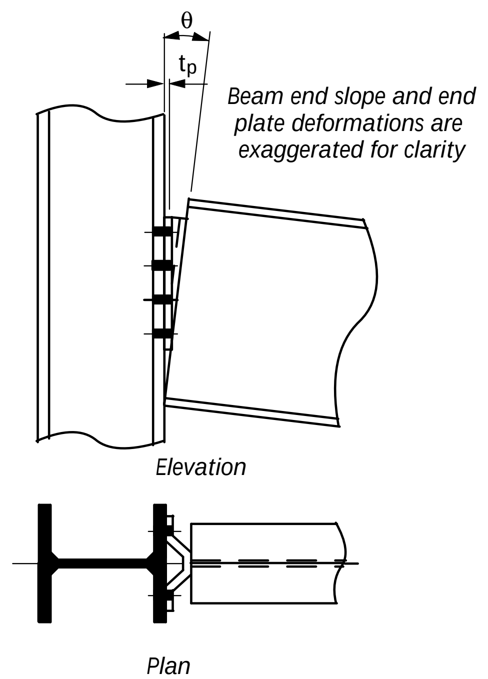


Figure 5.6
Behavior of flexible end plate connections

Structural integrity

All floor beam-to-column connections must be designed to resist a tying force of at least 75 kN, a force which can be carried by the simplest of end plates *(see Appendix A)*. For certain multi-storey buildings it will be necessary to check the connection for large tying forces to satisfy the structural integrity requirements of BS 5950-1.[1]

The tying capacity of end plate connections is generally lower than that of web cleats or fin plates. As can be seen from the capacity tables in the yellow pages, the tying capacity of the standard plate can be as low as 35% of the connection's shear capacity, although for smaller beams this rises to a value of around 85%.

For connections to I-Column flanges the critical mode of failure will be found to be design check 11, the tension capacity of the end plate. To achieve a better tying capacity, the solution will generally involve either increasing the plate thickness to 10 or 12mm or alternatively reducing the bolt gauge (cross centres). Another method of accommodating tying forces is by means of steel reinforcement in the concrete floors carrying all or part of the tie force back to the steel frame.

Worked examples

Four worked examples are provided in Section 5.6 to illustrate the full set of design checks of Section 5.5.

Connection capacity tables

Capacity tables for flexible end plate connections using ordinary or Flowdrill bolts are given in Tables H.20 and H.21 and using Hollo-Bolts in Tables H.22 and H.23 in the yellow pages. The tables cover both S275 and S355 beams and are detailed in accordance with the standard geometry presented in Figure 5.5 and Table H.18.

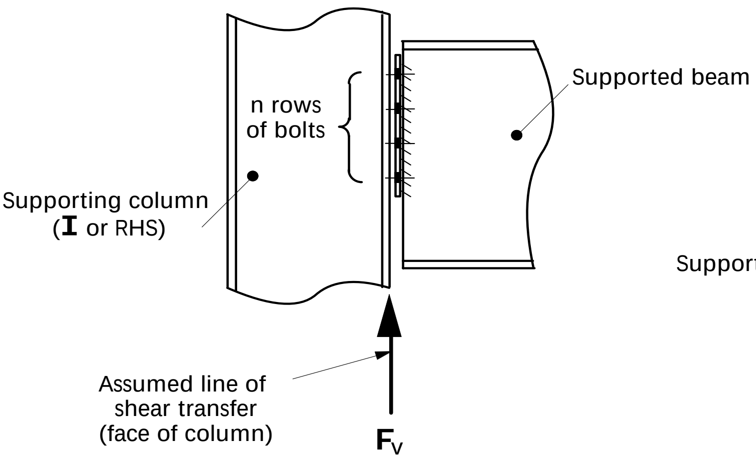
Values of the connection shear and tying capacities are tabulated together with simple aids to check the support and the beam notch (if applicable).

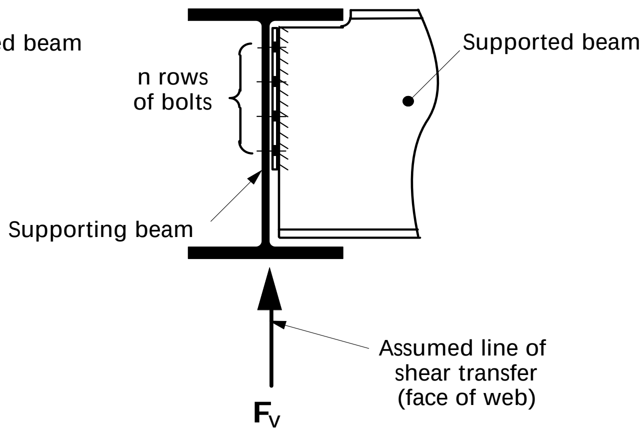
5.5 DESIGN PROCEDURES

Recommended design model

Any simple equilibrium analysis is suitable for design. The one used in this publication is in accordance with traditional UK design practice and is based on the simply supported beam end reaction.

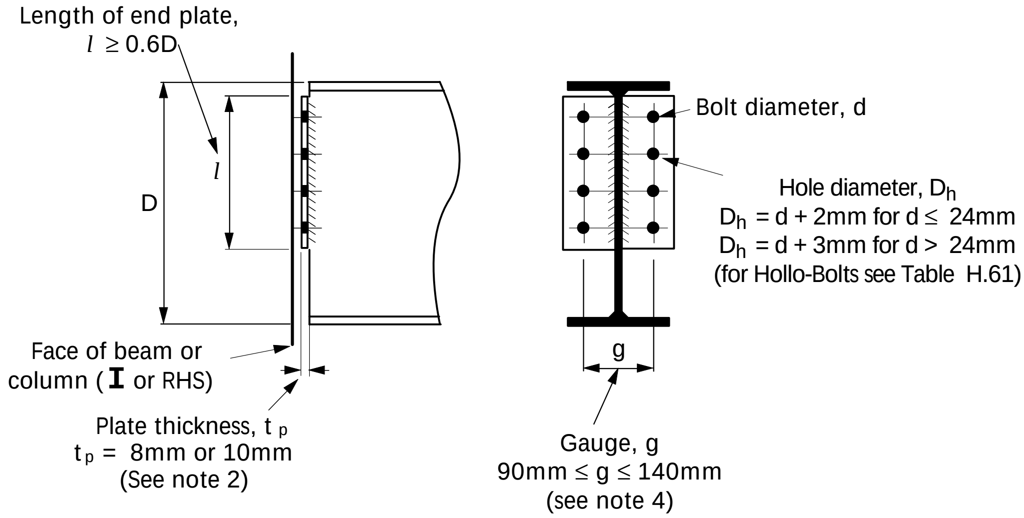
The design procedure applies to beams connected to the column flange, the column web, the supporting beam web or to RHS columns. Although the figures show partial depth end plates the design procedures apply equally to full depth end plates.

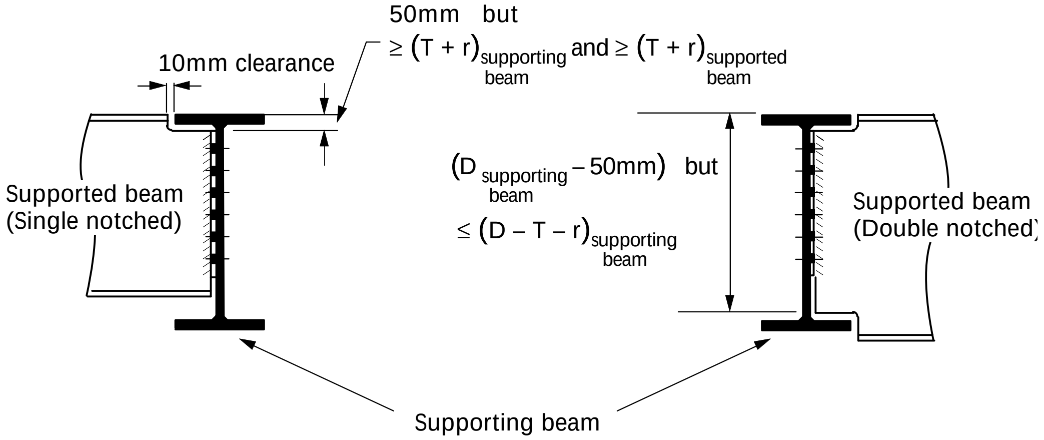




Check 1	**Recommended detailing practice**		
Check 2	**Supported beam**	–	Welds
Check 3	*Not applicable*		
Check 4	**Supported beam**	–	Capacity at the connection
Check 5	**Supported beam**	–	Capacity at a notch
Check 6	**Supported beam**	–	Local stability of notched beam
Check 7	**Unrestrained supported beam**	–	Overall stability of notched beam
Check 8	**Supporting beam/column**	–	Bolt group
Check 9	**Supporting beam/column**	–	Connecting elements
Check 10	**Supporting beam/column**	–	Local capacity
Check 11	**Structural integrity**	–	Connecting elements
Check 12	**Structural integrity**	–	Supported beam
Check 13	**Structural integrity**	–	Tension bolt group/welds
Check 14	**Structural integrity**	–	Supporting column web (UC or UB)
Check 15	**Structural integrity**	–	Supporting column wall (RHS)
Check 16	*Not applicable*		

CHECK 1 — Recommended detailing practice





Notes

(1) The end plate is generally positioned close to the top flange of the beam to provide adequate positional restraint. Plate length of at least 0.6D is usually adopted to give "nominal torsional restraint" (BS 5950-1, Table 13 and clause 4.2.2).

(2) Although it may be possible to satisfy the design requirements with t_p < 8mm, it is not recommended in practice because of the likelihood of weld distortion during fabrication and damage during transportation. If structural integrity checks are critical the end plate thickness may be increased to 10mm or 12mm

(3) The plate thickness and gauge limitations apply equally to partial depth and to full depth end plates.

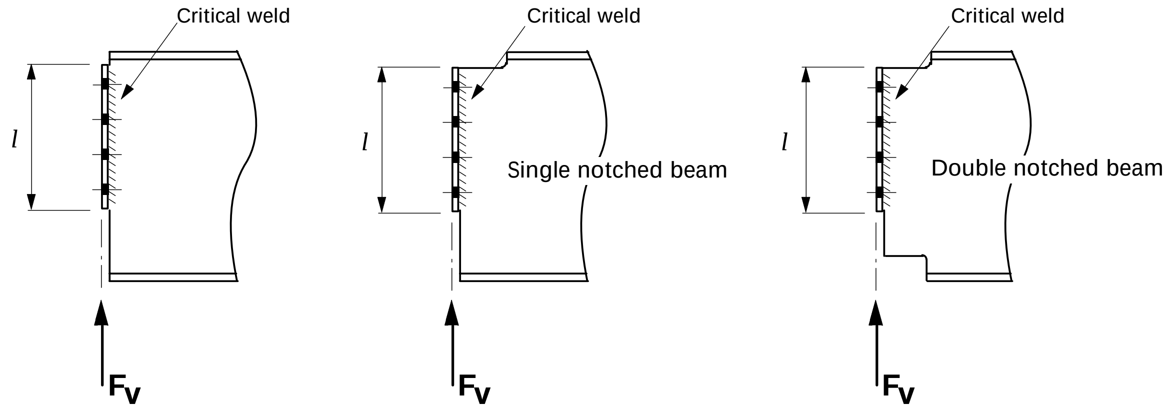
(4) In addition, for connections to RHS columns, g should be at least 0.3 x face width.

(5) Detail requirements for Flowdrill and Hollo-Bolt connections to RHS columns should also comply with Tables H.60 and H.61 of the yellow pages.

In beam-to-**I** section column flange connections, where it is required to comply with structural integrity requirements for a tie force of 75kN, the connection must have at least 2 no. M20, 8.8 bolts in tension with $l \geq$ 140mm, $t_c \geq$ 8mm and g ≤ 140mm.

For greater tie forces and other connections (e.g. to column webs and RHS), checks 11 to 15, as appropriate, should be carried out.

CHECK 2	Supported beam - Welds



Capacity of fillet welds connecting end plate to beam web

Basic requirement:

$F_v \leq P_{weld}$

P_{weld} = capacity of the fillet weld

= $p_w\, l_{we}\, a$

p_w = design strength of the weld (BS 5950-1, Table 37)

where:

l_{we} = total effective length of the weld to beam web (BS 5950-1, cl. 6.8.2)

= $2\,(l - 2s)$

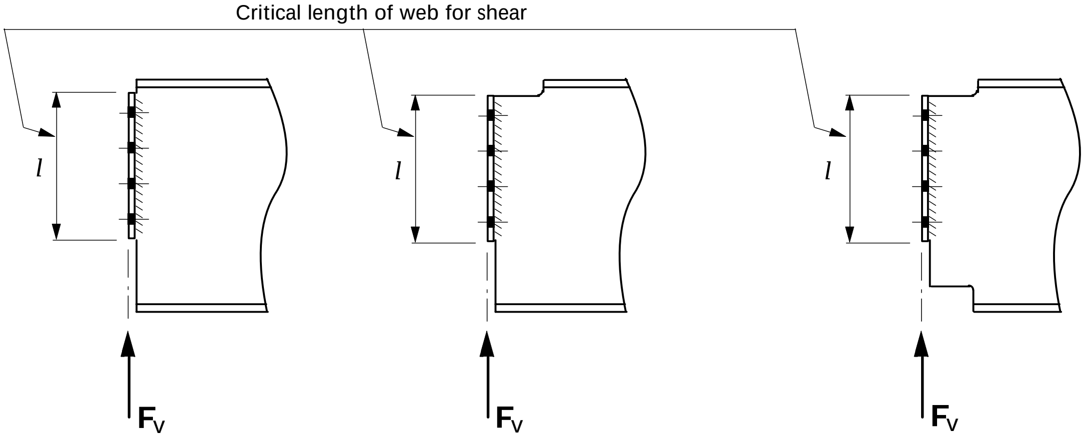
a = effective throat size of the weld

= 0.7s (normally)

s = leg length of the fillet weld

CHECK 3	***Not applicable (see Table 3.1)***

CHECK 4	Supported beam - Capacity at connection



Shear capacity of the beam web at the end plate

Basic requirement:

$F_v \leq P_v$

P_v = shear capacity of the beam connected to the end plate

$P_v = 0.6\, p_y\, A_v$

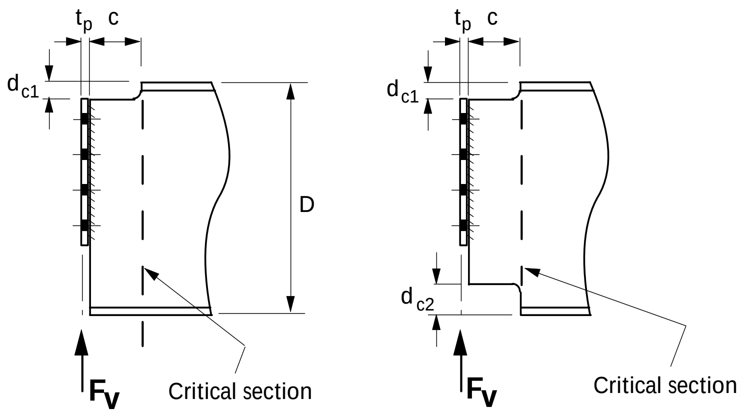
where:

$A_v = 0.9\, l\, t_w$

(BS 5950-1, cl. 4.2.3)

t_w = thickness of beam web

CHECK 5	Supported beam - Capacity at a notch



Shear and bending interaction at the notch:

Basic requirement:

$$F_v\,(t_p + c) \le M_{cN}$$

M_{cN} for Single notched beam:

For low shear (i.e. $F_v \le 0.75P_{vN}$)

$$M_{cN} = p_y Z_N$$

For high shear (i.e. $F_v > 0.75P_{vN}$)

$$M_{cN} = 1.5 p_y Z_N \left(1 - \left(\frac{F_v}{P_{vN}}\right)^2\right)^{1/2}$$

M_{cN} for Double notched beam:

For low shear (i.e. $F_v \le 0.75P_{vN}$)

$$M_{cN} = \frac{p_y t_w}{6}\left(D - d_{c1} - d_{c2}\right)^2$$

For high shear (i.e. $F_v > 0.75P_{vN}$)

$$M_{cN} = \frac{p_y t_w}{4}\left(D - d_{c1} - d_{c2}\right)^2 \left(1 - \left(\frac{F_v}{P_{vN}}\right)^2\right)^{1/2}$$

where:

M_{cN} = moment capacity of the beam at the notch in the presence of shear.

P_{vN} = shear capacity at the notch
= $0.6\, p_y A_{vN}$

A_{vN} = $(D - d_{c1})\, t_w$
(for single notched beam)

A_{vN} = $0.9\,(D - d_{c1} - d_{c2})\, t_w$
(for double notched beam)

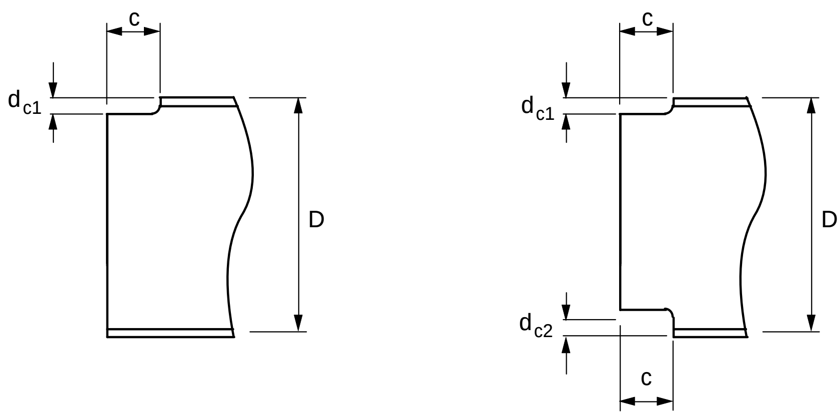
t_p = thickness of end plate

t_w = thickness of beam web

c = length of notch

Z_N = elastic section modulus of the tee section at the notch

CHECK 6	Supported beam - Local stability of notched beam



When the beam is <u>restrained</u> against lateral torsional buckling, no account need be taken of notch stability provided the following conditions are met:

For one flange notched [14],[I5]

Basic requirement:

d_{c1}	$\leq$	D/2 **and:**			
c	$\leq$	D	for	$D/t_w \leq 54.3$	(S275 steel)
c	$\leq$	$\frac{160000D}{(D/t_w)^3}$	for	$D/t_w > 54.3$	(S275 steel)
c	$\leq$	D	for	$D/t_w \leq 48.0$	(S355 steel)
c	$\leq$	$\frac{110000D}{(D/t_w)^3}$	for	$D/t_w > 48.0$	(S355 steel)

For both flanges notched [15]

Basic requirement:

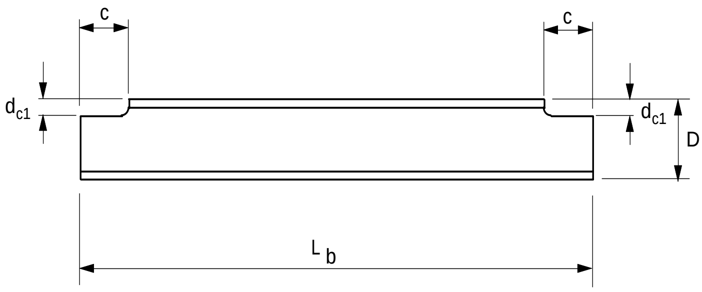
Max (d_{c1} , d_{c2})	$\leq$	D/5 **and:**			
c	$\leq$	D	for	$D/t_w \leq 54.3$	(S275 steel)
c	$\leq$	$\frac{160000D}{(D/t_w)^3}$	for	$D/t_w > 54.3$	(S275 steel)
c	$\leq$	D	for	$D/t_w \leq 48.0$	(S355 steel)
c	$\leq$	$\frac{110000D}{(D/t_w)^3}$	for	$D/t_w > 48.0$	(S355 steel)

where:

t_w = thickness of supported beam web

Where the notch length c exceeds these limits, either suitable stiffening should be provided or the notch should be checked to references 14, 15 and 16.

CHECK 7	Unrestrained supported beam Overall stability of notched beam



When a notched beam is unrestrained against lateral torsional buckling, the overall stability of the beam should be checked.

Notes:

(1) This check is only applicable for beams with one flange notched. Guidance on double-notched beams is given in Section 5.12 of Reference 17.

(2) If the notch length c and/or notch depth d_{c1} are different at each end, then the larger values for c and d_{c1} should be used.

(3) Beams should be checked for lateral torsional buckling to BS 5950-1[1], clause 4.3 with a modified effective length (L_E) which takes account of notches.

(4) The solution below gives the modified effective length (L_E) based on references 18, 19 and 20. It is only valid for $c/L_b < 0.15$ and $d_{c1}/D < 0.2$ (beams with notches outside these limits should be checked as tee sections, or stiffened).

$$L_E = L_b \left(1 + \frac{2c}{L_b}(K^2 + 2K)\right)^{1/2}$$

$$K = K_o/\lambda_b$$

$$\lambda_b = \frac{u\,v\,L_b}{r_y}$$

where: x, u, v and r_y are for the un-notched **I** beam section and are defined in BS 5950-1
Conservatively, $u = 0.9$ and $v = 1.0$

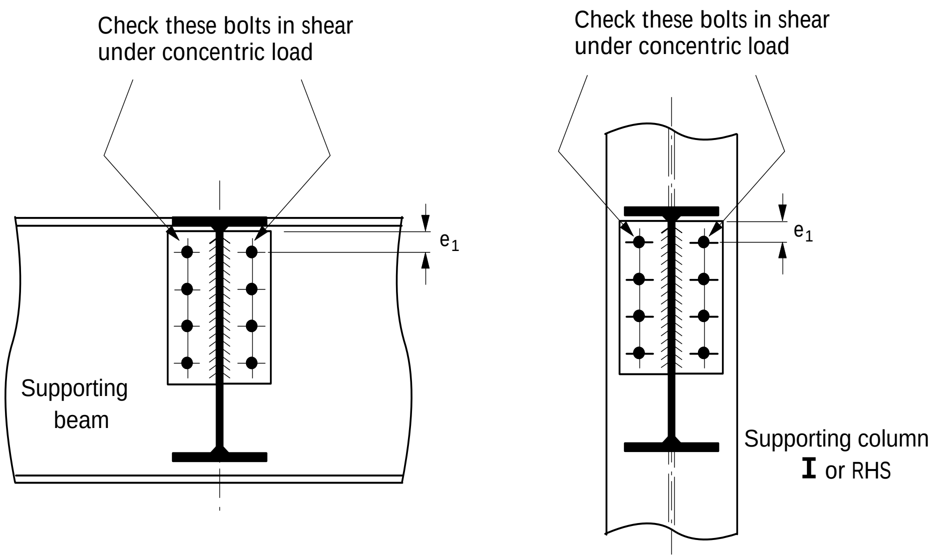
for $\lambda_b < 30$ $K_o = 1.1\, g_o\, x$ but $\leq 1.1\, K_{max}$

for $\lambda_b \geq 30$ $K_o = g_o\, x$ but $\leq K_{max}$

g_o and K_{max} are tabulated below:

$\frac{c}{L_b}$	g_o	K_{max}	
		UB section	UC section
≤ .025	5.56	260	70
.050	5.88	280	80
.075	6.19	290	90
.100	6.50	300	95
.125	6.81	305	95
.150	7.13	315	100

CHECK 8	Supporting beam/column - Bolt group



Shear capacity of bolt group connecting end plate to supporting beam or column

Basic requirement:

$F_v \quad \leq \quad \Sigma P_s$

$P_s \quad = \quad$ shear capacity of single bolt

$\quad = \quad p_s A_s$*

but for the top pair of bolts, P_s is the smaller of:

$p_s A_s$* or $0.5\, k_{bs} e_1\, t_p\, p_{bs}$

where:

p_s = shear strength of a bolt

A_s = shear area of a bolt

t_p = thickness of end plate

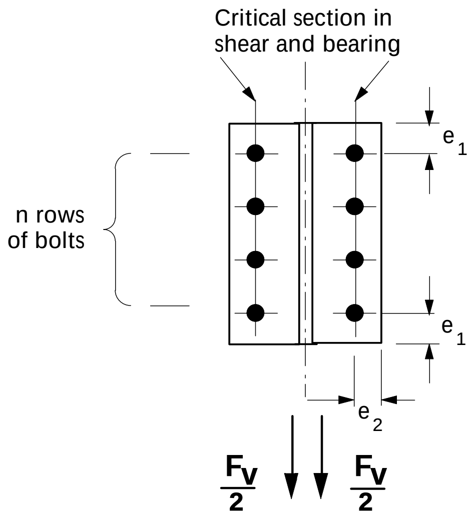
p_{bs} = bearing strength of end plate

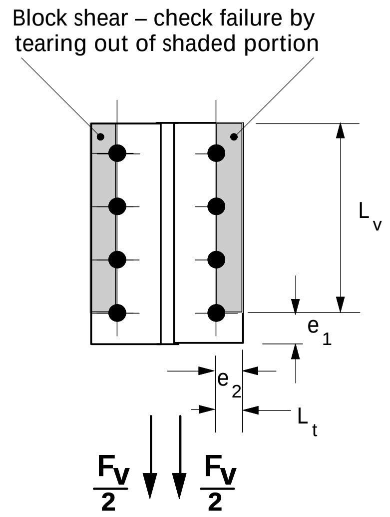
e_1 = end distance

k_{bs} = 1.0 for standard clearance holes, Flowdrill and Hollo-Bolts

* For Hollo-Bolts $p_s A_s$ should be taken as the shear capacity as given in Table H.56 of the yellow pages

CHECK 9	Supporting beam/column- Connecting elements **(End plate adjacent to supporting beam or column)**





Shear and bearing capacity of end plate connected to supporting beam or column

(i) For shear:

Basic requirement:

$F_v/2 \leq P_{v.min}$

$P_{v.min}$ = shear capacity of the end plate

= smaller of Plain shear capacity P_v and Block shear capacity P_r

Plain shear

$P_v = \min(0.6\, p_y A_v,\ 0.7\, p_y K_e A_{v.net})$

$A_v = 0.9\,(2e_1 + (n-1)\,p)\,t_p$

$A_{v.net} = A_v - n\,D_h\,t_p$

Block shear

$P_r = 0.6\, p_y\, t_p\big(L_v + K_e(L_t - kD_h)\big)$

$L_v = e_1 + (n-1)p$

$k = 0.5$

(ii) For bearing:

Basic requirement:

$F_v/2 \leq \Sigma P_{bs}$

ΣP_{bs} = bearing capacity of end plate (ie. for 'n' bolts)

P_{bs} = bearing capacity of end plate per bolt

$= k_{bs} d\, t_p\, p_{bs}$

but for the top bolt, P_{bs} is the smaller of:

$k_{bs} d\, t_p\, p_{bs}$ or $0.5\, k_{bs}\, e_1\, t_p\, p_{bs}$

where:

p = bolt pitch

d = diameter of bolt *

D_h = diameter of hole *

t_p = thickness of plate

p_{bs} = bearing strength of end plate

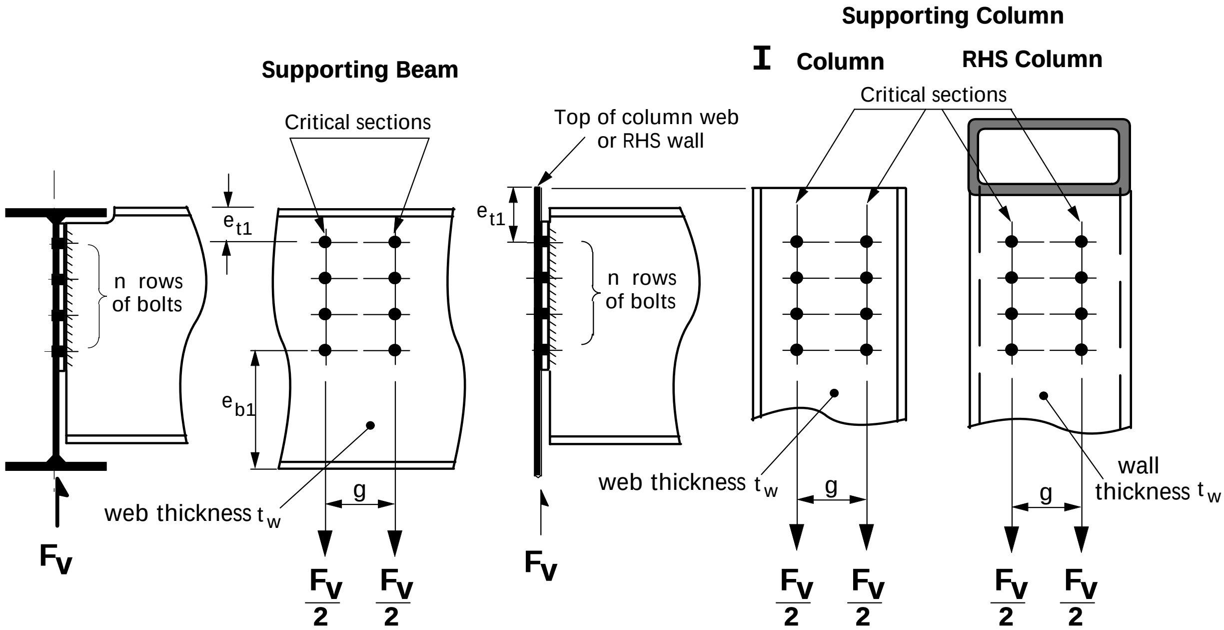
e_1 = end distance

K_e = 1.2 for S275 steel
= 1.1 for S355 steel

k_{bs} = 1.0 for standard clearance holes, Flowdrill and Hollo-Bolts

* For Hollo-Bolts d is the nominal bolt diameter and D_h is the hole diameter in the plate as given in Table H.61 of the yellow pages.

CHECK 10	Supporting beam/column - Local capacity (with one supported beam)



Local shear and bearing capacity of supporting beam web or column web or RHS wall for one supported beam

(i) For shear:

Basic requirement:

$F_v/2 \leq P_v$

P_v = local shear capacity of supporting beam web or **I** column web or RHS column wall

$P_v = \min(0.6\, p_y A_v,\ 0.7\, p_y K_e A_{v.net})$

$A_v = (e_t + (n-1)\,p + e_b)\,t_w$

$A_{v.net} = A_v - n D_h t_w$

(ii) For bearing:

Basic requirement:

$\frac{F_v}{2\,n} \leq P_{bs}$

P_{bs} = bearing capacity of supporting beam or column per bolt

$= k_{bs} d\, t_w\, p_{bs}$

p_{bs} = bearing strength of supporting beam or column

where:

e_t = smaller of e_{t1} and 5d

e_b = smallest of e_{b1}, g/2 and 5d (for supporting beam)

= smaller of g/2 and 5d (for supporting column)

p = bolt pitch

d = diameter of bolt *

D_h = diameter of hole *

t_w = thickness of supporting beam web or column web or RHS wall

K_e = 1.2 for S275 steel

= 1.1 for S355 steel

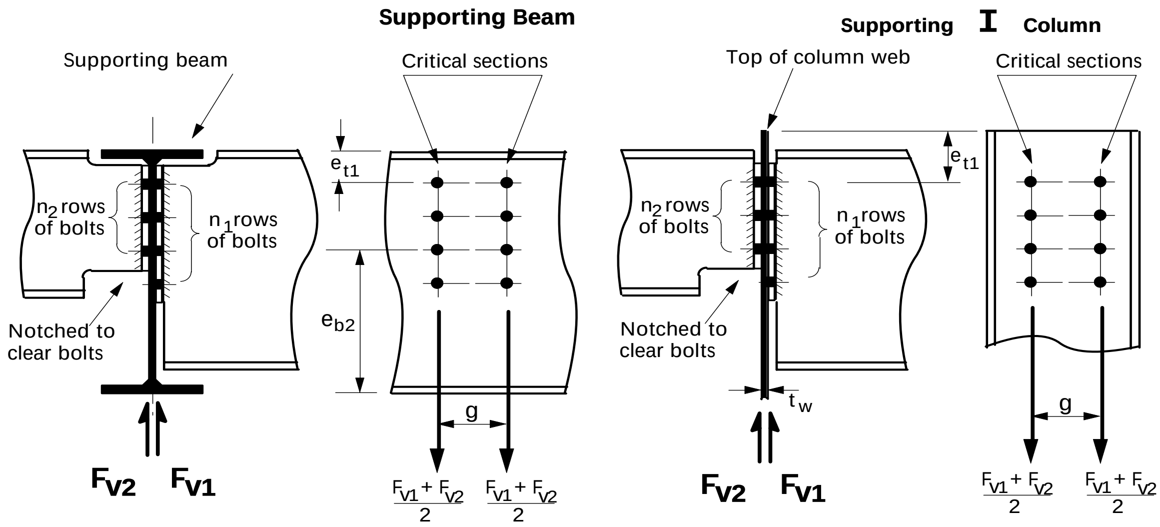
k_{bs} = 1.0 for standard clearence holes, Flowdrill and Hollo-Bolts

* For Hollo-Bolts d is the nominal bolt diameter and D_h is the hole diameter in the RHS as given in Table H.61 of the yellow pages.

For Flowdrill the diameter of the hole is the bolt diameter.

Note: The above check (i) is for local shear only; the effects of any global shear forces must also be considered.
If the beam is connected to a rolled column flange, and the thickness of the column flange is less than the thickness of the cleat then the bearing capacity of the flange should also be checked.

CHECK 10 (continued)	Supporting beam/column - Local capacity **(with two supported beams)**



Local shear and bearing capacity of supporting beam web or column web for two supported beams

(i) For shear:

Basic requirement:

$$\frac{F_{v1A}}{2} + \frac{F_{v2}}{2} \leq P_v$$

F_{v1A} = $F_{v1}\,\dfrac{n_2}{n_1}$

P_v = local shear capacity of supporting beam web or column web

P_v = min $(0.6\,p_y\,A_v\,,\ 0.7\,p_y K_e\,A_{v.net})$

A_v = $(e_t + (n_2 - 1)\,p + e_b)\,t_w$

$A_{v.net}$ = $A_v - \ n_2\,D_h\,t_w$

(ii) For bearing:

Basic requirement:

$$\frac{F_{v1}}{2n_1} + \frac{F_{v2}}{2n_2} \leq P_{bs}$$

P_{bs} = bearing capacity of supporting beam or column per bolt

= $d\,t_w\,p_{bs}$

p_{bs} = bearing strength of supporting beam or column

where:

e_t = smaller of e_{t1} and 5d

e_b = smallest of e_{b2}, g/2, p and 5d (for supporting beam)

= smallest of g/2, p and 5d (for supporting column)

p = bolt pitch

d = diameter of bolt

D_h = diameter of hole

t_w = thickness of supporting beam web or column web

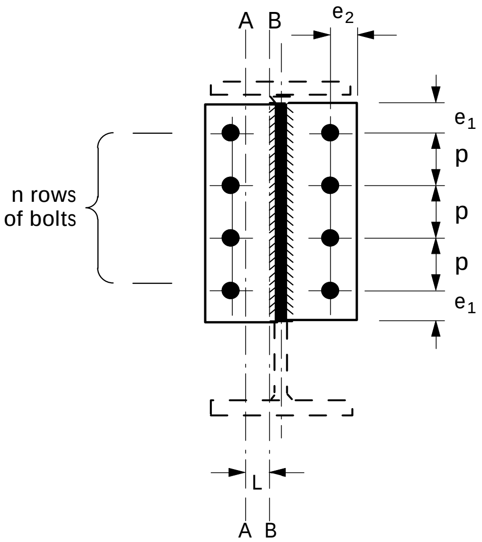
K_e = 1.2 for S275 steel

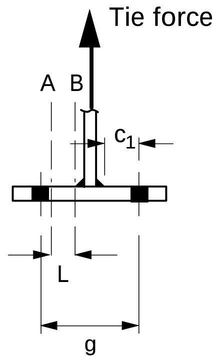
= 1.1 for S355 steel

Note: The above check (i) is for local shear only; the effects of any global shear forces must also be considered.

CHECK 11

Structural integrity - connecting elements





Note: This check is only needed if it is necessary to comply with structural integrity requirements

Structural integrity – tension capacity of end plate

Basic requirement:

Tie force $\leq$ Tying capacity of end plate

Tying capacity of end plate $= \dfrac{2(M_{uA} + M_{uB})}{L}$ (see Appendix C)

M_{uA} = moment capacity of end plate at section AA

$= \dfrac{p_u \, L_{eA} \, t_p^2}{4}$

M_{uB} = moment capacity of end plate at section BB

$= \dfrac{p_u \, L_{eB} \, t_p^2}{4}$

p_u = design tensile strength

$= U_s /1.25$ (see inset box)

Design tensile strength	
	$p_u = U_s /1.25$
S275	328 N/mm^2
S355	392 N/mm^2
Values of U_s taken from BS EN 10025: 1993[21]	

where:

L_{eA} = effective length of plastic hinge at section AA

$= 2e_{eA} + (n - 1)\, p_{eA}$

$e_{eA} = e_1$ but $\leq e_2$

$p_{eA} = p$ but $\leq 2e_2$

L_{eB} = effective length of plastic hinge at section BB

$= 2e_{eB} + (n - 1)\, p_{eB}$

$e_{eB} = e_1$ but $\leq c_1 + D_h/2$

$p_{eB} = p$ but $\leq 2c_1 + D_h$

$c_1 = 0.5\,(g - t_w - 2s)$

L = distance between plastic hinges

$= c_1 - D_h/2$

t_p = thickness of end plate

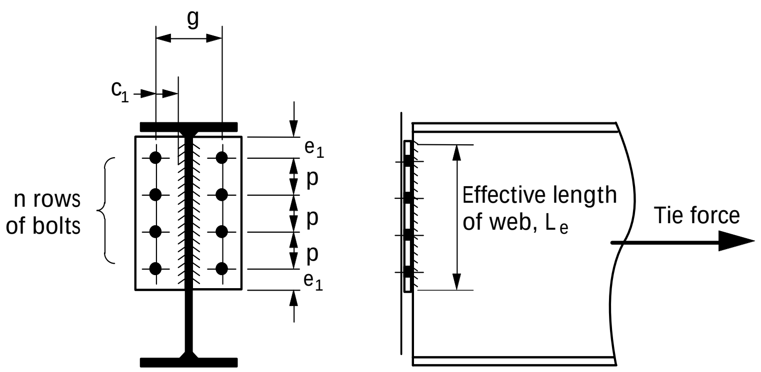
D_h = diameter of hole *

t_w = thickness of beam web

s = leg length of the fillet weld

* For Hollo-Bolts D_h is the hole diameter in the plate as given in Table H.61 of the yellow pages.

CHECK 12	Structural Integrity - Supported beam



Note: This check is only needed if it is necessary to comply with structural integrity requirements

(i) Structural integrity – tension capacity of beam web

Basic requirement:

Tie force $\leq$ Tension capacity of beam web

Tension capacity of beam web $= L_e\, t_w\, p_y$

where:

L_e = effective length

$= 2e_e + (n - 1)p_e$

$e_e = e_1$ but $\leq c_1 + D_h/2$

$p_e = p$ but $\leq 2c_1 + D_h$

$c_1 = 0.5\,(g - t_w - 2s)$

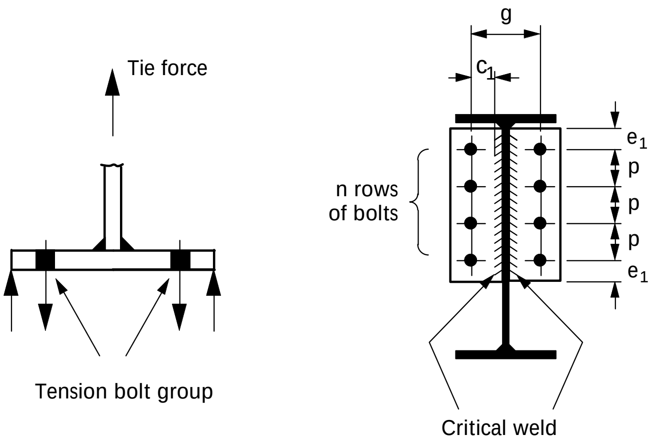
t_w = thickness of beam web

D_h = diameter of hole

p = bolt pitch

s = leg length of the fillet weld

CHECK 13	Structural integrity - Tension bolt group/welds



Note: These checks are only needed if it is necessary to comply with structural integrity requirements

Basic requirement:

(i) Structural integrity – tension capacity of bolts in presence of extreme prying

Tie force $\leq$ Tension capacity of bolt group

Tension capacity of bolt group $= 2n\, A_t\, p_{tr}$ *

p_{tr} = reduced tension strength of a bolt in presence of extreme prying

= 300N/mm² for 8.8 bolts (see Appendix D)

* See note 3 for Flowdrill or Hollo-Bolts

(ii) Structural integrity – weld tension capacity

Tie force $\leq$ Tension capacity of beam web/end plate weld

Tension capacity of beam web/end plate weld $= 2\left(1.25 p_w a\, (L_e - 2s)\right)$

where:

A_t = tensile stress area of a bolt

n = number of rows of bolts

$L_e = 2e_e + (n - 1)\, p_e$

$e_e = e_1$ but $\leq c_1 + D_h/2$

$p_e = p$ but $\leq 2c_1 + D_h$

$c_1 = 0.5\,(g - t_w - 2s)$

t_w = thickness of beam web

a = effective throat size of weld

= 0.7s (normally)

s = leg length of the fillet weld

p = bolt pitch

p_w = design strength of the weld (BS 5950-1, Table 37)

D_h = diameter of hole

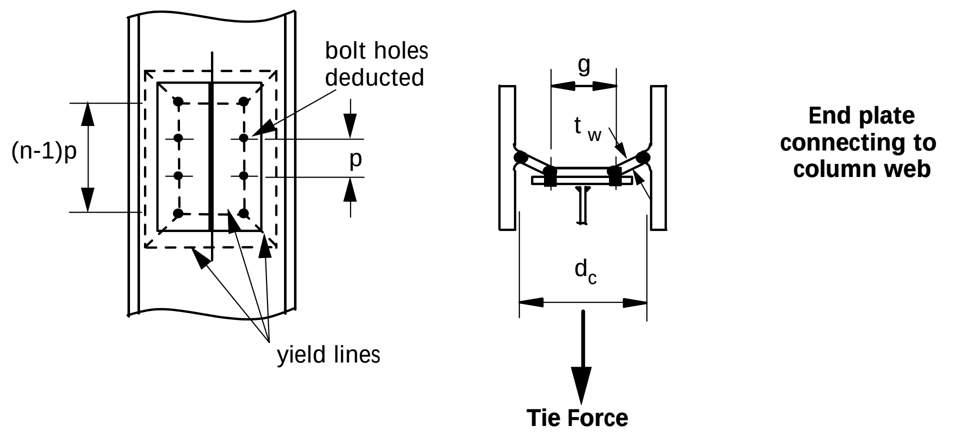
Notes:

(1) The reduced tension strength (p_{tr}) is only used when end plate design for structural integrity is based on Appendix C or CHECK 11.

(2) Where a beam is attached to one side of a column web without a beam on the opposite side, or to RHS column, the bolt tensions have to be resisted by local bending of the web or RHS wall. UC webs can resist 75kN but need to be checked if the tying force is higher. UB webs need to be checked for 75kN and higher tying forces. CHECK 14 proposes a design model which could be used for this purpose. CHECK 15 proposes a design model for checking the wall of RHS column.

(3) For Flowdrill or Hollo-Bolt connections the value $A_t p_{tr}$ is replaced by the Structural integrity Tensile capacity (P_{si}) taken from Tables H.55b and H.56 respectively in the yellow pages.

CHECK 14	Structural integrity – Supporting column web (UC **or** UB) **H**



Note: This check is only needed if it is necessary to comply with structural integrity requirements

Structural integrity – Tying capacity of rolled column web in the presence of axial compression in the column

Basic requirement:

Tie force ≤ Tying capacity of column web

$$\text{Tying capacity of column web} = \frac{8\,M_u}{1-\beta_1}\left(\eta_1 + 1.5(1-\beta_1)^{0.5}(1-\gamma_1)^{0.5}\right) *$$

M_u = moment capacity of column web per unit length

$$= \frac{p_u\, t_w^{\,2}}{4}$$

p_u = design tensile strength of the column

= $U_s/1.25$ (see inset box)

* Factor 1.5 in the equation includes an allowance for the axial compression in the column.

where:

$$\eta_1 = \frac{(n\text{-}1)\,p - \frac{n}{2}\,D_h}{d_c}$$

$$\beta_1 = \frac{g}{d_c}$$

$$\gamma_1 = \frac{D_h}{d_c}$$

d_c = depth of column between fillets

t_w = thickness of column web

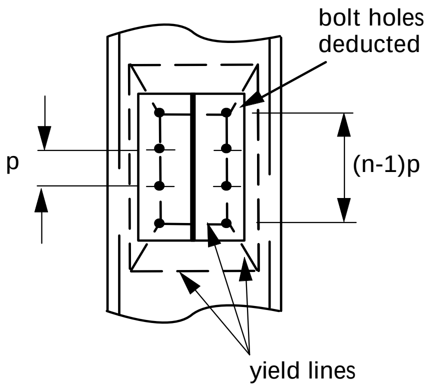
g = gauge (cross centres)

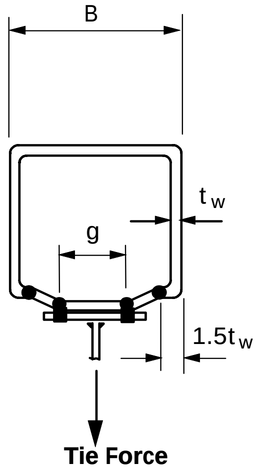
D_h = diameter of hole

Design tensile strength	
	$p_u = U_s/1.25$
S275	328 N/mm^2
S355	392 N/mm^2
Values of U_s taken from BS EN 10025: 1993[21]	

Note: The check is required for either single-sided connections to the rolled column web or unequally loaded double-sided connections to the rolled column web.

CHECK 15	Structural integrity – Supporting column wall (RHS)





End plate connecting to RHS column

Note: This check is only needed if it is necessary to comply with structural integrity requirements

Structural integrity – Tying capacity of RHS wall in the presence of axial compression in the column

Basic requirement:

Tie force $\leq$ Tying capacity of RHS column wall

Tying capacity RHS column wall $= \dfrac{8\,M_u}{1-\beta_1}\left(\eta_1 + 1.5(1-\beta_1)^{0.5}(1-\gamma_1)^{0.5}\right)$ *

M_u = moment capacity of RHS column wall per unit length

$= \dfrac{p_u\,t_w^2}{4}$

p_u = design tensile strength of the RHS column

$= U_s/1.25$ (see inset box)

* Factor 1.5 in the equation includes an allowance for the axial compression in the column.

Design tensile strength	
	$p_u = U_s/1.25$
S275	328 N/mm^2
S355	392 N/mm^2
Values of U_s taken from BS EN 10210-1: 1994[3]	

where:

$\eta_1 = \dfrac{(n-1)\,p - \frac{n}{2}D_h}{(B-3t_w)}$

$\beta_1 = \dfrac{g}{(B-3t_w)}$

$\gamma_1 = \dfrac{D_h}{(B-3t_w)}$

B = overall width of RHS column wall to which the connection is made

t_w = thickness of RHS column

D_h = diameter of hole in RHS

= bolt diameter for Flowdrill

= hole diameter (in RHS) given in Table H.61 of yellow pages for Hollo-Bolt

g = gauge (cross centres)

n = number of rows of bolts

p = bolt pitch

5.6 WORKED EXAMPLES

The worked examples show design calculations for typical standard connections. Each example demonstrates first the use of the capacity tables (yellow pages) and then full checks according to the procedures in Section 5.5. The full checks will normally only need to be applied to non-standard connections but their application to standard connections demonstrates the validity of the much simpler process when using standard details.

When calculations must be made for non-standard connections some design checks may be omitted where it is obvious, from inspection of the detail, that a check is not critical. In the case of Example 1, *Check 3 and Check 9 Block Shear* calculation are not made since they are never a critical factor for end plates with bolts spaced at reasonable centres in the plate length. However, if a connection design was made using a long plate with bolts concentrated at one end, it is possible that *Block Shear failure* could occur before *Plain Shear failure* and then the *Block shear* checks should be made.

Check 7, dealing with overall stability of an unrestrained beam, should be undertaken by the member designer taking account of any notching required at the ends of the supported beam in order to facilitate the use of a simple connection.

Checks 11 to 15 deal with structural integrity in the presence of an axial tie force required to be developed in some members to ensure the steel frame is sufficiently robust, or in the case of some multi-storey buildings, to localise accidental damage. When tying capacity is not required these checks may be omitted.

Example 1

Example 1 covers design checks for a two sided beam-to-beam connection. A 200 x 10 end plate is used for both beams since it is necessary to have common bolt centres in the supporting beam.

Example 2

Example 2 demonstrates the additional design checks required when a beam-to-column web connection must be designed to resist tying forces.

Example 3

Example 3 is a beam connection to an RHS column using normal grade 8.8 bolts in Flowdrill threaded holes to connect a flexible end plate to the column wall. The beam sizes and vertical reactions as in Example 1, so only checks which are different to those in the first example are shown. A 150 x 8 end plate is used for the smaller beam and a 200 x 10 for the larger beam since the connecting bolts to the column are not common to both beams.

The connection design also considers tie forces in the beams but it should be noted that the tie forces are ignored in checks for vertical reactions and vertical reactions are ignored in checks for tie forces.

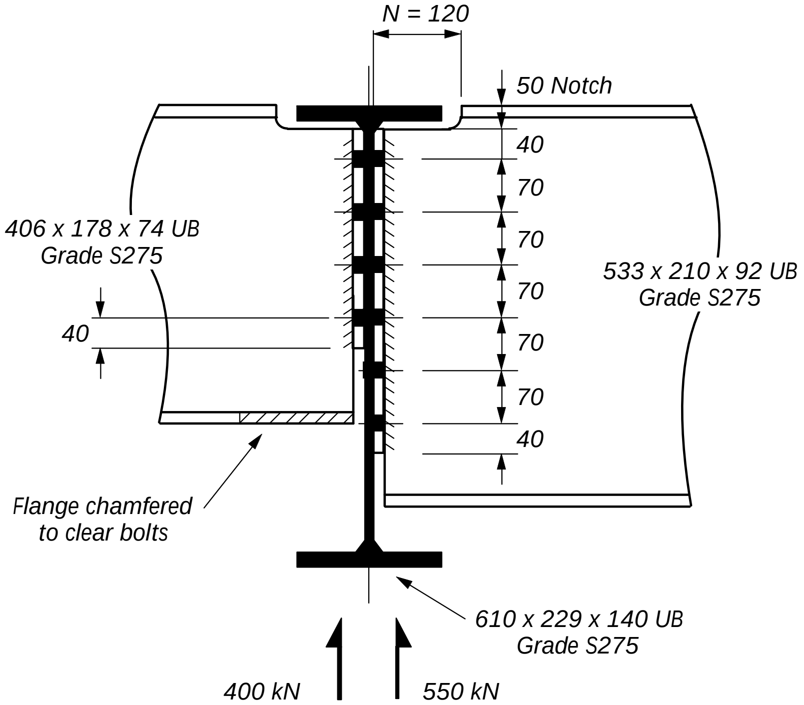
Example 4

Example 4 covers the same beam for connections to an RHS column as in Example 3 but uses Hollo-Bolts to connect a flexible end plate to the column wall. However, Hollo-Bolts require larger than normal holes so different standard details (see Table H.3) have to be used to maintain an adequate edge distance from the hole lines.

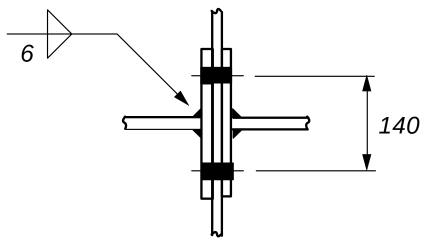
CALCULATION SHEET BCSA	Job No: Joints in Steel Construction - Simple Connections		Sheet 1 of 12
	Title: Example 1 - Flexible End Plates - Beam to Beam		
	Client: SCI/BCSA Connections Group		
	Calcs by: RS	Checked by: AM	Date: May 2002

DESIGN EXAMPLE 1

Check the following beam to beam connection for the design forces shown.
Yellow pages used for initial selection of End plates.



See Figure 5.5 and Note below



Note: Bolt gauge set to accommodate the larger beam

See Figure 5.5 and Yellow pages Table H.3

Flexible End Plates:	200 x 10 (Type EB4)	200 x 10 (Type EB6)

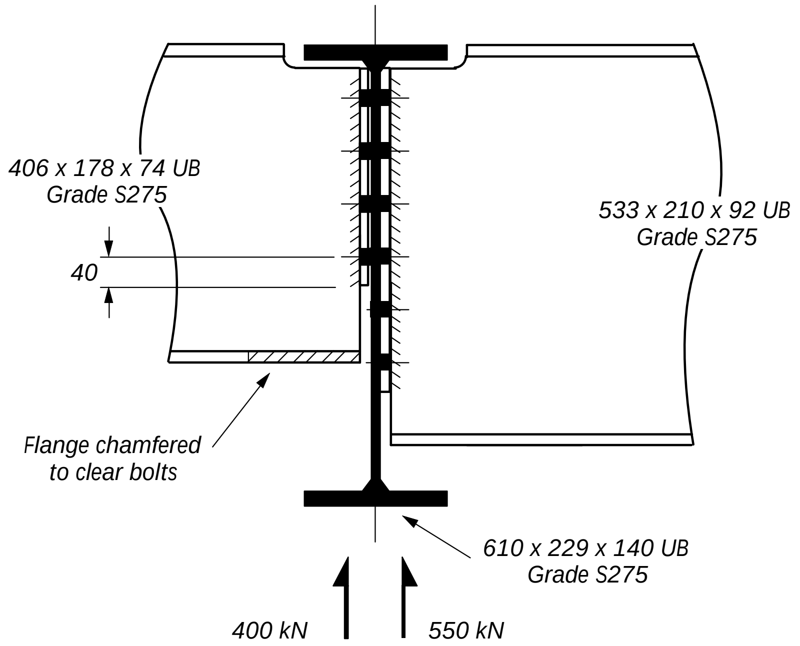
Design Information:

Bolts:	M20 8.8
Welds:	6mm fillet
Material:	All S275

Note: End plate size and details are governed by the larger beam, hence the end plate for the smaller beam is 200 x 10 and not 150 x 8 as recommended in Figure 5.5

Title	Example 1 - Flexible End Plates - Beam to Beam	Sheet	2 of 12

CONNECTION DESIGN USING CAPACITY TABLES FROM YELLOW PAGES



406 x 178 x 74 UB Grade S275	533 x 210 x 92 UB Grade S275	
End plate type EB4(200 x 10)	**End plate type EB6 (200 x 10)**	
Note: End plate size is determined by the size of the opposing beam.		
Welds 6mm fillet	*Welds 6mm fillet*	
Bolts M20 8.8	*Bolts M20 8.8*	
Bolts at 140 cross centres	*Bolts at 140 cross centres*	
4 rows of bolts	*6 rows of bolts*	
From Capacity tables in yellow pages	*From Capacity table H.20 in yellow pages*	*Yellow Pages Table H.20*
The values obtained from the capacity Table H.20 are for a 150 x 8 end plate (type EA4)		
Connection shear capacity **= 409kN > 400kN**	*Connection shear capacity* **= 645kN > 550kN**	**∴ O.K.**
Maximum notch length **= 218mm > 120mm**	*Maximum notch length* **= 229mm > 120mm**	**∴ O.K.**

Web thickness of supporting beam = 13.1mm — *Table H.64*

Minimum support thickness = (5.6 + 5.8) = 11.4mm < 13.1mm — **∴ O.K.**

Connection is adequate

Title *Example 1 - Flexible End Plates - Beam to Beam*	Sheet *3 of 12*

SUMMARY OF FULL DESIGN CHECKS FOR EXAMPLE 1

Note: *Values given are overall capacities unless otherwise noted.*

<table>
<tr><th rowspan="2">Sheet Nos</th><th rowspan="2" colspan="2">CHECK</th><th colspan="2">406UB (S275)</th><th colspan="2">533UB (S275)</th><th colspan="2">610UB (S275)</th></tr>
<tr><th>Capacity</th><th>Applied Load</th><th>Capacity</th><th>Applied Load</th><th>Capacity</th><th>Applied Load</th></tr>
<tr><td>4</td><td colspan="2">CHECK 1
- Recommended detailing practice</td><td colspan="6">All recommendations adopted</td></tr>
<tr><td>4</td><td>CHECK 2
Supported Beam</td><td>Capacity of Fillet Welds (kN)</td><td>514</td><td>400</td><td>772</td><td>550</td><td colspan="2">Not Applicable</td></tr>
<tr><td>4</td><td>CHECK 3</td><td></td><td colspan="4">Not applicable</td><td colspan="2">Not Applicable</td></tr>
<tr><td>5</td><td>CHECK 4
Supported Beam
- Capacity at the connection of the beam web to the end plate</td><td>Shear capacity (kN)</td><td>409</td><td>400</td><td>645</td><td>550</td><td colspan="2">Not Applicable</td></tr>
<tr><td></td><td></td><td></td><td colspan="4">CRITICAL CHECK IN THIS EXAMPLE</td><td colspan="2"></td></tr>
<tr><td>6 & 7</td><td>CHECK 5
Supported Beam
- Capacity at the notch</td><td>(Bending capacity, kNm)</td><td>89</td><td>48</td><td>164</td><td>66</td><td colspan="2">Not Applicable</td></tr>
<tr><td>8</td><td>CHECK 6
Supported Beam
- Local stablity at a notch
(Beam restrained)</td><td>Notch length mm</td><td>412.8</td><td>110</td><td>533.1</td><td>110</td><td colspan="2">Not Applicable</td></tr>
<tr><td></td><td></td><td></td><td colspan="4">Notch length (c) < Specified limits</td><td colspan="2"></td></tr>
<tr><td>8</td><td>CHECK 7
LTB of Supported beam
(Beam restrained)</td><td>-</td><td colspan="4">Not applicable</td><td colspan="2">Not Applicable</td></tr>
<tr><td>9</td><td>CHECK 8
Supporting Beam
- Bolt Group shear capacity</td><td>(Capacity of bolt group, kN)</td><td>735</td><td>400</td><td>1103</td><td>550</td><td colspan="2">Not Applicable</td></tr>
<tr><td>10 & 11</td><td>CHECK 9
Supporting Beam
- Connecting Elements
(Strength of plate)</td><td>Shear (kN)
Bearing (Capacity per bolt line, kN)</td><td>400
368</td><td>200
200</td><td>589
552</td><td>275
275</td><td colspan="2">Not Applicable</td></tr>
<tr><td>12</td><td>CHECK 10
Supporting Beam
- Capacity
(Local capacity of beam web)</td><td>Shear kN
Bearing (Capacity per bolt, kN)</td><td colspan="4">Not Applicable</td><td>771
121</td><td>384
96</td></tr>
</table>

Title *Example 1 - Flexible End Plates - Beam to Beam*	Sheet *4 of 12*

CHECK 1: Recommended detailing practice

End Plates: 200×10mm thick

Length, l = 290mm (>0.6D for 406 UB)

= 430mm (>0.6D for 533 UB)

Bolts: 20mm dia at 140mm cross centres

CHECK 2: Supported Beam - Welds

Capacity of Fillet Welds to connecting end plate to beam web

Basic requirement: $F_v \leq P_{weld}$

For 406 x 178 x 74 UB Grade S275

Capacity of fillet weld, $P_{weld} = p_w \, l_{we} \, a$

Total effective length of weld, $l_{we} = 2(l - 2s)$

$= 2(290 - (2 \times 6)) = 556$mm

For 6mm fillet welds, classification 35, S275 steel

$p_w = 220$N/mm^2 — *p_w from BS 5950-1 Table 37*

Throat size, $a = 0.7\,s = 0.7 \times 6 = 4.2$mm

$$\therefore P_{weld} = \frac{220 \times 556 \times 4.2}{10^3}$$

$= 514$kN

$F_v = 400\text{kN} < 514\text{kN}$ **∴ O.K.**

For 533 x 210 x 92 UB Grade S275

Capacity of fillet weld, $P_{weld} = p_w \, l_{we} \, a$

Total effective length of weld, $l_{we} = 2(l - 2s)$

$= 2((430 - (2 \times 6)) = 836$mm

For 6mm fiillet welds, classification 35, S275 steel

$p_w = 220$N/mm^2 — *p_w from BS 5950-1 Table 37*

Throat size, $a = 0.7\,s = 0.7 \times 6 = 4.2$mm

$$\therefore P_{weld} = \frac{220 \times 836 \times 4.2}{10^3}$$

$= 772$kN

$F_v = 550\text{kN} < 772\text{kN}$ **∴ O.K.**

CHECK 3: Not applicable

Title *Example 1 - Flexible End Plates - Beam to Beam*	Sheet *5 of 12*

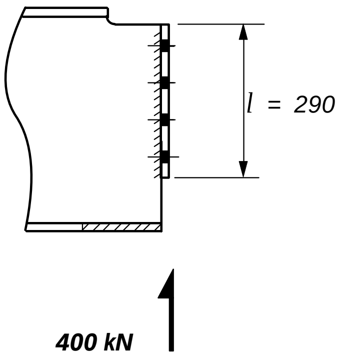
CHECK 4: Supported Beam - Capacity at connection

Shear capacity of beam web at the end plate

Basic requirement: $F_v \leq P_v$

Shear capacity of beam web, $P_v = 0.6\ p_y\ A_v$

For 406 x 178 x 74 UB Grade S275



Shear area of beam web,

$A_v = 0.9 \times 290 \times 9.5$

$= 2480\text{mm}^2$

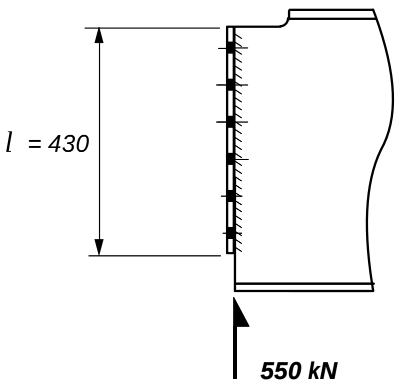
Shear capacity of beam web,

$$P_v = \frac{0.6 \times 275 \times 2480}{10^3}$$

$= 409\text{kN}$

$\boldsymbol{F_v = 400\text{kN} < 409\text{kN}}$ **∴ O.K.**

For 533 x 210 x 92 UB Grade S275



Shear area of beam web,

$A_v = 0.9 \times 430 \times 10.1$

$= 3909\text{mm}^2$

Shear capacity of beam web,

$$P_v = \frac{0.6 \times 275 \times 3909}{10^3}$$

$= 645\text{kN}$

$\boldsymbol{F_v = 550\text{kN} < 645\text{kN}}$ **∴ O.K.**

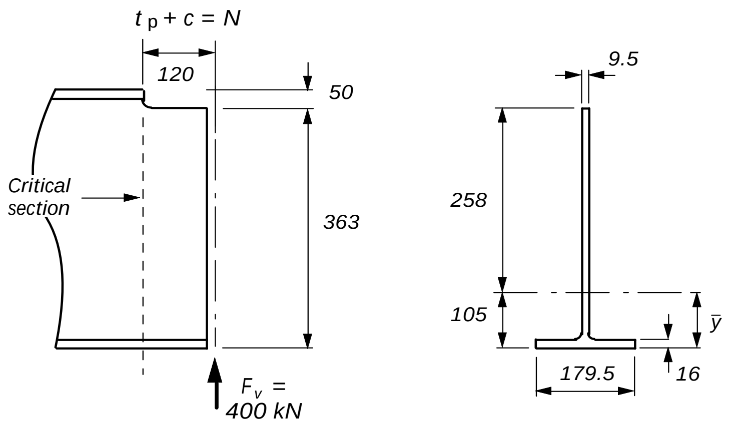
Title *Example 1 - Flexible End Plates - Beam to Beam* Sheet *6 of 12*

CHECK 5: Supported Beam - Capacity at the notch

Shear and bending interaction at the notch

Basic requirement for bending: $F_v (t_p + c) \leq M_{cN}$

For 406 x 178 x 74 UB Grade S275



For low shear conditions:

$F_v \leq 0.75P_{vN}$ and $M_{cN} = p_y Z_N$

$P_{vN} = 0.6\, p_y A_{vN}$

$A_{vN} = 363 \times 9.5 = 3449\text{mm}^2$

$P_{vN} = \dfrac{0.6 \times 275 \times 3449}{10^3} = 569\text{kN}$

$0.75P_{vN} = 0.75 \times 569 = 427\text{kN}$

$F_v = 400\text{kN} < 427\text{kN}$ ∴ *Low shear criteria for bending applies*

and $M_{cN} = p_y Z_N$

Taking moments of area about bottom flange:

$(179.5 \times 16 \times 8) + (347 \times 9.5 \times 189) = \big((179.5 \times 16) + (347 \times 9.5)\big) \times \bar{y}$

$\bar{y} = 105\text{mm}$

Second moment of area about neutral axis:

$$I_{xx} = \frac{1}{10^4}\left(\frac{179.5 \times 16^3}{12} + (179.5 \times 16 \times 97^2)\right) + \frac{1}{10^4}\left(\frac{9.5 \times 347^3}{12} + (347 \times 9.5 \times 84.5^2)\right)$$

$= 8370\text{cm}^4$

$$Z_N = \frac{I_{xx}}{y_{max}} = \frac{8370}{25.8} = 324\text{cm}^3$$

Moment capacity of the beam at the notch in the presence of shear

$M_{cN} = p_y Z_N$

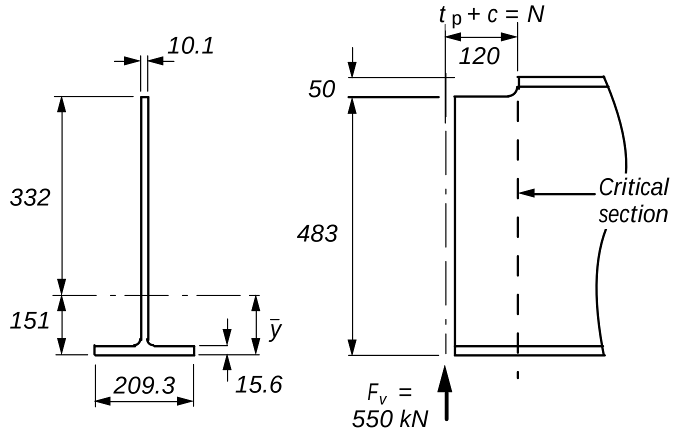
$$= \frac{275 \times 324}{10^3} = 89.1 \text{ kNm}$$

Eccentric moment $F_v(t_p + c) = \dfrac{400 \times 120}{10^3} = 48\text{kNm}$

48kNm < 89.1kNm **∴ O.K.**

Title Example 1 - Flexible End Plates - Beam to Beam	Sheet 7 of 12

For 533 x 210 x 92 UB Grade S275



For low shear conditions:

$F_v \leq 0.75P_{vN}$ and $M_{cN} = p_y Z_N$

$P_{vN} = 0.6\, p_y\, A_{vN}$

$A_{vN} = (D - d_{c1})\, t_w$

$= 483 \times 10.1 = 4878\text{mm}^2$

$\therefore P_{vN} = \dfrac{0.6 \times 275 \times 4878}{10^3} = 805\text{kN}$

$0.75P_{vN} = 0.75 \times 805 = 604\text{kN}$

$F_v = 550\text{kN} \leq 604\text{kN}$ $\therefore$ *Low shear criteria for bending applies*

and $M_{cN} = p_y Z_N$

Taking moments of area about bottom flange:

$(209.3 \times 15.6 \times 7.8) + (467.4 \times 10.1 \times 249.3) = \big((209.3 \times 15.6) + (467.4 \times 10.1)\big) \times \bar{y}$

$\bar{y} = 151\text{mm}$

Second moment of area about neutral axis:

$$I_{xx} = \frac{1}{10^4}\left(\frac{209.3 \times 15.6^3}{12} + (209.3 \times 15.6 \times 143.2^2)\right) + \frac{1}{10^4}\left(\frac{10.1 \times 467.4^3}{12} + (467.4 \times 10.1 \times 98.3^2)\right)$$

$$= 19858\text{cm}^4$$

$$Z_N = \frac{I_{xx}}{y_{max}} = \frac{19858}{33.2} = 598\text{cm}^3$$

Moment capacity of the beam at the notch in the presence of shear

$$M_{cN} = p_y\, Z_N$$

$$= \frac{275 \times 598}{10^3} = 164\text{ kNm}$$

Eccentric moment

$$F_v(t_p + c) = \frac{550 \times 120}{10^3} = 66\text{kNm}$$

66kNm < 164kNm **$\therefore$ O.K.**

Title *Example 1 - Flexible End Plates - Beam to Beam*	Sheet *8 of 11*

CHECK 6: Supported Beam - Local Stablity of notched beam

When the beam is restrained against lateral torsional buckling no account need be taken of notch stability provided that:

For one flange notched beam in S275 steel

Basic requirements:

Notch depth $d_{c1} \leq \frac{D}{2}$

and $c \leq D$ for $\frac{D}{t_w} \leq 54.3$

$c \leq \frac{160000D}{(D/t_w)^3}$ for $\frac{D}{t_w} > 54.3$

For 406 x 178 x 74 UB Grade S275 *(c = 110mm)*

Notch depth $d_{c1} = 50mm < \frac{412.8}{2} = 206.4mm$ ∴ O.K.

$\frac{D}{t_w} = \frac{412.8}{9.5} = 43.5 < 54.3$

$c = 110mm < 412.8mm$ ∴ O.K.

For 533 x 210 x 92 UB *(c =110 mm)*

Notch depth $d_{c1} = 50mm < \frac{533.1}{2} = 266.6mm$ ∴ O.K..

$\frac{D}{t_w} = \frac{533.1}{10.1} = 52.8 < 54.3$

$c = 110mm < 533.1mm$ ∴ O.K.

CHECK 7: Not applicable

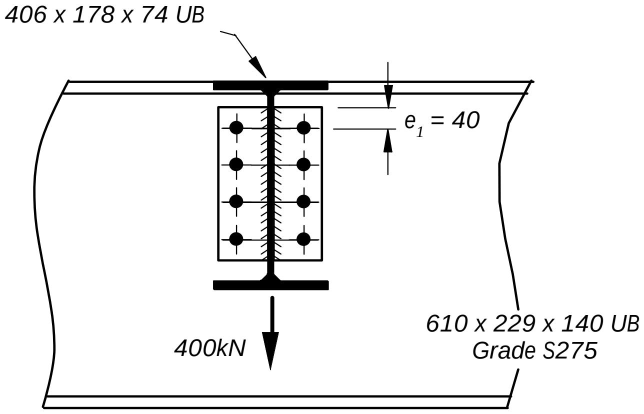
(because the supported beam is being considered as restrained against lateral torsional buckling)

Title Example 1 - Flexible End Plates - Beam to Beam	Sheet 9 of 12

CHECK 8: Supporting Beam - Bolt Group

Basic requirement: $F_v \leq \Sigma P_s$

For 406 x 178 x 74 UB



Shear capacity of single bolt, $P_s = p_s . A_s$

For M20 8.8 bolts, $P_s = \dfrac{375 \times 245}{10^3}$

$= 91.9kN$

p_s from BS 5950-1 Table 30. See also Bolt capacities Yellow pages Table H.49

But for top pair of bolts P_s is the lesser of $p_s A_s$ or $0.5 k_{bs} e_1 t_p p_{bs}$ (where $k_{bs} = 1.0$ for standard clearance holes)

p_{bs} from BS 5950-1 Table 32

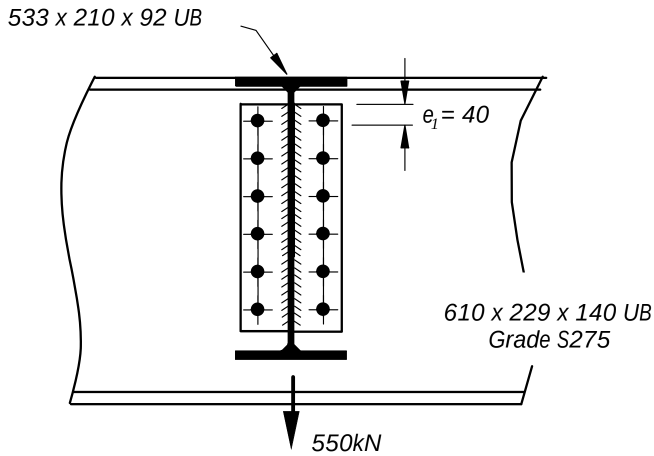
$0.5\ k_{bs}\ e_1\ t_p\ p_{bs} = \dfrac{0.5 \times 1.0 \times 40 \times 10 \times 460}{10^3}$

$= 92kN \quad \therefore$ use 91.9kN

$\Sigma P_s = 8 \times 91.9 = 735kN$

$\mathbf{F_v = 400kN < 735kN}$ **$\therefore$ O.K.**

For 533 x 210 x 92 UB



$\Sigma P_s = 12 \times 91.9 = 1103kN$

$\mathbf{F_v = 550kN < 1103kN}$ **$\therefore$ O.K.**

Title *Example 1 - Flexible End Plates - Beam to Beam*	Sheet *10 of 12*

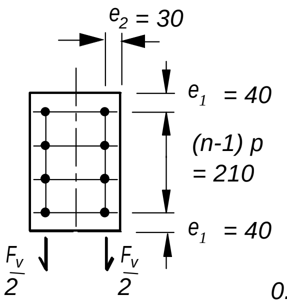
CHECK 9: Supporting Beam - Connecting elements

Shear and bearing of end plate connected to supporting beam

(i) Basic requiremenf for shear: $\frac{F_v}{2} \leq P_{v.min}$

For 406 x 178 x 74 UB

Shear capacity of end plate, $P_{v.min}$ is the smaller of plain shear capacity P_v and block shear capacity P_r



$F_v = 400kN$

Plain shear, $P_v = \min(0.6\ p_y\ A_v\ ,\ 0.7\ p_y K_e\ A_{v.net})$

Shear area, $A_v = 0.9\,(2e_1 + (n-1)\,p)\,t_p$

$= 0.9\,(80 + 210) \times 10 \qquad = 2610mm^2$

Net area, $A_{v.net} = A_v - n\,D_h\,t_p$

$= 2610 - (4 \times 22 \times 10) \qquad = 1730mm^2$

$0.6\,p_y\ A_v = \frac{0.6 \times 275 \times 2610}{10^3} \qquad = 431kN$

$0.7\,p_y K_e\ A_{v.net} = \frac{0.7 \times 275 \times 1.2 \times 1730}{10^3} \qquad = 400kN$

$\therefore\ P_v = 400kN$

Block shear $P_r = 0.6 p_y\,t_p\,(L_v + K_e\,(L_t - kD_h))$ — See Note below

$L_v = e_1 + (n-1)p = 40 + 210 \quad = 250mm$

$L_t = e_2 \qquad = 30mm$

$k = 0.5$

$K_e = 1.2$ (for S275)

$\therefore\ P_r = \frac{0.6 \times 275 \times 10\,(250 + 1.2(30 - 0.5 \times 22))}{10^3}$

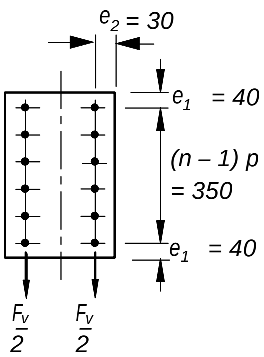
$= 450kN$

$P_{v.min} = \min(P_v\ , P_r)$

$= 400kN$

$F_v/2$ **= 200kN < 400kN** — **∴ O.K.**

For 533 x 210 x 92 UB



$F_v = 550kN$

Plain shear, P_v

$A_v = 0.9(80 + 350) \times 10 \qquad = 3870mm^2$

$A_{v.net} = 3870 - (6 \times 22 \times 10) \qquad = 2550mm^2$

$0.6\,p_y\ A_v = \frac{0.6 \times 275 \times 3870}{10^3} \qquad = 639kN$

$0.7\,p_y K_e\ A_{v.net} = \frac{0.7 \times 275 \times 1.2 \times 2550}{10^3} \qquad = 589kN$

$\therefore\ P_v = 589kN$

Block shear $P_r = \frac{0.6 \times 275 \times 10\,(390 + 1.2(30 - 0.5 \times 22))}{10^3}$ — See Note below

$= 681kN$

$P_{v.min} = \min(P_v\ ,\ P_r)$

$= 589kN$

$F_v/2$ **= 275kN < 589kN** — **∴ O.K.**

NOTE: *Block shear checks have been shown here, but they are never critical for well proportioned end plates. However, if the bolt spacing is concentrated at one part of a plate then these checks may be critical.*

Title *Example 1 - Flexible End Plates - Beam to Beam*	Sheet *11 of 12*

(ii) Basic requirement for bearing: $\frac{F_v}{2} \leq \Sigma P_{bs}$

For 406 x 178 x 74 UB

Bearing capacity, $P_{bs} = k_{bs} d\, t_p\, p_{bs}$

but for top bolt $P_{bs} = \min\left(k_{bs} d\, t_p\, p_{bs},\ 0.5 k_{bs}\, e_1\, t_p\, p_{bs} \right)$

p_{bs} from BS 5950-1 Table 32

$$k_{bs} d\, t_p\, p_{bs} = \frac{1.0 \times 20 \times 10 \times 460}{10^3} = 92kN$$

$$0.5 k_{bs} e_1\, t_p\, p_{bs} = \frac{1.0 \times 20 \times 10 \times 460}{10^3} = 92kN$$

$$\Sigma P_{bs} = 3 \times 92 + 92 = 368kN$$

$$\frac{F_v}{2} = \frac{400}{2} = 200kN < 368kN$$

∴ O.K.

For 533 x 210 x 92 UB

Bearing capacity as above

$$\Sigma P_{bs} = 5 \times 92 + 92 = 552kN$$

$$\frac{F_v}{2} = \frac{550}{2} = 275kN < 552kN$$

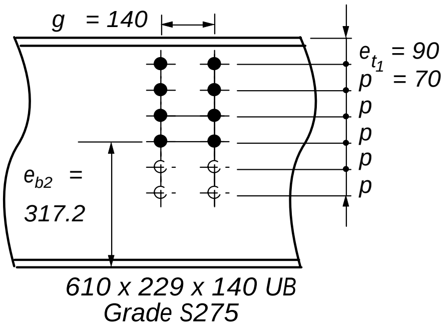
∴ O.K.

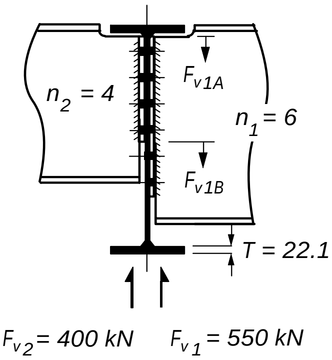
Title	Example 1 - Flexible End Plates - Beam to Beam	Sheet	12 of 12

CHECK 10 : Supporting Beam - Capacity

Local Shear & Bearing Capacity of beam web supporting two beams

(i) Basic requirement for shear: $\frac{F_{v1A}}{2} + \frac{F_{v2}}{2} \leq P_v$





For double sided portion, $\frac{F_{v1A}}{2} + \frac{F_{v2}}{2} \leq P_v$

$F_{v1A} = F_{v1} \frac{n_2}{n_1} = 550 \times \frac{4}{6} = 367kN$

Shear capacity, $P_v = \min\left(0.6\, p_y\, A_v\ ,\ 0.7\, p_y\, K_e\, A_{v.net}\right)$

Gross shear area, $A_v = \left(e_t + (n_2 - 1)\, p + e_b\right)\, t_w$

$e_t = \min(e_{t1}\ ,\ 5d) = 90mm$

$e_b = \min(e_{b2}\ ,\ {}^{g}/_{2},\ p,\ 5d) = 70mm$

$A_v = (90 + (4 - 1)\, 70 + 70) \times 13.1 = 4847mm^2$

$\therefore 0.6\, p_y\, A_v = \frac{0.6 \times 265 \times 4847}{10^3} = 771kN$

Net shear area, $A_{v.net} = A_v - n_2\, D_h\, t_w$

$= 4847 - (4 \times 22 \times 13.1) = 3694mm^2$

$\therefore 0.7 p_y K_e A_{v.net} = \frac{0.7 \times 265 \times 1.2 \times 3694}{10^3} = 822kN$

$\therefore P_v = 771kN$

$\frac{F_{v1A}}{2} + \frac{F_{v2}}{2} = \frac{367}{2} + \frac{400}{2} = 384kN$

384kN < 771kN **∴ O.K**

Note: *The above check is for **local** shear only; the effects of any global shear forces must also be considered.*

(ii) Basic requirement for bearing: $\frac{F_{v1}/2}{n_1} + \frac{F_{v2}/2}{n_2} \leq P_{bs}$

Bearing capacity of supporting beam per bolt, $P_{bs} = d\, t_w\, p_{bs}$

$= \frac{20 \times 13.1 \times 460}{10^3} = 121kN$ — p_{bs} from BS 5950-1 Table 32

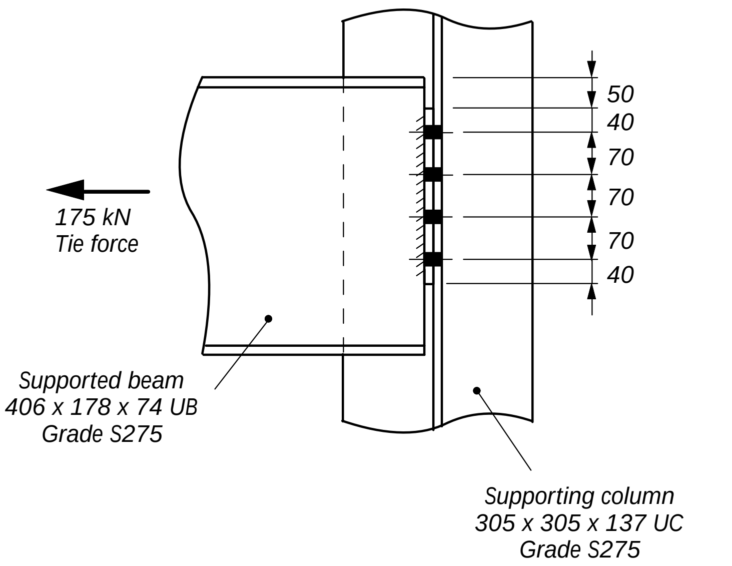
$\frac{F_{v1}/2}{n_1} + \frac{F_{v2}/2}{n_2} = \frac{550/2}{6} + \frac{400/2}{4} = 96kN$

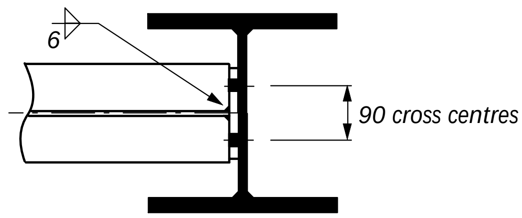
96kN < 121kN **∴ O.K**

CALCULATION SHEET BCSA	Job No: Joints in Steel Construction - Simple Connections		Sheet: 1 of 8
	Title: Example 2 - Flexible end plate - Beam to UC column web - Structural Integrity		
	Client: SCI/BCSA Connections Group		
	Calcs by: RS	Checked by: AM	Date: May 2002

DESIGN EXAMPLE 2

Check the following beam to column connection for the tie force shown.



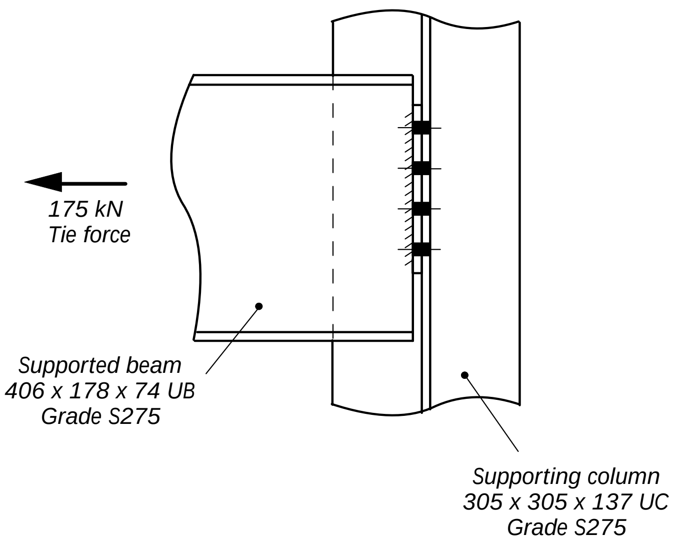


Design Information:

Bolts:	*M20 8.8 @ 90 c/c*	
End Plates:	*150 x 8 (Type EA4)*	*See figure 5.5 and Yellow pages Table H.3*
Welds:	*6mm fillet*	
Material:	*All S275 steel*	

Title *Example 2 - Flexible end plate - Beam to UC Web - Structural Integrity*	Sheet *2 of 8*

CONNECTION DESIGN USING CAPACITY TABLES FROM YELLOW PAGES



End plate 150 x 8 Type EA4

Bolts M20 8.8

Bolts at 90 cross centres

4 rows of bolts

6mm fillet weld

From Capacity table H.20 in yellow pages

Connection tying capacity = 239kN — *Yellow pages Table H.20*

Tie force = 175kN < 239kN — ***∴ O.K.***

The beam side of the connection is adequate

Note *:*

(1) The tying capacity of the connection given in the tables in the yellow pages is the least of the values obtained from CHECKS 11, 12 & 13.

(2) Beams connecting into a column web must also be checked for web bending as shown in CHECK 14 (Sheet 7).

Title	Sheet
Example 2 - Flexible end plate - Beam to UC Web - Structural Integrity	*3 of 8*

SUMMARY OF FULL DESIGN CHECKS FOR EXAMPLE 2

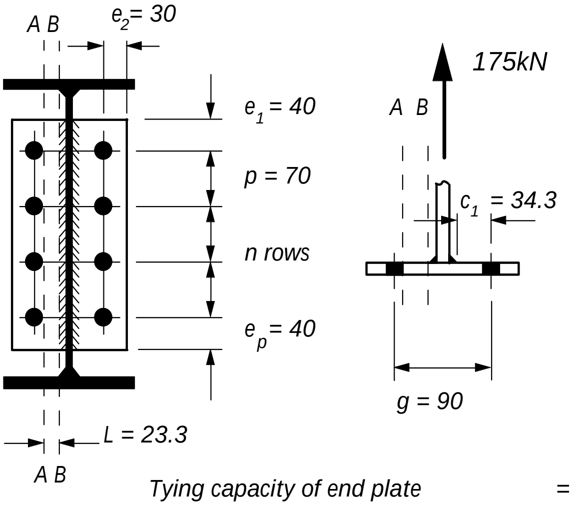
Sheet Nos	**CHECK**	**406UB S275**		**305UC S275**	
		Capacity	**Applied Load**	**Capacity**	**Applied Load**
4	***CHECK 11*** *Structural Integrity* *- Connecting Elements* *Tension capacity of end plate (kN)*	***239*** **CRITICAL CHECK BEAM SIDE**	***175***	*Not Applicable*	
5	***CHECK 12*** *Structural Integrity* *- Supported Beam* *Tension capacity of beam web (kN)*	*758*	*175*	*Not Applicable*	
6	***CHECK 13*** *Structural Integrity* *Bolts - Tension capacity of bolt group (kN)*	*588*	*175*	*Not Applicable*	
	Structural Integrity *Welds - Weld tension capacity (kN)*	*642*	*175*	*Not Applicable*	
7 & 8	***CHECK 14*** *Structural Integrity* *- Tying capacity of column web (kN)*	*Not Applicable*		***356*** **CRITICAL CHECK COLUMN SIDE**	***175***

Title	Example 2 - Flexible end plate - Beam to UC Web - Structural Integrity	Sheet	4 of 8

CHECK 11 : Structural Integrity - Connecting Elements

Tension Capacity of End Plate

Basic requirement: **Tie Force** $\leq$ **Tying capacity of end plate**



s	=	fillet size
	=	6 mm
t_w	=	web thickness
	=	9.5mm
c_1	=	$\frac{1}{2}(g - t_w - 2s)$
	=	$\frac{1}{2}(90 - 9.5 - 2 \times 6)$
	=	34.3mm

Tying capacity of end plate $= \dfrac{2\,(M_{uA} + M_{uB})}{L}$

Moment capacity of end plate at section AA $\quad M_{uA} = \dfrac{p_u\; L_{eA}\; t_p^2}{4}$

p_u = design tensile strength = $328N/mm^2$ (for S275)

t_p = plate thickness = 8mm

L_{eA} = eff. length of plastic hinge at section AA

= $2e_{eA} + (n-1)p_{eA}$

e_{eA} = e_1 but < e_2 = 30mm

p_{eA} = p but $\leq$ $2e_2$ = 60mm

$\therefore$ L_{eA} = $(2 \times 30) + (4-1)60$ = 240mm

$\therefore$ $M_{uA} = \dfrac{328 \times 240 \times 8^2}{4 \times 10^3} = 1260kNmm$

Moment capacity of end plate at section BB $\quad M_{uB} = \dfrac{p_u\; L_{eB}\; t_p^2}{4}$

L_{eB} = eff. length of plastic hinge at section BB

= $2e_{eB} + (n-l)p_{eB}$

e_{eB} = e_1 but $\leq$ $c_1 + D_h/2$ = 40mm

p_{eB} = p but $\leq$ $2c_1 + D_h$ = 70mm

$\therefore$ L_{eB} = $(2 \times 40) + (4-1)70$ = 290mm

$\therefore$ $M_{uB} = \dfrac{328 \times 290 \times 8^2}{4 \times 10^3} = 1522kNmm$

L = distance between plastic hinges = $c_1 - (D_h/2)$

= $34.3 - (22/2)$ = 23.3mm

$$\frac{2(M_{uA} + M_{uB})}{L} = \frac{2(1260 + 1522)}{23.3} = 239kN$$

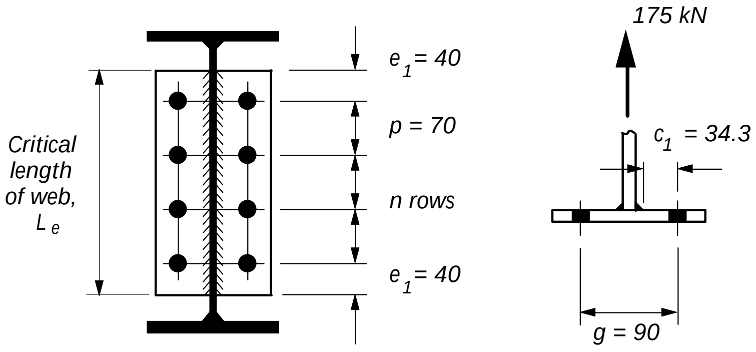
Tie force = 175kN < 239kN $\quad$ **$\therefore$ O.K.**

Title	Example 2 - Flexible end plate - Beam to UC Web - Structural Integrity	Sheet	5 of 8

CHECK 12: Structural Integrity - Supported Beam

(i) Tension Capacity of Beam Web

Basic requirement: **Tie force** $\leq$ **Tension capacity of beam web**



$c_1 = \frac{1}{2}(g - t_w - 2s)$

s = fillet size (leg length) = 6mm

t_w = web thickness = 9.5mm

Tension capacity of beam web = $L_e \ t_w \ p_y$

Effective length, L_e = $2e_e + (n - 1)\, p_e$

$e_e = e_1$ but $\leq c_1 + D_h/2$

= 40mm

$p_e = p$ but $\leq 2c_1 + D_h$

= 70mm

$L_e = (2 \times 40) + (4 - 1)\, 70$

= 290mm

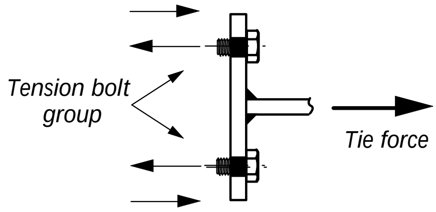
$\therefore \quad L_e\, t_w\, p_y = \dfrac{290 \times 9.5 \times 275}{10^3}$ = 758kN

Tie force = 175kN < 758kN **$\therefore$ O.K.**

Title	Example 2 - Flexible end plate - Beam to UC Web - Structural Integrity	Sheet	6 of 8

CHECK 13: Structural Integrity - Bolt Group/Welds

(i) Tension Capacity of Bolts In Presence of Extreme Prying



Basic requirement: Tie force $\leq$ Tension capacity of bolt group

Tension capacity of bolt group $= 2\,n\,A_t\,p_{tr}$

p_{tr} = *reduced tension strength of bolts in presence of extreme prying*

= *300N/mm² for grade 8.8 bolts* — *Appendix D bolt capacity tables yellow pages Table H.49*

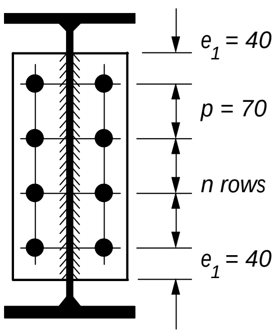
A_t = *tensile stress area of bolt* = 245mm^2

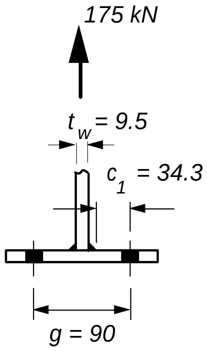
$$2\,n\,A_t\,p_{tr} = \frac{2 \times 4 \times 245 \times 300}{10^3} = 588\text{kN}$$

Tie force = 175kN < 588kN **∴ O.K.**

(ii) Weld Tension Capacity

Basic requirement: Tie force $\leq$ Tension capacity of beam web/end plate weld





s = *leg length of weld*

= 6mm

t_w = *web thickness*

= 9.5mm

$$c_1 = \frac{1}{2}(g - t_w - 2s) = \frac{1}{2}(90 - 9.5 - 2 \times 6) = 34.3\text{mm}$$

Tension capacity of beam web/end plate weld $= 2\big(1.25 p_w\, a(L_e - 2s)\big)$

For ordinary bolt connection D_h = 22mm

$L_e = 2e_e + (n-1)p_e$

$e_e = e_1$ but $\leq c_1 + D_h/2$ = 40mm

$p_e = p$ but $\leq 2c_1 + D_h$ = 70mm

$L_e = (2 \times 40) + (4-1)70$ = 290mm

Design strength of weld, p_w = 220N/mm^2 — *p_w from BS 5950-1 Table 36*

Weld throat thickness, a = $0.7\,s$ = 0.7×6 = 4.2mm

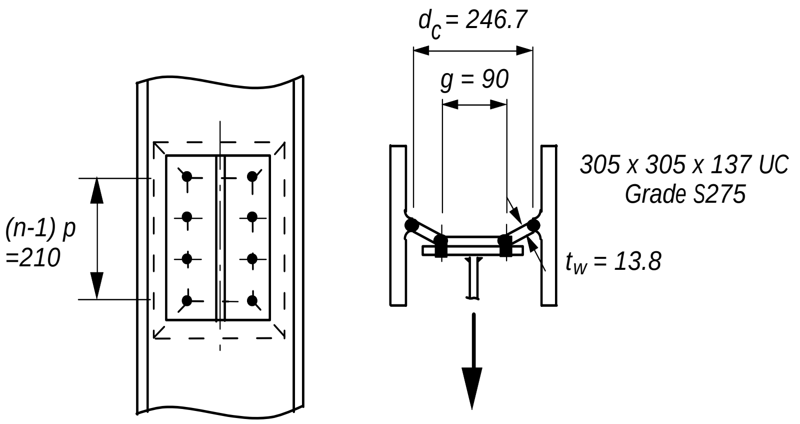
$$2\big(1.25 p_w\, a(L_e - 2s)\big) = \frac{2\big(1.25 \times 220 \times 4.2 \times (290 - 2 \times 6)\big)}{10^3} = 642\text{kN}$$

Tie force = 175kN < 642kN **∴ O.K.**

Title	Example 2 - Flexible end plate - Beam to UC Web - Structural Integrity	Sheet	7 of 8

CHECK 14: Structural Integrity – Capacity of Supporting Column Web

Basic requirement: **Tie Force** $\leq$ **Tying capacity of column web**



Tying capacity of column web $= \dfrac{8\,M_u}{1-\beta_1}\left[\eta_1 + 1.5(1-\beta_1)^{0.5}(1-\gamma_1)^{0.5}\right]$

M_u = moment capacity of column web per unit length

$= \dfrac{p_u\, t_w^2}{4}$

$= \dfrac{328 \times 13.8^2}{4 \times 10^3} \quad = \quad 15.6\text{kNm/mm}$

$\eta_1 = \dfrac{(n-1)\; p - \frac{n}{2}\; D_h}{d_c}$

$= \dfrac{(4-1) \times 70 - \frac{4}{2} \times 22}{246.7} \quad = \quad 0.673$

$\beta_1 = \dfrac{g}{d_c}$

$= \dfrac{90}{246.7} \quad = \quad 0.365$

$\gamma_1 = \dfrac{D_h}{d_c}$

$= \dfrac{22}{246.7} \quad = \quad 0.089$

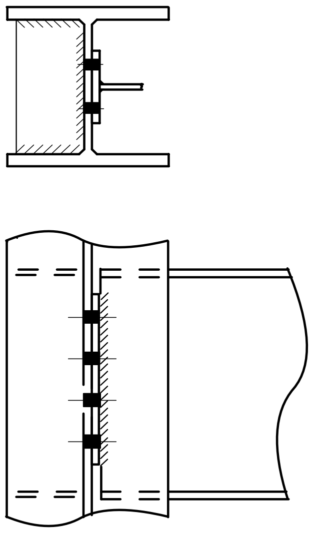
Tying capacity of column web $= \dfrac{8 \times 15.6}{1-0.365}\left[0.673 + 1.5(1-0.365)^{0.5} \times (1-0.089)^{0.5}\right]$

$= 196.5\left[0.673 + 1.195 \times 0.954\right] \quad = \quad 356\text{kN}$

Tie force = 175kN < 356kN **∴ O.K.**

Title	Example 2 - Flexible end plate - Beam to UC Web - Structural Integrity	Sheet	8 of 8

If column web fails to satisfy the criteria shown on sheet 7 then sfiffeners fillet welded on one side to the web and flanges would be required thus:



CALCULATION SHEET BCSA	Job: Joints in Steel Construction - Simple Connections		Sheet: 1 of 12
	Title: Example 3 - Flexible End Plates - Beam to RHS using Flowdrill		
	Client: SCI/BCSA Connections Group		
	Calcs by: RS	Checked by: AW / AM	Date: May 2002

DESIGN EXAMPLE 3

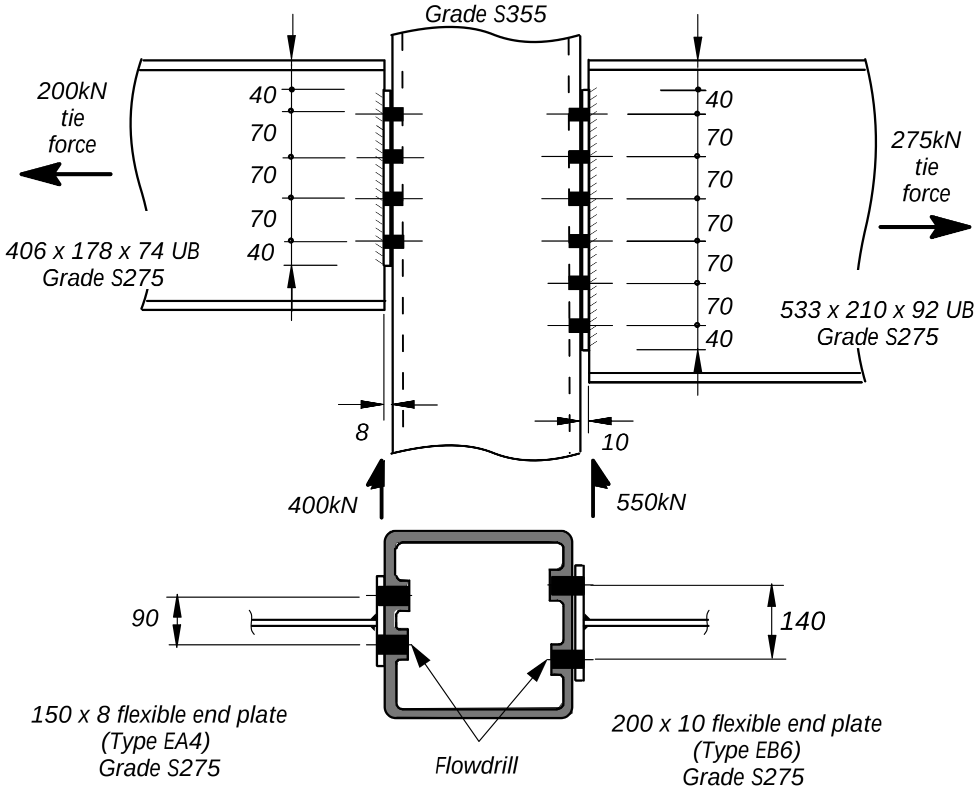
REF.

Check the following beam to RHS column connection for the design forces shown using Grade 8.8 bolts in Flowdrill threaded holes in the column.

In this example the tie force is less than the shear force. The tie force has been derived on the basis that the floor beams are supplemented as ties by the use of reinforcement bars located in the slab. It is important that the reinforcement is adequately anchored to the steelwork for it to be effective in resisting tie forces.

Note: *The connections should be checked independently for (i) Shear forces and (ii) Tie forces and NOT for both forces acting at the same time*

Yellow pages used for initial selection of end plates.



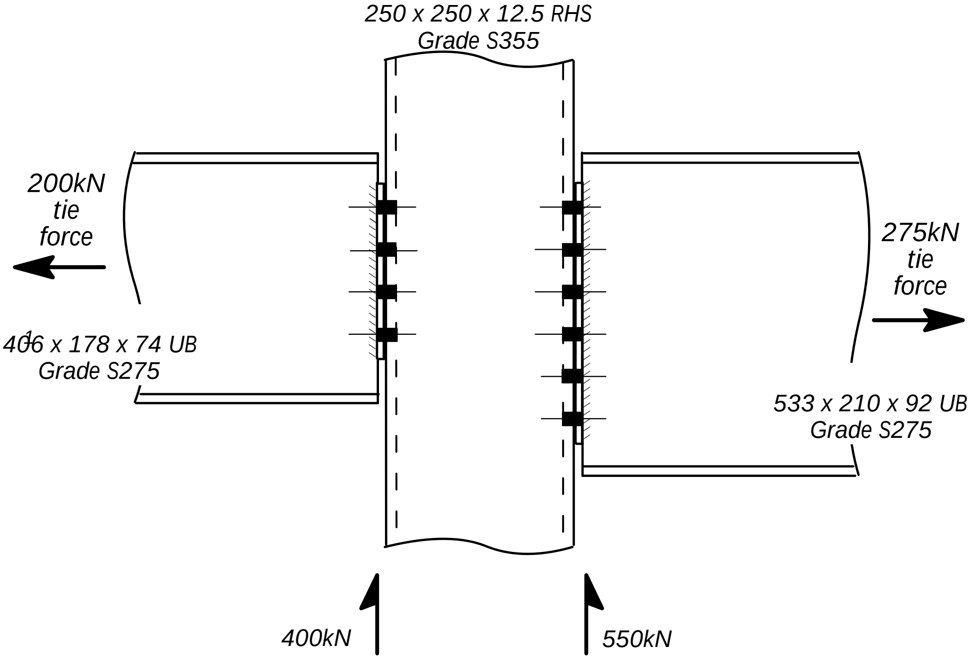
See Figure 5.5 and Yellow Pages Table H.3

Design Information:

Bolts:	M20 8.8, Flowdrill
Welds:	6mm fillet
Column:	S355
Beams:	S275
End plates:	S275

Title *Example 3 - Flexible End Plates - Beam to RHS using Flowdrill*	Sheet *2 of 12*

CONNECTION DESIGN USING CAPACITY TABLES FROM YELLOW PAGES



406 x 178 x 74 UB Grade S275
End plate type EA4
Welds 6mm fillet
Bolts M20 8.8
Bolts at 90 cross centres
4 rows of bolts
From Capacity Table H.20 in yellow pages
Connection shear capacity
= 409kN > 400kN

Minimum support thickness
= 4.6mm < 12.5mm

Connection tying capacity
= 239kN > 200kN ∴ O.K.

The beam side of the connection is adequate

533 x 210 x 92 UB Grade S275
End plate type EB6
Welds 6mm fillet
Bolts M20 8.8
Bolts at 140 cross centres
6 rows of bolts
From Capacity Table H.20 in yellow pages
Connection shear capacity
= 645kN > 550kN *Yellow Pages Table H.20* ***∴ O.K.***

Minimum support thickness
= 4.9mm < 12.5mm ***∴ O.K.***

Connection tying capacity
= 270kN $\not>$ 275kN ***∴ FAILS***

The beam side of the connection fails due to insufficient tying capacity. End plate thickness may be increased to 12mm.

Note:
(1) The tying capacity of the connection given in the tables in the yellow pages is the least value obtained from CHECKS 11, 12 and 13.
(2) Beams connecting to RHS wall must also be checked for RHS wall bending as shown in CHECK 15 (Sheet 11 & 12).

Title	Example 3 - Flexible End Plates - Beam to RHS using Flowdrill	Sheet 3 of 12

SUMMARY OF FULL DESIGN CHECKS FOR EXAMPLE 3

Notes (i) CHECKS 1 to 9, where applicable, are generally as shown in Example 1 and are not repeated in this example, but the calculated capacities are summarised below.

(ii) In accordance with BS 5950-1; tie forces are ignored when checking the capacity to resist vertical reactions and vertical reactions are ignored when calculating the capacity to resist tie forces.

(iii) Values shown ***** are different to Example 1. Example 1 uses 10mm plate whereas Example 3 uses 8mm plate.

Sheet Nos	CHECK		406UB S275		533UB S275		RHS Column S355 406UB Side		RHS Column S355 533UB Side	
			Capacity	Applied Load	Capacity	Applied Load	Capacity	Applied Load	Capacity	Applied Load
See Example 1 Sheets 4 to 11	**CHECK 1** Recommended detailing practice		All recommendations adopted							
	CHECK 2 Supported Beam- Weld Capacity (kN)		514	400	772	550	Not Applicable			
	CHECK 3		Not Applicable							
	CHECK 4 Supported Beam - Capacity at connection	Shear (kN)	**409**	**400**	**645**	**550**	Not Applicable			
			CRITICAL CHECK FOR VERTICAL SHEAR							
	CHECKS 5, 6, 7		Not Applicable				Not Applicable			
	CHECK 8 Supporting Column - Bolt Group shear capacity	Shear - bolt group (kN)	735	400	1103	550	Not Applicable			
	CHECK 9 Supporting Column - Connecting Element (Strength of plate)	Shear (kN) Bearing (Capacity per bolt line, kN)	320* 294*	200 200	589 552	275 275	Not Applicable			
4 & 5	**CHECK 10** Supporting Beam - Capacity (Local capacity of beam web)	Shear (kN) Bearing (per bolt, kN)	Not Applicable				940 138	200 50.0	1367 138	275 45.8
6 & 7	**CHECK 11** Structural Integrity - Connecting Elements Tension capacity of end plate	Tension (kN)	239	200	**270**	**275**	Not Applicable			
					FAILS (TIE FORCE)					
8 & 9	**CHECK 12** Structural Integrity - Supported Beam. Tension capacity of beam web	Tension (kN)	758	200	1194	275	Not Applicable			
10	**CHECK 13** Structural Integrity - Tension bolt group - Welds	Tension (kN)	Not Applicable				584 650	200 200	876 974	275 275
10	**CHECK 14**		Not Applicable				Not Applicable			
11&12	**CHECK 15** Structural Integrity - Supporting column wall	Tension (kN)	Not Applicable				400	200	787	275

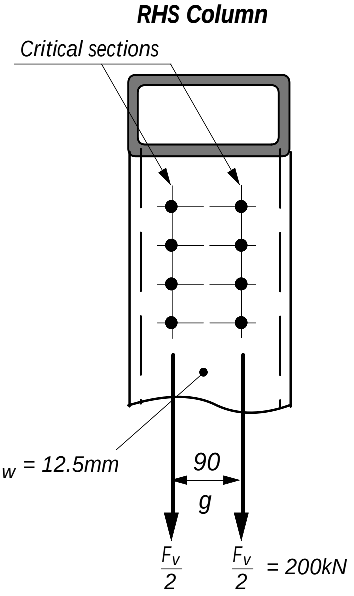
Title	Example 3 - Flexible End Plates - Beam to RHS using Flowdrill	Sheet	4 of 12

CHECK 10 : Supporting Column - Local Capacity

Shear & Bearing Capacity of column wall

(i) Basic requirement for shear: $\frac{F_v}{2} \leq P_v$

For 406 x 178 x 74 UB side



Shear capacity, P_v $= \min\left(0.6\, p_y\, A_v,\ 0.7\, p_y K_e\, A_{v.net}\right)$

Gross shear area $A_v = \left(e_t + (n-1)\,p + e_b\right)\, t_w$

e_b = smaller of $g/2$ and $5\,d$ = 45mm

e_t = smaller of e_{t1} and $5\,d$

since the connection is not near the top of column e_{t1} is not applicable

$e_t = 5d = 5 \times 20 = 100\text{mm}$

$A_v = \left(100 + (4-1)\,70 + 45\right) \times 12.5 = 4438\text{mm}^2$

$\therefore$ $0.6\, p_y A_v = \frac{0.6 \times 355 \times 4438}{10^3} = 945\text{kN}$

Net shear area, $A_{v.net} = A_v - n\, D_h\, t_w$

for Flowdrill connections D_h = bolt diameter = 20mm

$A_{v.net} = 4438 - (4 \times 20 \times 12.5) = 3438\text{mm}^2$

$\therefore$ $0.7 p_y K_e A_{v.net} = \frac{0.7 \times 355 \times 1.1 \times 3438}{10^3} = 940\text{kN}$

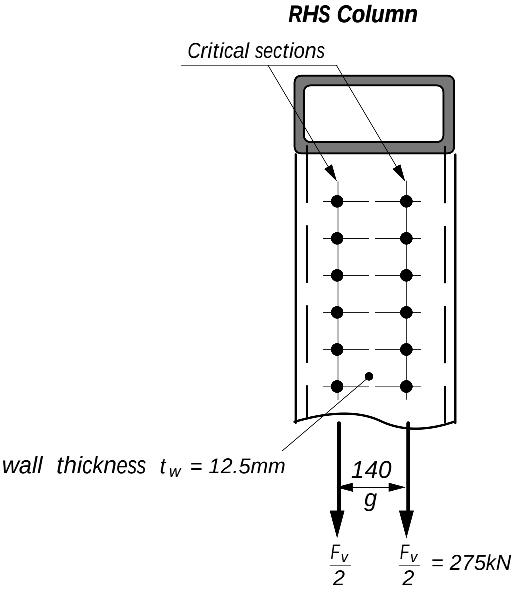
$P_v = 940\text{kN}$

$\frac{F_v}{2} = \frac{400}{2} = 200\text{kN}$

$\boldsymbol{\frac{F_v}{2} = 200\text{kN} < 940\text{kN}}$ **∴ O.K.**

Title	Example 3 - Flexible End Plates - Beam to RHS using Flowdrill	Sheet	5 of 12

For 533 x 210 x 92 UB side



e_b = smaller of $g/2$ and $5d$ = 70mm

e_t = $5d$ = 5 x 20 = 100mm

A_v = $(100 + (6-1)\ 70 + 70) \times 12.5$ = $6500mm^2$

$\therefore\ 0.6\ p_y\ A_v = \dfrac{0.6 \times 355 \times 6500}{10^3} = 1385kN$

$A_{v.net} = 6500 - (6 \times 20 \times 12.5) = 5000mm^2$

$\therefore\ 0.7\ p_y\ K_e\ A_{v.net} = \dfrac{0.7 \times 355 \times 1.1 \times 5000}{10^3} = 1367kN$

P_v = 1367kN

$\mathbf{\dfrac{F_v}{2} = 275kN < 1367kN}$ **∴ O.K.**

Note: *The above check is for **local** shear only; the effects of any global shear forces must also be considered.*

(ii) Basic requirement for bearing: $\dfrac{F_v}{2n} \leq P_{bs}$

Bearing capacity of column wall per bolt, $P_{bs} = k_{bs}\ d\ t_w\ p_{bs}$

$= \dfrac{1.0 \times 20 \times 12.5 \times 550}{10^3} = 138kN$ — p_{bs} from BS 5950-1 Table 32

For 406 x 178 x 74 UB side

$\mathbf{\dfrac{F_v}{2n} = \dfrac{400}{2 \times 4} = 50.0kN < 138kN}$ **∴ O.K.**

For 533 x 210 x 92 UB side

$\mathbf{\dfrac{F_v}{2n} = \dfrac{550}{2 \times 6} = 45.8kN < 138kN}$ **∴ O.K.**

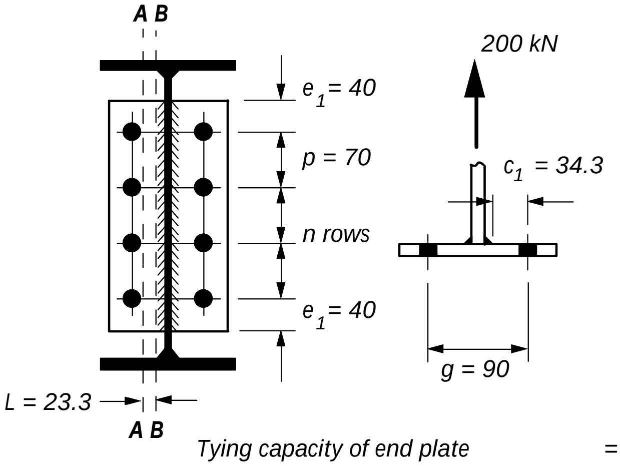
Title	Example 3 - Flexible End Plates - Beam to RHS using Flowdrill	Sheet	6 of 12

CHECK 11: Structural Integrity - Connecting Elements

Tension Capacity of End Plate

Basic requirement: **Tie Force** $\leq$ **Tying capacity of end plate**

For 406 x 178 x 74 UB side



s = leg length of weld = 6 mm

t_w = web thickness = 9.5mm

$c_1 = \frac{1}{2}(g - t_w - 2s)$

$= \frac{1}{2}(90 - 9.5 - 2 \times 6)$

$= 34.3\text{mm}$

Tying capacity of end plate $= \dfrac{2\,(M_{uA} + M_{uB})}{L}$

Moment capacity of end plate at section AA $\quad M_{uA} = \dfrac{p_u \; L_{eA} \; t_p^2}{4}$

p_u = design tensile strength = 328N/mm^2 $(U_s / 1.25)$

t_p = plate thickness = 8mm

L_{eA} = eff. length of plastic hinge at section AA

$= 2e_{eA} + (n-1)p_{eA}$

$e_{eA} = e_1$ but $\leq e_2$ = 30mm

$p_{eA} = p$ but $\leq 2e_2$ = 60mm

$\therefore L_{eA} = (2 \times 30) + (4-1)60 = 240\text{mm}$

$\therefore M_{uA} = \dfrac{328 \times 240 \times 8^2}{4 \times 10^3} = 1260\text{kNmm}$

Moment capacity of end plate at section BB $\quad M_{uB} = \dfrac{p_u \; L_{eB} \; t_p^2}{4}$

L_{eB} = eff. length of plastic hinge at section BB

$= 2e_{eB} + (n-l)p_{eB}$

For Flowdrill connections D_h = hole diameter in plate = 22mm — Yellow pages Table H.60

$e_{eB} = e_1$ but $\leq c_1 + D_h/2$ = 40mm

$c_1 + D_h/2 = 34.35 + 22/2 = 45.3\text{mm}$

$p_{eB} = p$ but $\leq 2c_1 + D_h$ = 70mm

$2c_1 + D_h = 2 \times 34.3 + 22 = 90.6\text{mm}$

$\therefore L_{eB} = (2 \times 40) + (4-1)70 = 290\text{mm}$

$\therefore M_{uB} = \dfrac{328 \times 290 \times 8^2}{4 \times 10^3} = 1522\text{kNmm}$

L = distance between plastic hinges = $c_1 - (D_h/2)$

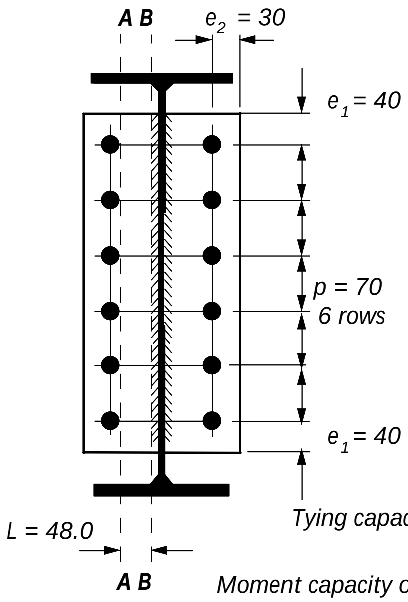
$= 34.3 - (22/2) = 23.3\text{mm}$

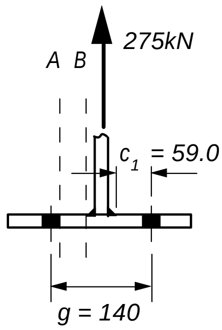
$\dfrac{2(M_{uA} + M_{uB})}{L} = \dfrac{2 \times (1260 + 1522)}{23.3} = 239\text{kN}$

Tie force = 200kN < 239kN $\quad$ **∴ O.K.**

Title		Sheet	
	Example 3 - Flexible End Plates - Beam to RHS using Flowdrill		*7 of 12*

For 533 x 210 x 92 UB side





s = leg length of weld

= 6 mm

t_w = web thickness

= 10.1mm

$$c_1 = \frac{1}{2}\,(g - t_w - 2s)$$

$$= \frac{1}{2}\,(140 - 10.1 - 2 \times 6)$$

= 59.0mm

Tying capacity of end plate $= \dfrac{2\,(M_{uA} + M_{uB})}{L}$

Moment capacity of end plate at section AA $\quad M_{uA} = \dfrac{p_u\;L_{eA}\;t_p^2}{4}$

p_u = design tensile strength = 328N/mm^2 $\quad (U_s / 1.25)$

t_p = plate thickness = 10mm

L_{eA} = eff. length of plastic hinge at section AA

$= 2e_{eA} + (n - 1)p_{eA}$

$e_{eA} = e_1$ but $\leq e_2$ = 30mm

$p_{eA} = p$ but $\leq 2e_2$ = 60mm

$\therefore L_{eA} = (2 \times 30) + (6 - 1)60$ = 360mm

$$\therefore M_{uA} = \frac{328 \times 360 \times 10^2}{4 \times 10^3} = 2952\text{kNmm}$$

Moment capacity of end plate at section BB $\quad M_{uB} = \dfrac{p_u\;L_{eB}\;t_p^2}{4}$

L_{eB} = eff. length of plastic hinge at section BB

$= 2e_{eB} + (n - 1)p_{eB}$

For Flowdrill connections D_h = hole diameter in plate = 22mm — *Yellow pages Table H.60*

$e_{eB} = e_1$ but $\leq c_1 + D_h/2$ = 40mm

$c_1 + D_h/2$ = 59.0 + 22/2 = 70.0mm

$p_{eB} = p$ but $\leq 2c_1 + D_h$ = 70mm

$2c_1 + D_h$ = $2 \times 59.0 + 22$ = 140mm

$\therefore L_{eB} = (2 \times 40) + (6 - 1)70$ = 430mm

$$\therefore M_{uB} = \frac{328 \times 430 \times 10^2}{4 \times 10^3} = 3526\text{kNmm}$$

L = distance between plastic hinges = $c_1 - (D_h/2)$

= 59.0 – 22/2 = 48.0mm

$$\frac{2(M_{uA} + M_{uB})}{L} = \frac{2 \times (2952 + 3526)}{48} = 270\text{kN}$$

Tie force = 275kN $\nleq$ 270kN

End plate FAILS thickness may be increased to 12mm

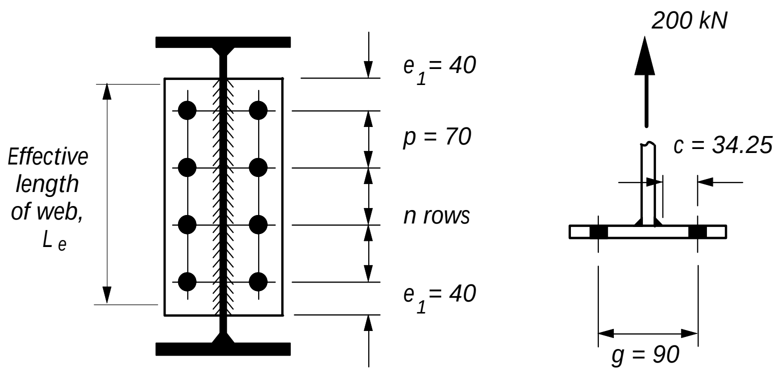
Title	Example 3 - Flexible End Plates - Beam to RHS using Flowdrill	Sheet	8 of 12

CHECK 12: Structural Integrity - Supported Beam

Tension Capacity of Beam Web

(i) Basic requirement: **Tie force $\leq$ Tension capacity of beam web**

For 406 x 178 x 74 UB Grade S275



$c_1 = 0.5(g - t_w - 2s)$

s = leg length of weld = 6mm

t_w = web thickness = 9.5mm

Tension capacity of beam web = $L_e \; t_w \; p_y$

Effective length, $L_e = 2e_e + (n - 1)\, p_e$

For Flowdrill connections D_h = hole diameter in plate = 22mm

$e_e = e_1$ but $\leq c_1 + D_h/2$

$c_1 + D_h/2 = 34.3 + 22/2 = 45.3\text{mm}$

$e_e = 40\text{mm}$

$p_e = p$ but $\leq 2c_1 + D_h$

$2c_1 + D_h = 2 \times 34.3 + 22 = 90.6\text{mm}$

$p_e = 70\text{mm}$

$L_e = (2 \times 40) + (4 - 1)\, 70$

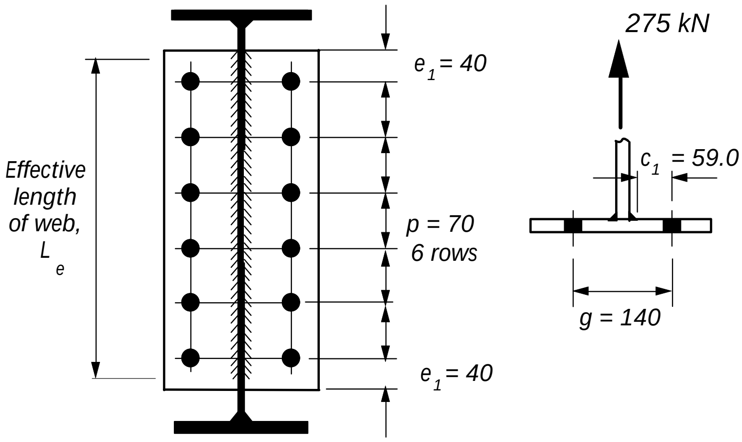
$= 290\text{mm}$

$\therefore \quad L_e\, t_w\, p_y = \dfrac{290 \times 9.5 \times 275}{10^3} = 758\text{kN}$

Tie force = 200kN < 758kN **∴ O.K.**

Title	Example 3 - Flexible End Plates - Beam to RHS using Flowdrill	Sheet	9 of 12

For 533 x 210 x 92 UB Grade S275

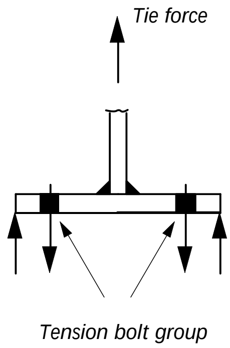


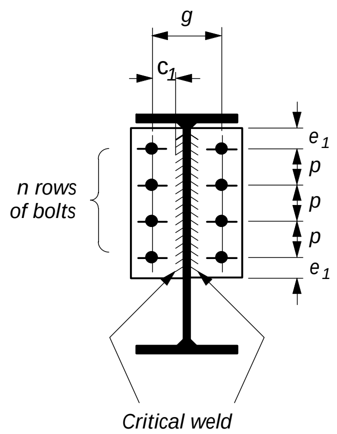
	c_1	=	$0.5\ (g - t_w - 2s)$		
	s	=	leg length of weld	=	6mm
	t_w	=	web thickness	=	10.1mm
Tension capacity of beam web		=	$L_e\ \ t_w\ \ p_y$		
Effective length, L_e		=	$2e_e + (n - 1)\ p_e$		
For Flowdrill connections D_h		=	hole diameter in plate	=	22mm
	e_e	=	e_1 but $\leq$ $c_1 + D_h/2$		
	$c_1 + D_h/2$	=	$59.0 + 22/2$	=	70.0mm
	e_e	=	40mm		
	p_e	=	p but $\leq$ $2c_1 + D_h$		
	$2c_1 + D_h$	=	$2 \times 59.0 + 22$	=	140mm
	p_e	=	70mm		
	L_e	=	$(2 \times 40) + (6 - 1)\ 70$		
		=	430mm		
$\therefore$	$L_e\ t_w\ p_y$	=	$\dfrac{430 \times 10.1 \times 275}{10^3}$	=	1194kN

Tie force = 275kN < 1194kN ***$\therefore$ O.K.***

Title	Example 3 - Flexible End Plates - Beam to RHS using Flowdrill	Sheet	10 of 12

CHECK 13 : Structural Integrity - Tension Capacity of Bolts and Welds





(i) Tension capacity of bolts

Basic requirement: **Tie Force** $\leq$ **Tension capacity of bolt group**

Flowdrill Pull-out Capacity Yellow pages Table H.55b

Tension capacity of bolt group $= 2\,n\,P_{si}$

P_{si} = Flowdrill Structural integrity Tensile Capacity taken from the Yellow pages

= 73kN for a grade 8.8 bolt in a 12.5mm RHS column wall

For 406 x 178 x 74 UB Side

Tension capacity of bolt group $= 2 \times 4 \times 73 = 584\text{kN}$

Tie Force = 200kN < 584kN ∴ **O.K.**

For 533 x 210 x 92 UB Side

Tension capacity of bolt group $= 2 \times 6 \times 73 = 876\text{kN}$

Tie force = 275kN < 876kN ∴ **O.K.**

(ii) Weld Tension Capacity

Basic requirement: **Tie force** < **Tie capacity of weld**

Tension capacity of weld $= 2\,(1.25\,p_w\,a\,(L_e - 2s))$

For 406 x 178 x 74 UB Side

L_e from Check 12 Sheet 8
p_w from BS 5950-1 Table 37

L_e = 290mm

p_w = 220N/mm2

$a = 0.7\,s = 0.7 \times 6 = 4.2\text{mm}$

∴ Tension capacity of weld $= \dfrac{2 \times (1.25 \times 220 \times 4.2 \times (290 - 2 \times 6))}{10^3}$

= 642kN

Tie force = 200kN < 642kN ∴ **O.K.**

For 533 x 210 x 92 UB Side

L_e from Check 12 Sheet 9

L_e = 430mm

∴ Tension capacity of weld $= \dfrac{2 \times (1.25 \times 220 \times 4.2 \times (430 - 2 \times 6))}{10^3}$

= 966kN

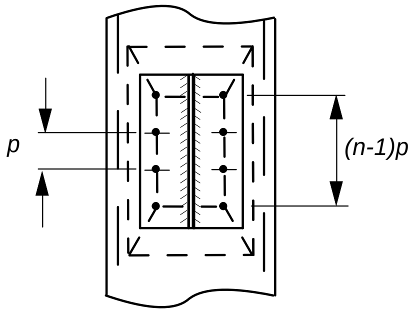
Tie force = 275kN < 966kN ∴ **O.K.**

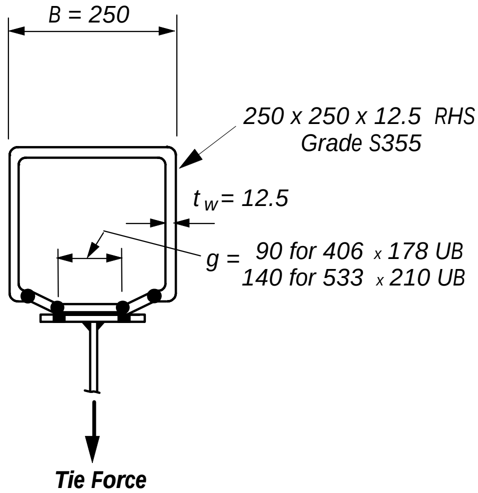
CHECK 14: not applicable

Title	Example 3 - Flexible End Plates - Beam to RHS using Flowdrill	Sheet	11 of 12

CHECK 15: Structural Integrity – Capacity of Supporting Column Wall (RHS)

Basic requirement: **Tie Force $\leq$ Tying capacity of RHS column wall**





For 406 x 178 x 74 UB Side

Tying capacity of column wall $= \dfrac{8\,M_u}{1-\beta_1}\left[\eta_1 + 1.5(1-\beta_1)^{0.5}(1-\gamma_1)^{0.5}\right]$

M_u = moment capacity of RHS column wall per unit length

$= \dfrac{p_u\, t_w^2}{4}$

$= \dfrac{392 \times 12.5^2}{4 \times 10^3} = 15.3\text{kNm/mm}$

D_h = 20mm (Bolt diameter for Flowdrill)

$\eta_1 = \dfrac{(n-1)\ p - \frac{n}{2}\ D_h}{B - 3t_w}$

$= \dfrac{(4-1)\ 70 - \frac{4}{2} \times 20}{250 - 3 \times 12.5} = 0.8$

$\beta_1 = \dfrac{g}{B-3t_w}$

$= \dfrac{90}{212.5} = 0.424$

$\gamma_1 = \dfrac{D_h}{B-3t_w}$

$= \dfrac{20}{212.5} = 0.094$

Tying capacity of column wall $= \dfrac{8 \times 15.3}{1-0.424}\left[0.8 + 1.5(1-0.424)^{0.5} \times (1-0.094)^{0.5}\right]$

$= 212.5\left[0.8 + 1.5 \times 0.759 \times 0.952\right] = 400\text{kN}$

Tie force = 200N < 400kN **∴ O.K.**

Title	*Example 3 - Flexible End Plates - Beam to RHS using Flowdrill*	Sheet	*12 of 12*

For 533 x 210 x 92 UB Side

$$\eta_1 = \frac{(n-1)\ p - \frac{n}{2}\ D_h}{B - 3t_w}$$

$$= \frac{(6-1)\ 70 - \frac{6}{2} \times 20}{250 - 3 \times 12.5} = 1.36$$

$$\beta_1 = \frac{g}{B - 3t_w}$$

$$= \frac{140}{212.5} = 0.659$$

Tying capacity of column wall $= \dfrac{8\ M_u}{1 - \beta_1} \left[\eta_1 + 1.5(1 - \beta_1)^{0.5}\ (1 - \gamma_1)^{0.5} \right]$

$$= \frac{8 \times 15.3}{1 - 0.659} \left[1.36 + 1.5(1 - 0.659)^{0.5} \times (1 - 0.094)^{0.5} \right]$$

$$= 358.9 \left[1.36 + 1.5 \times 0.584 \times 0.952 \right]$$

$$= 787kN$$

Tie force = 275kN < 787kN **∴ O.K.**

CALCULATION SHEET BCSA	Job: Joints in Steel Construction - Simple Connections		Sheet: 1 of 15
	Title: Example 4 - Flexible End Plates - Beam to RHS column using Hollo-Bolt		
	Client: SCI/BCSA Connections Group		
	Calcs by: RS	Checked by: AW / AM	Date: May 2002

REF.

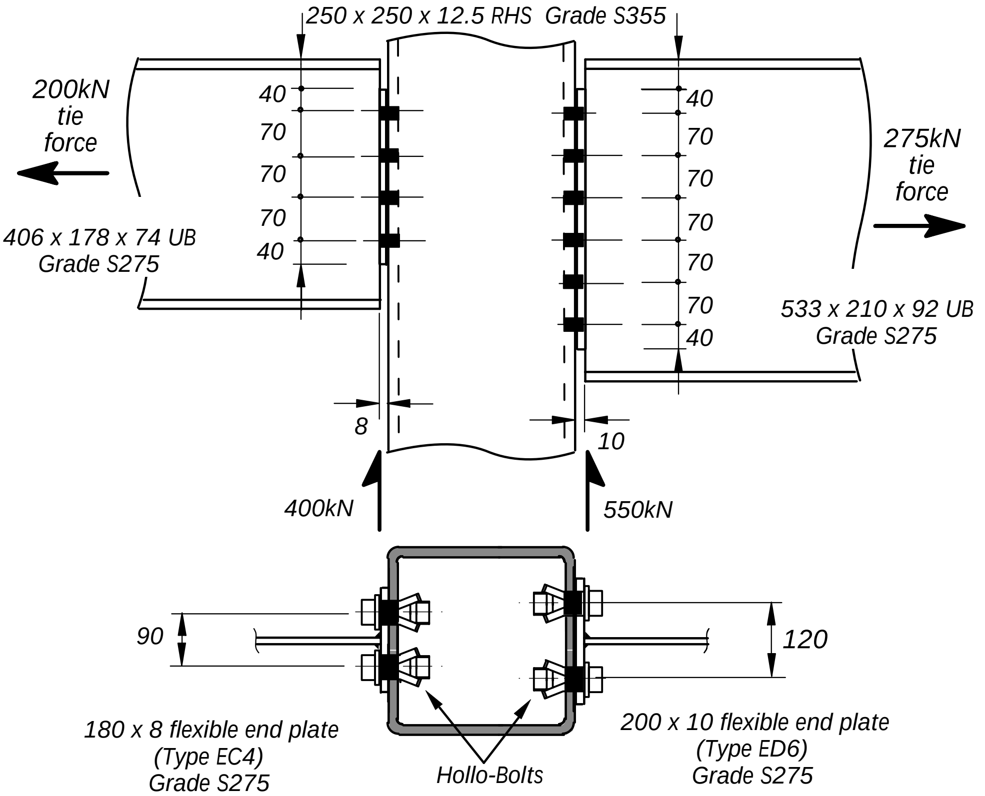
DESIGN EXAMPLE 4

Check the following beam to RHS column connection for the design forces shown using Hollo-Bolt connectors to the column.

In this example the tie force is less than the shear force. The tie force has been derived on the basis that the floor beams are supplemented as ties by the use of reinforcement bars located in the slab. It is important that the reinforcement is adequately anchored to the steelwork for it to be effective in resisting tie forces.

Note: The connections should be checked independently for (i) Shear forces and (ii) Tie forces and NOT for both forces acting at the same time

Yellow pages used for initial selection of end plates.



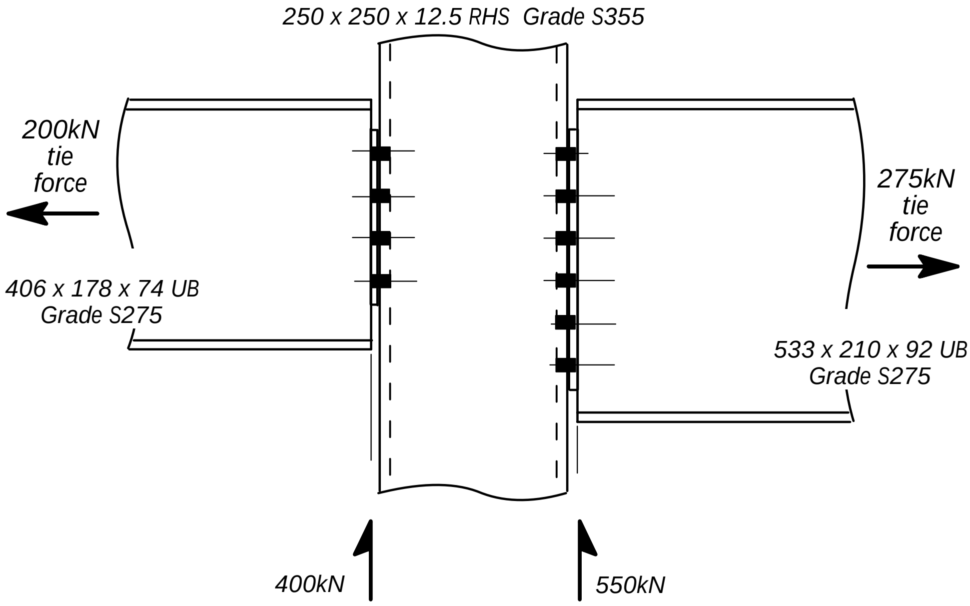
See figure 5.5 and Yellow pages Table H.3

Design Information:

Bolts:	*M20 8.8, Hollo-Bolts*
Welds:	*6mm fillet*
Column:	*S355*
Beams:	*S275*
End plates:	*S275*

Title	*Example 4 - Flexible End Plates - Beam to RHS column using Hollo-Bolt*	Sheet *2 of 15*

CONNECTION DESIGN USING CAPACITY TABLES FROM YELLOW PAGES



406 x 178 x 74 UB Grade S275	533 x 210 x 92 UB Grade S355	
End plate type EC4	***End plate type ED6***	
Welds 6mm fillet	***Welds 6mm fillet***	
Bolts M20 8.8	***Bolts M20 8.8***	
Bolts at 90 cross centres	*Bolts at 120 cross centres*	
4 rows of bolts	*4 rows of bolts*	
From Capacity Table H.22 in yellow pages	*From Capacity Table H.22 in yellow pages*	*Yellow Pages Table H.22*
Connection shear capacity	*Connection shear capacity*	
= 409kN > 400kN	***= 645kN > 550kN***	***∴ O.K.***
Minimum support thickness	*Minimum support thickness*	
= 4.8mm < 12.5mm	***= 4.9mm < 12.5mm***	***∴ O.K.***
Connection tying capacity	*Connection tying capacity*	
= 363kN > 200kN	***= 448kN > 275kN***	***∴ O.K.***
The beam side of the connection is adequate	***The beam side of the connection is adequate***	

Note:

(1) The tying capacity of the connection given in the tables in the yellow pages is the least value obtained from CHECKS 11, 12 and 13.

(2) Beams connecting to RHS wall must also be checked for RHS wall bending as shown in CHECK 15.

Title	Example 4 - Flexible End Plates - Beam to RHS column using Hollo-Bolt	Sheet	2 of 15

SUMMARY OF FULL DESIGN CHECKS FOR EXAMPLE 4

Notes (i) *Checks 1 to 4, where applicable are all as shown in Example 1 and are not repeated in this example but the calculated capacities are summarised below.*

(ii) *In accordance with BS 5950-1; tie forces are ignored when checking the capacity to resist vertical reactions and vertical reactions are ignored when calculating the capacity to resist tie forces.*

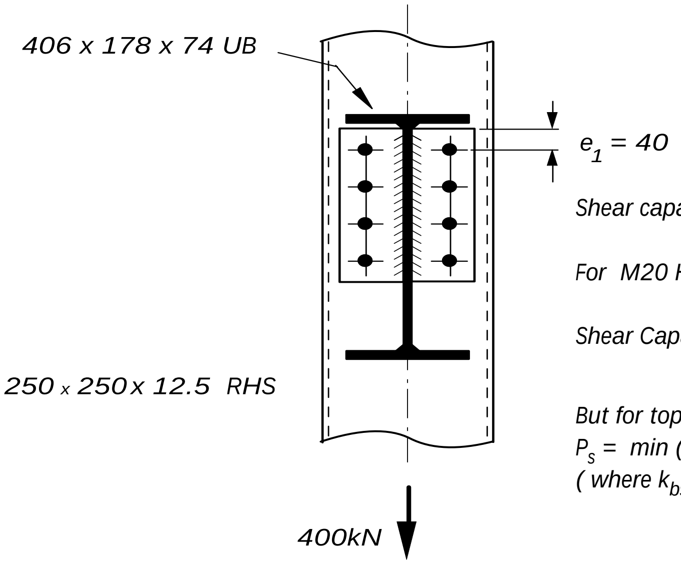
Sheet Nos	CHECK		406UB (S275)		533UB (S275)		RHS Column (S355)			
							406UB Side		533UB Side	
			Capacity	Applied Load	Capacity	Applied Load	Capacity	Applied Load	Capacity	Applied Load
See Example 1 Sheets 4 to 5	**CHECK 1** - Recommended detailing practice		All recommendations adopted							
	CHECK 2 Supported Beam - Weld Capacity (kN)		514	400	772	550	Not Applicable			
	CHECK 3		Not Applicable				Not Applicable			
	CHECK 4 Supported Beam - Capacity at connection	Shear (kN)	**409**	**400**	**645**	**550**	Not Applicable			
			CRITICAL CHECK, VERTICAL SHEAR							
	CHECK 5, 6, 7		Not Applicable				Not Applicable			
4	**CHECK 8** Supporting Column - Bolt Group shear capacity	Shear - bolt group (kN)	747	400	1184	550	Not Applicable			
5 & 6	**CHECK 9** Supporting Column - Connecting Element (Strength of plate)	Shear (kN)	224	200	409	275	Not Applicable			
		Bearing (Capacity per bolt line, kN)	294	200	552	275				
7 & 8	**CHECK 10** Supporting Beam - Capacity (Local capacity of beam web)	Shear (kN)	Not Applicable				735	200	1025	275
		Bearing (Capacity per bolt,kN)					138	50.0	138	45.8
9 & 10	**CHECK 11** Structural Integrity - Connecting Elements Tension capacity of end plate	Tension (kN)	**362**	**200**	**448**	**275**	Not Applicable			
			CRITICAL CHECK, TIE FORCE							
11 & 12	**CHECK 12** Structural Integrity -Supported Beam Tension capacity of beam web	Tension (kN)	758	200	1194	275	Not Applicable			
13	**CHECK 13** Structural Integrity - Tension bolt group	Tension (kN)	Not Applicable				584	200	876	275
	- Welds						650	200	974	275
13	**CHECK 14**		Not Applicable				Not Applicable			
14 & 15	**CHECK 15** Structural Integrity - Supporting column wall	Tension (kN)	Not Applicable				361	200	578	275

Title	Example 4 - Flexible End Plates - Beam to RHS column using Hollo-Bolt	Sheet 4 of 15

CHECK 8: Supporting Column - Bolt Group

Basic requirement: $F_v \leq \Sigma P_s$

For 406 x 178 x 74 UB side



Shear capacity of single bolt, $P_s = p_s A_s$

For M20 Hollo-Bolts

Shear Capacity P_s = 100kN — p_s from Yellow pages Table H.56

But for top pair of bolts
$P_s = \min(p_s A_s,\ 0.5\, k_{bs}\, e_1\, t_p\, p_{bs})$
(where $k_{bs} = 1.0$)

$$0.5\, k_{bs}\, e_1\, t_p\, p_{bs} = \frac{0.5 \times 1.0 \times 40 \times 8 \times 460}{10^3} = 73.6\text{kN}$$

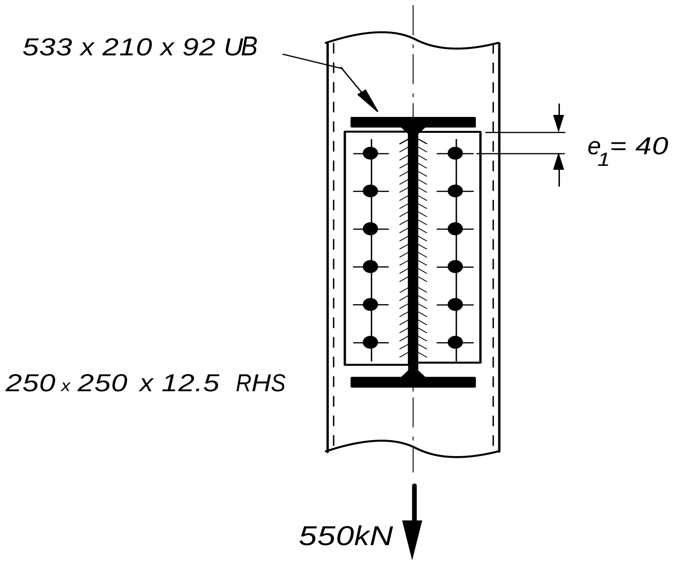
p_{bs} from BS 5950-1 Table 32

$$\Sigma P_s = 6 \times 100 + 2 \times 73.6 = 747\text{kN}$$

$$\mathbf{F_v = 400kN < 747kN}$$

∴ O.K.

For 533 x 210 x 92 UB side



$$0.5\, k_{bs}\, e_1\, t_p\, p_{bs} = \frac{0.5 \times 1.0 \times 40 \times 10 \times 460}{10^3} = 92\text{ kN}$$

p_{bs} from BS 5950-1 Table 32

$$\Sigma P_s = 10 \times 100 + 2 \times 92 = 1184\text{kN}$$

$$\mathbf{F_v = 550kN < 1184kN}$$

∴ O.K.

Title	*Example 4 - Flexible End Plates - Beam to RHS column using Hollo-Bolt*	Sheet	*5 of 15*

CHECK 9: Supporting Column - Connecting elements

Shear and bearing of end plates connected to supporting column

(i) Basic requirement for shear: $\frac{F_v}{2} \leq P_{v.min}$

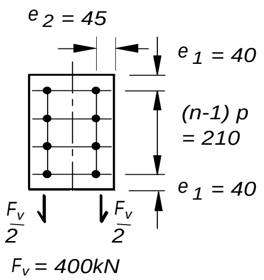
For 406 x 178 x 74 UB

Shear capacity of end plate $P_{v.min}$ is the smaller of plain shear capacity P_v and block shear capacity P_r

Plain Shear *capacity* $P_v = \min(0.6\, p_y\, A_v \ ,\ 0.7\, p_y K_e A_{v.net})$

Shear area, $A_v = 0.9\,(2e_1 + (n-1)\,p)\, t_p$

$= 0.9\,(80 + 210) \times 8 = 2088\text{mm}^2$



Net area, $A_{v.net} = A_v - n\, D_h\, t_p$

for Hollo-Bolt connections $D_h = 35\text{mm}$ — *D_h from Yellow pages Table H.3*

$A_{v.net} = 2088 - (4 \times 35 \times 8) = 968\text{mm}^2$

$0.6\, p_y\, A_v = \frac{0.6 \times 275 \times 2088}{10^3} = 344\text{kN}$

$0.7\, p_y K_e A_{v.net} = \frac{0.7 \times 275 \times 1.2 \times 968}{10^3} = 224\text{kN}$

Plain shear capacity $P_v = 224\text{kN}$

Block Shear *capacity* $P_r = 0.6 p_y\, t_p\, (L_v + K_e\,(L_t - kD_h))$

$L_v = e_1 + (n-1)p = 40 + 210 = 250\text{mm}$ — *See Note below*

$L_t = e_2 = 45\text{mm}$

$k = 0.5$

$K_e = 1.2$ *(for S275)*

$\therefore\ P_r = \frac{0.6 \times 275 \times 8 \times (250 + 1.2(45 - 0.5 \times 35))}{10^3}$

$= 374\text{kN}$

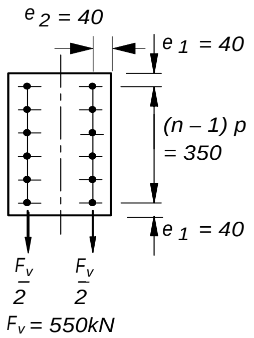
$P_{v.min} = \min(P_v, P_r)$

$= 224\text{kN}$

$F_v/2$ **= 200kN < 224kN** — **∴ O.K.**

For 533 x 210 x 92 UB side

$A_v = 0.9(80 + 350) \times 10 = 3870\text{mm}^2$



$A_{v.net} = 3870 - (6 \times 35 \times 10) = 1770\text{mm}^2$

$0.6\, p_y\, A_v = \frac{0.6 \times 275 \times 3870}{10^3} = 639\text{kN}$

$0.7\, p_y K_e A_{v.net} = \frac{0.7 \times 275 \times 1.2 \times 1770}{10^3} = 409\text{kN}$

$\therefore P_v = 409\text{kN}$

$P_r = \frac{0.6 \times 275 \times 10 \times (390 + 1.2(40 - 0.5 \times 35))}{10^3}$ — *See Note below*

$= 688\text{kN}$

$P_{v.min} = \min(P_v, P_r)$

$= 409\text{kN}$

$F_v/2$ **= 275kN < 409kN** — **∴ O.K.**

Note: *Block shear checks have been shown here, but they are never critical for well proportioned end plates. However, if the bolt spacing is concentrated at one part of a plate then these checks may be critical.*

Title	Example 4 - Flexible End Plates - Beam to RHS column using Hollo-Bolt	Sheet	6 of 15

(ii) Basic requirement for bearing: $\frac{F_v}{2} \leq \Sigma P_{bs}$

For 406 x 178 x 74 UB side

Bearing capacity, $P_{bs} = k_{bs}\, d\, t_p\, p_{bs}$

but for the top bolt, $P_{bs} = \min(k_{bs}\, d\, t_p\, p_{bs}\ ,\ 0.5k_{bs}\, e_1\, t_p\, p_{bs})$

for Hollo-bolt connections d = nominal bolt diameter = 20mm

$$k_{bs}\, d\, t_p\, p_{bs} = \frac{1.0 \times 20 \times 8 \times 460}{10^3} = 73.6\text{kN}$$

$$0.5\, k_{bs}\, e_1\, t_p\, p_{bs} = \frac{0.5 \times 1.0 \times 40 \times 8 \times 460}{10^3} = 73.6\text{kN}$$

p_{bs} from BS 5950-1 Table 32

$$\Sigma P_{bs} = 3 \times 73.6 + 73.6 = 294\text{kN}$$

$$\frac{F_v}{2} = 200\text{kN} < 294\text{kN}$$

∴ O.K.

For 533 x 210 x 92 UB side

Bearing capacity, $P_{bs} = k_{bs}\, d\, t_p\, p_{bs}$

but for the top bolt, $P_{bs} = \min(k_{bs}\, d\, t_p\, p_{bs}\ ,\ 0.5k_{bs}\, e_1\, t_p\, p_{bs})$

$$k_{bs}\, d\, t_p\, p_{bs} = \frac{1.0 \times 20 \times 10 \times 460}{10^3} = 92\text{kN}$$

$$0.5\, k_{bs}\, e_1\, t_p\, p_{bs} = \frac{0.5 \times 1.0 \times 40 \times 10 \times 460}{10^3} = 92\text{kN}$$

$$\Sigma P_{bs} = 5 \times 92 + 92 = 552\text{kN}$$

$$\frac{F_v}{2} = 275\text{kN} < 552\text{kN}$$

∴ O.K.

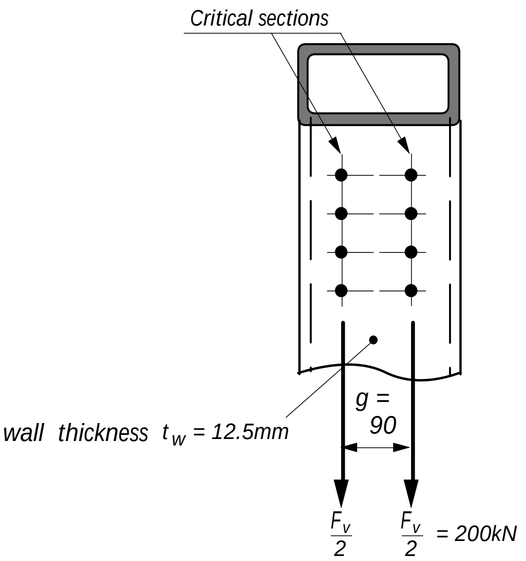
Title	Example 4 - Flexible End Plates - Beam to RHS column using Hollo-Bolt	Sheet	7 of 15

CHECK 10 : Supporting Column - Local Capacity

Shear & Bearing Capacity of column wall

(i) Basic requirement for shear: $\frac{F_v}{2} \leq P_v$

For 406 x 178 x 74 UB side



Shear capacity, $P_v = \min(0.6\, p_y A_v,\ 0.7\, p_y K_e A_{v.net})$

Gross shear area, $A_v = (e_t + (n-1)\,p + e_b)\ t_w$

for Hollo-Bolt connection d = bolt diameter = 20mm

e_b = smaller of $g/2$ and $5\,d$ = 45mm

e_t = smaller of e_{t1} and $5\,d$

since the connection is not near the top of column e_{t1} is not applicable

$e_t = 5d = 5 \times 20 = 100\text{mm}$

$A_v = (100 + (4-1)\,70 + 45) \times 12.5 = 4438\text{mm}^2$

$\therefore\ 0.6\, p_y A_v = \dfrac{0.6 \times 355 \times 4438}{10^3} = 945\text{kN}$

Net shear area, $A_{v.net} = A_v - n\ D_h\, t_w$

for Hollo-Bolt connections D_h = hole diameter = 35mm — D_h from Yellow pages Table H.61

$A_{v.net} = 4438 - (4 \times 35 \times 12.5) = 2688\text{mm}^2$

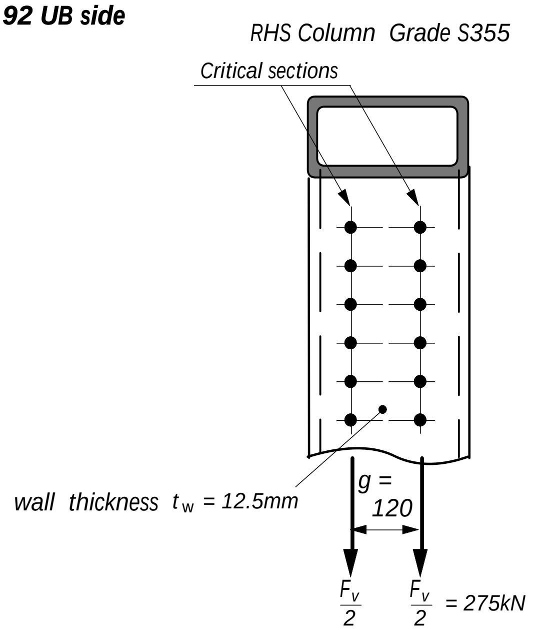
$\therefore\ 0.7\, p_y K_e A_{v.net} = \dfrac{0.7 \times 355 \times 1.1 \times 2688}{10^3} = 735\text{kN}$

$\therefore\ P_v = 735\text{kN}$

$\frac{F_v}{2}$ **= 200kN < 735kN** — **∴ O.K.**

Title	Example 4 - Flexible End Plates - Beam to RHS column using Hollo-Bolt	Sheet	8 of 15

For 533 x 210 x 92 UB side



e_b = smaller of $g/2$ and $5d$ = 60mm

e_t = $5d$ = 5 x 20 = 100mm

A_v = $(100 + (6-1)\,70 + 60) \times 12.5$ = 6375mm^2

∴ $0.6\,p_y\,A_v = \dfrac{0.6 \times 275 \times 6375}{10^3}$ = 1052kN

$A_{v.net}$ = $6375 - (6 \times 35 \times 12.5)$ = 3750mm^2

∴ $0.7\,p_y\,K_e\,A_{v.net} = \dfrac{0.7 \times 355 \times 1.1 \times 3750}{10^3}$ = 1025kN

∴ P_v = 1025kN

$\dfrac{F_v}{2}$ **= 275kN < 1025kN** **∴ O.K.**

Note: *The above check is for* ***local*** *shear only; the effects of any global shear forces must also be considered.*

(ii) Basic requirement for bearing: $\dfrac{F_v}{2n} \leq P_{bs}$

Bearing capacity of column wall per bolt P_{bs} = $k_{bs} d\, t_w\, p_{bs}$
(where k_{bs} = 1.0)

p_{bs} from BS 5950-1 Table 32

$= \dfrac{1.0 \times 20 \times 12.5 \times 550}{10^3}$ = 138kN

For 406 x 178 x 74 UB side

$\dfrac{F_v}{2n} = \dfrac{400}{2 \times 4}$ **= 50kN < 138kN** **∴ O.K.**

For 533 x 210 x 92 UB side

$\dfrac{F_v}{2n} = \dfrac{550}{2 \times 6}$ **= 45.8kN < 138kN** **∴ O.K.**

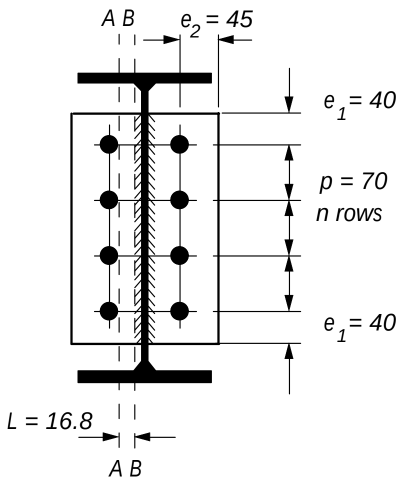
Title	*Example 4 - Flexible End Plates - Beam to RHS column using Hollo-Bolt*	Sheet	*9 of 15*

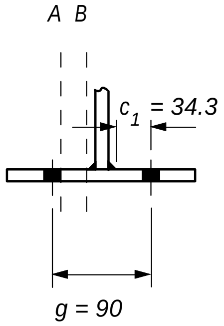
CHECK 11: Structural Integrity - Connecting Elements

Tension Capacity of End Plate

Basic requirement: **Tie Force** $\leq$ **Tying capacity of end plate**

For 406 x 178 x 74 UB side





s = leg length of weld
= 6 mm

t_w = web thickness
= 9.5mm

$$c_1 = \frac{1}{2}(g - t_w - 2s)$$
$$= \frac{1}{2}(90 - 9.5 - 2 \times 6)$$
$$= 34.3\text{mm}$$

$$L = c_1 - D_h/2$$
$$= 34.3 - 35/2 = 16.8\text{mm}$$

Tying capacity of end plate $= \dfrac{2\,(M_{uA} + M_{uB})}{L}$

Moment capacity of end plate at section AA $\quad M_{uA} = \dfrac{p_u\ L_{eA}\ t_p^2}{4}$

p_u = design tensile strength = 328N/mm^2 (U_s / 1.25) — *Steel strengths Yellow pages Table H.45*

t_p = plate thickness = 8mm

L_{eA} = eff. length of plastic hinge at section AA
= $2e_{eA} + (n - 1)p_{eA}$

e_{eA} = e_1 but $\leq$ e_2 = 40mm

p_{eA} = p but $\leq$ $2e_2$ = 70mm

$\therefore$ L_{eA} = $(2 \times 40) + (4 - 1)70$ = 290mm

$$\therefore\ M_{uA} = \frac{328 \times 290 \times 8^2}{4 \times 10^3} = 1522\text{kNmm}$$

Moment capacity of end plate at section BB $\quad M_{uB} = \dfrac{p_u\ L_{eB}\ t_p^2}{4}$

L_{eB} = eff. length of plastic hinge at section BB
= $2e_{eB} + (n - 1)p_{eB}$

e_{eB} = e_1 but $\leq$ $c_1 + D_h/2$ = 40mm

D_h = 35mm (Hole diameter for Hollo-Bolt) — *D_h from Yellow Pages Table H.61*

p_{eB} = p but $\leq$ $2c_1 + D_h$ = 70mm

$\therefore$ L_{eB} = $(2 \times 40) + (4 - 1)70$ = 290mm

$$\therefore\ M_{uB} = \frac{328 \times 290 \times 8^2}{4 \times 10^3} = 1522\text{kNmm}$$

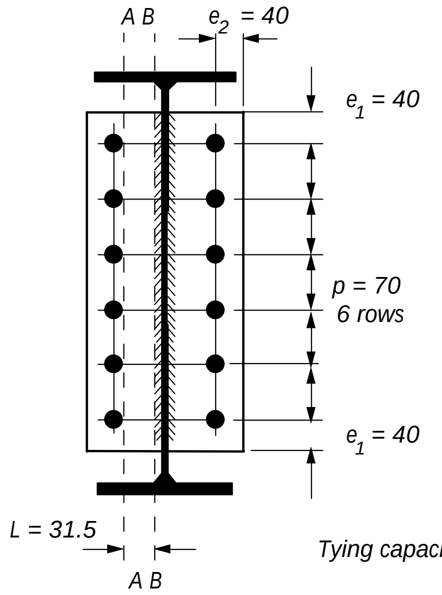
L = distance between plastic hinges = $c_1 - (D_h/2)$
= $34.3 - (35/2)$ = 16.8mm

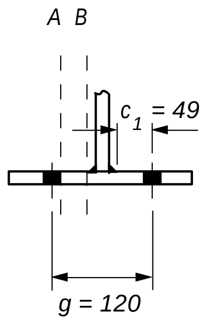
$$\frac{2(M_{uA} + M_{uB})}{L} = \frac{2(1522 + 1522)}{16.8} = 362\text{kN}$$

Tie force = 200N < 362kN $\quad$ **$\therefore$ O.K.**

Title	Example 4 - Flexible End Plates - Beam to RHS column using Hollo-Bolt	Sheet	10 of 15

For 533 x 210 x 92 UB side





s = leg length of weld
= 6 mm

t_w = web thickness
= 10.1mm

$c_1 = \frac{1}{2}(g - t_w - 2s)$

$= \frac{1}{2}(120 - 10.1 - 2 \times 6)$

= 49mm

Tying capacity of end plate $= \frac{2\,(M_{uA} + M_{uB})}{L}$

Moment capacity of end plate at section AA $\quad M_{uA} = \frac{p_u \; L_{eA} \; t_p^2}{4}$

p_u = design tensile strength = $328 N/mm^2$ $\quad (U_s / 1.25)$

t_p = plate thickness = 10mm

L_{eA} = eff. length of plastic hinge at section AA

$= 2e_{eA} + (n - 1)p_{eA}$

$e_{eA} = e_1$ but $\leq e_2$ = 40mm

$p_{eA} = p$ but $\leq 2e_2$ = 70mm

$\therefore \; L_{eA} = (2 \times 40) + (6 - 1)70$ = 430mm

$\therefore \; M_{uA} = \frac{328 \times 430 \times 10^2}{4 \times 10^3}$ = 3526kNmm

Moment capacity of end plate at section BB $\quad M_{uB} = \frac{p_u \; L_{eB} \; t_p^2}{4}$

L_{eB} = eff. length of plastic hinge at section BB

$= 2e_{eB} + (n - 1)p_{eB}$

$e_{eB} = e_1$ but $\leq c_1 + D_h/2$ = 40mm

D_h = 35mm (Hole diameter for Hollo-Bolt) $\qquad$ D_h from Yellow Pages Table H.61

$p_{eB} = p$ but $\leq 2c_1 + D_h$ = 70mm

$\therefore \; L_{eB} = (2 \times 40) + (6 - 1)70$ = 430mm

$\therefore \; M_{uB} = \frac{328 \times 430 \times 10^2}{4 \times 10^3}$ = 3526kNmm

L = distance between plastic hinges = $c_1 - (D_h/2)$

$= 49 - (35/2)$ = 31.5mm

$\frac{2(M_{uA} + M_{uB})}{L} = \frac{2(3526 + 3526)}{31.5}$ = 448kN

Tie force = 275kN < 448kN $\qquad$ **∴ O.K.**

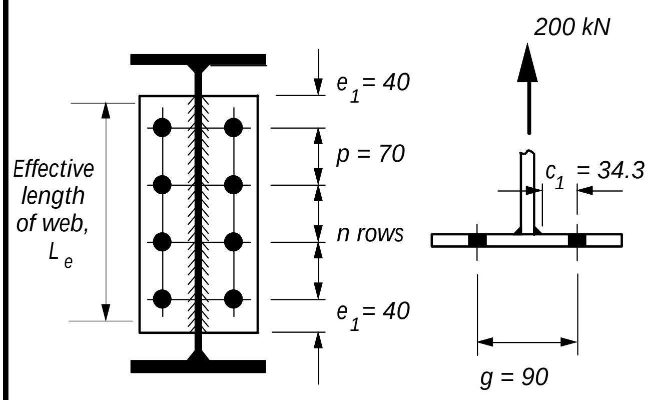
Title	Example 4 - Flexible End Plates - Beam to RHS column using Hollo-Bolt	Sheet	11 of 15

CHECK 12 : Structural Integrity - Supported Beam

(i) Tension capacity of Beam web

Basic requirement: **Tie force** $\leq$ **Tension capacity of beam web**

For 406 x 178 x 74 Grade S275



s = leg length of weld = 6mm

t_w = web thickness = 9.5mm

D_h = 35mm

$c_1 = \frac{1}{2}(g - t_w - 2s)$

$= \frac{1}{2}(90 - 9.5 - 2 \times 6)$

$= 34.3$mm

Tension capacity of beam web $= L_e \; t_w \; p_y$

Effective length, $L_e = 2e_e + (n-1)\,p_e$

$e_e = e_1$ but $\leq c_1 + D_h/2$

$= 40$mm (e_1)

$c_1 + D_h/2 = 34.3 + 35/2 = 51.8$mm

$p_e = p$ but $\leq 2c_1 + D_h$

$= 70$mm

$2c_1 + D_h = 2 \times 34.3 + 35 = 103.5$mm

$L_e = (2 \times 40) + (4-1)\,70$

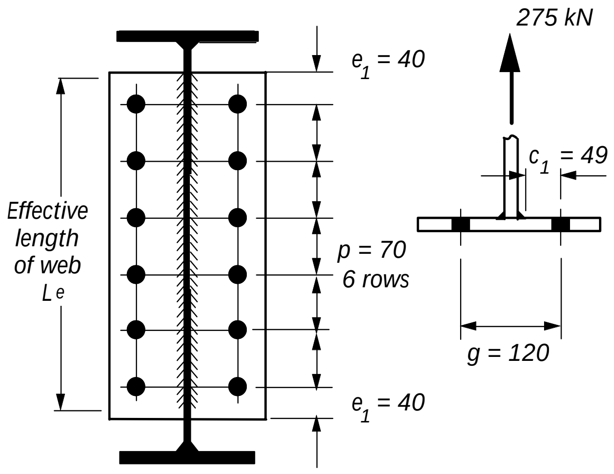
$= 290$mm

$\therefore \; L_e\, t_w\, p_y = \dfrac{290 \times 9.5 \times 275}{10^3} = 758$kN

Tie force = 200kN < 758kN **∴ O.K.**

Title	Example 4 - Flexible End Plates - Beam to RHS column using Hollo-Bolt	Sheet	12 of 15

For 533 x 210 x 92 UB Grade S275



s = leg length of weld

= 6mm

t_w = web thickness

= 10.1mm

$c_1 = \frac{1}{2}(g - t_w - 2s)$

$= \frac{1}{2}(120 - 10.1 - 2 \times 6)$

= 49mm

Tension capacity of beam web $= L_e \; t_w \; p_y$

Effective length, $L_e = 2e_e + (n-1)\,p_e$

$e_e = e_1$ but $\le c_1 + D_h/2$

$= 40\text{mm}$

$c_1 + D_h/2 = 49 + 35/2 = 66.5\text{mm}$

$p_e = p$ but $\le 2c_1 + D_h$

$= 70\text{mm}$

$2c_1 + D_h = 2 \times 49 + 35 = 133\text{mm}$

$L_e = (2 \times 40) + (6-1)\,70$

$= 430\text{mm}$

$\therefore \quad L_e\, t_w\, p_y = \frac{430 \times 10.1 \times 275}{10^3} = 1194\text{kN}$

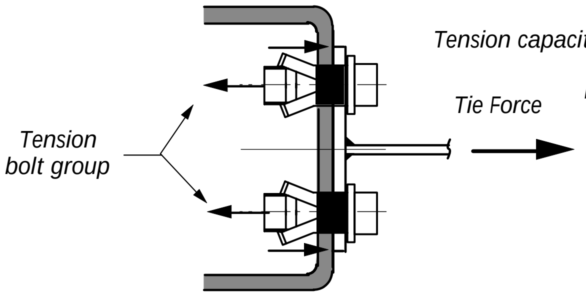
Tie force = 275kN < 1194kN **∴ O.K.**

Title	Example 4 - Flexible End Plates - Beam to RHS column using Hollo-Bolt	Sheet	13 of 15

CHECK 13: Structural Integrity - Tension Capacity of Bolts and Welds

(i) Tension Capacity of Bolts

Basic requirement: **Tie Force** $\leq$ **Tension capacity of tension bolt group**



Tension capacity = $2\ n\ P_{si}$ — *Yellow Pages Table H.56*

P_{si} = Hollo-Bolt Structural integrity Tensile Capacity

= 73kN

For 406 x 178 x 74 UB Side

Tension capacity = 2 x 4 x 73 = 584kN

Tie force = 200kN < 584kN — **∴ O.K.**

For 533 x 210 x 92 UB Side

Tension capacity = 2 x 6 x 73 = 876kN

Tie force = 275kN < 876kN — **∴ O.K.**

(ii) Weld Tension Capacity

Basic requirement: **Tie force < Tie capacity of weld**

Tension capacity of weld = $2\ (1.25\ p_w\ a\ (L_e - 2s))$

For 406 x 178 x 74 UB Side

L_e from Check 12 Sheet 11

L_e = 290mm

p_w = 220N/mm2 — *p_w from BS 5950-1 Table 37*

a = 0.7 s = 0.7 x 6 = 4.2mm

∴ Tension capacity of weld = $\dfrac{2 \times (1.25 \times 220 \times 4.2 \times (290 - 2 \times 6))}{10^3}$

= 642kN

Tie force = 200kN < 642kN — **∴ O.K.**

For 533 x 210 x 92 UB Side

L_e from Check 12 Sheet 11

L_e = 430mm

∴ Tension capacity of weld = $\dfrac{2 \times (1.25 \times 220 \times 4.2 \times (430 - 2 \times 6))}{10^3}$

= 966kN

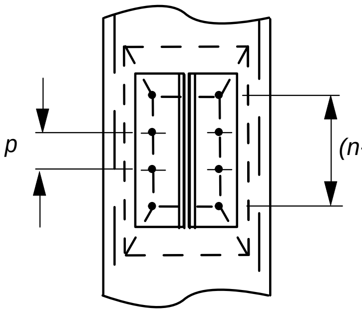
Tie force = 275kN < 966kN — **∴ O.K.**

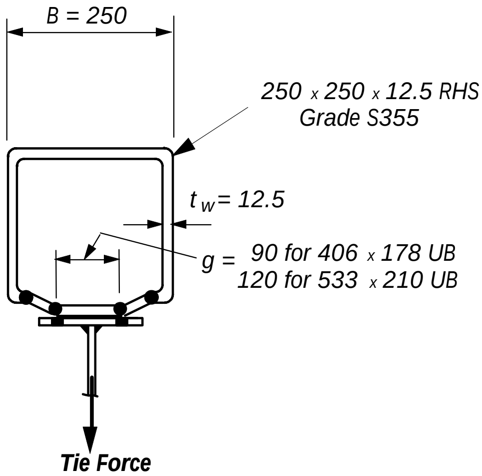
CHECK 14: not applicable

Title	Example 4 - Flexible End Plates - Beam to RHS column using Hollo-Bolt	Sheet	14 of 15

CHECK 15: Structural Integrity – Capacity of Supporting Column Wall (RHS)

Basic requirement: Tie Force $\leq$ Tying capacity of RHS column wall





For 406 x 178 x 74 UB Side

Tying capacity of column wall $= \dfrac{8\,M_u}{1-\beta_1}\left[\eta_1 + 1.5(1-\beta_1)^{0.5}(1-\gamma_1)^{0.5}\right]$

M_u = moment capacity of RHS column wall per unit length

$= \dfrac{p_u\,t_w^2}{4}$

$= \dfrac{392 \times 12.5^2}{4 \times 10^3} \quad = \quad 15.3\text{kNm/mm}$

D_h = 35mm (Hole diameter for Hollo-Bolt)

$\eta_1 = \dfrac{(n-1)\;p - \frac{n}{2}\;D_h}{B - 3t_w}$

$= \dfrac{(4-1)\;70 - \frac{4}{2} \times 35}{250 - 3 \times 12.5} \quad = \quad 0.659$

$\beta_1 = \dfrac{g}{B - 3t_w}$

$= \dfrac{90}{212.5} \quad = \quad 0.424$

$\gamma_1 = \dfrac{D_h}{B - 3t_w}$

$= \dfrac{35}{212.5} \quad = \quad 0.165$

Tying capacity of column wall $= \dfrac{8 \times 15.3}{1 - 0.424}\left[0.659 + 1.5(1-0.424)^{0.5} \times (1-0.165)^{0.5}\right]$

$= 212.5\left[0.659 + 1.5 \times 0.759 \times 0.914\right]$

$= 361\text{kN}$

Tie force = 200kN < 361kN **∴ O.K.**

Title	Example 4 - Flexible End Plates - Beam to RHS column using Hollo-Bolt	Sheet	15 of 15

For 533 x 210 x 92 UB Side

$$\eta_1 = \frac{(n-1)\ p - \frac{n}{2}\ D_h}{B - 3t_w}$$

$$= \frac{(6-1)\ 70 - \frac{6}{2} \times 35}{250 - 3 \times 12.5} = 1.15$$

$$\beta_1 = \frac{120}{212.5} = 0.565$$

Tying capacity of column wall $= \dfrac{8\ M_u}{1 - \beta_1} \left[\eta_1 + 1.5(1-\beta_1)^{0.5}\ (1-\gamma_1)^{0.5} \right]$

$$= \frac{8 \times 15.3}{1 - 0.565} \left[1.15 + 1.5(1-0.565)^{0.5} \times (1-0.165)^{0.5} \right]$$

$$= 281.4 \left[1.15 + 1.5 \times 0.660 \times 0.914 \right] = 578kN$$

Tie force = 275kN < 578kN **∴ O.K.**

6. FIN PLATES

6.1 INTRODUCTION

The fin plate connection consists of a length of plate welded in the workshop to the supporting member (Beam, **I** Column, RHS Column or CHS Column) and then bolted to the supported beam web on site (See Figure 6.1).

It is popular as it can be one of the quickest connections to fix and overcomes the problem of shared bolts in two-sided connections.

Rotation

Fin plate connections derive their rotational capacity from:

(i) hole distortions in the fin plate and/or the beam web;

(ii) out of plane bending of the fin plate;

(iii) shear deformation of the bolts.

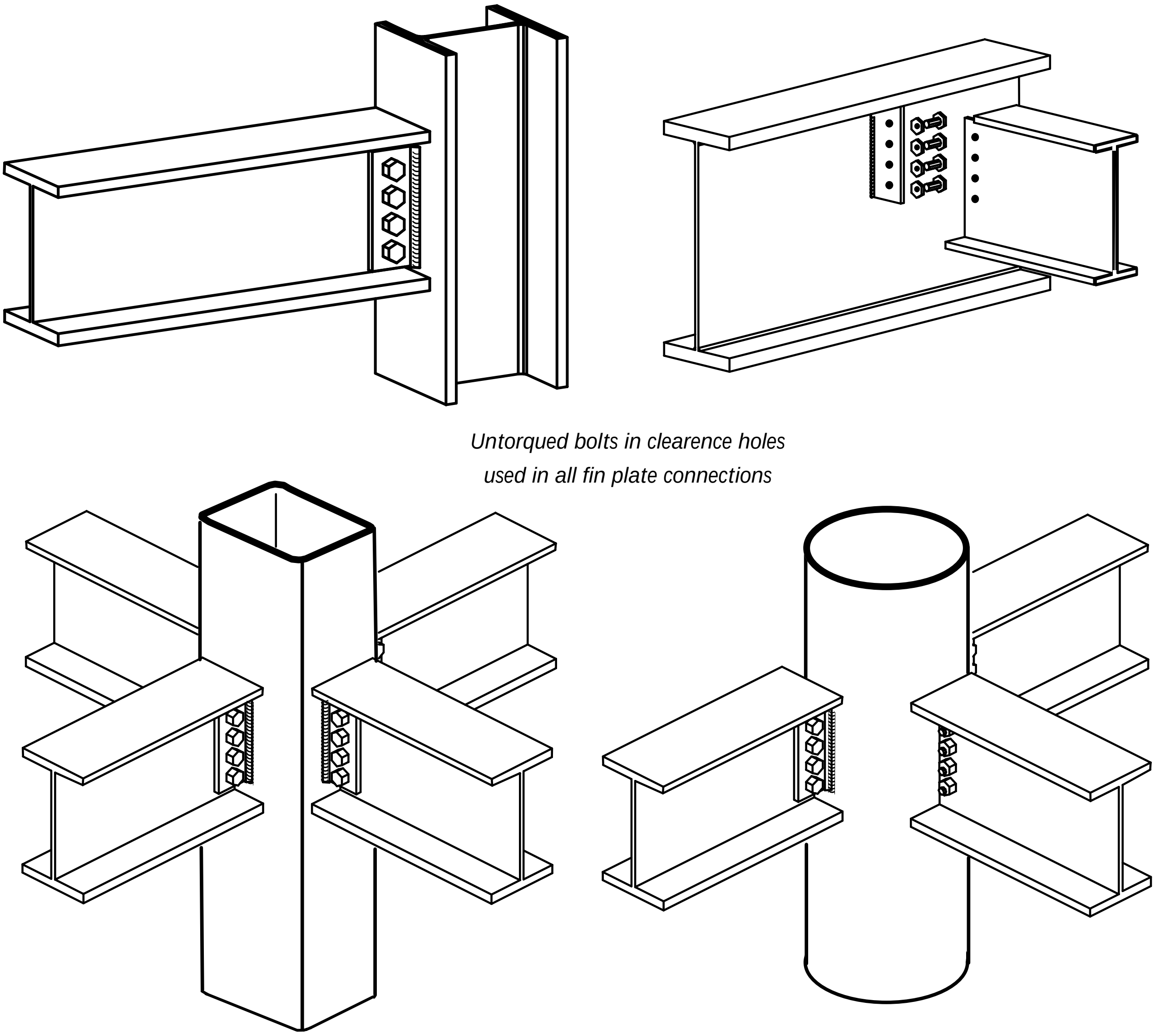


Figure 6.1 Fin plate. Beam-to-column and beam-to-beam connections

Additional deformation could be available from bolt slippage but this may be limited as most bolts will be in bearing from the outset. To ensure that such connections have sufficient rotation capacity, a series of tests [16][25] was undertaken to verify the proposed design model, and thus to establish the design procedure. However, for beams deeper than 610mm, caution must be exercised. In these cases further tests[25] have shown that additional geometrical precautions are needed, as described later in this Section.

The detailing rules which follow will ensure that unsatisfactory failure modes will not occur and that rotation capacity is achieved.

6.2 PRACTICAL CONSIDERATIONS

The fin plate is usually made by cropping and punching. Using standard flats - 100 x 10mm or 150 x 10mm flats should prove adequate for all short fin plates. With 10mm thick plates in S275 steel, 8mm fillet welds to the support will guard against any possibility of weld failure. Care should be taken to avoid undercut during welding.

Skewed and raking beams as well as moderate offsets can be easily accommodated.

Generally the connection is arranged with the supported beam web lying on the centreline of the supporting member and the fin plate offset as shown in Figure 6.2. On the site it is not always evident to which side of the fin plate the beam is to be bolted and so a consistent system for setting out should be established during detailing. Alternatively, the contact face of the fin plate should be marked.

Rather like the flexible end plate, there is little facility for site adjustment, and care must be taken with continuous runs of beams.

Short and long fin plates

Fin plates may be classified as short or long as follows:

$$\text{Short,} \quad \frac{t_f}{a} \geq 0.15 \qquad \text{Long,} \quad \frac{t_f}{a} < 0.15$$

Where 'a' is the distance between the first line of bolts and the face of the support. (See Figure 6.2)

One of the drawbacks with the short fin plate is access, particularly with beams framing into narrow column webs. Once a pair of adjacent columns have been pulled together with the first interconnecting beam, subsequent beams must have at least one bottom flange trimmed to allow them to be lowered into position as shown in Figure 6.3. Access for the podger spanner can also be difficult.

To overcome this problem, the plate can be increased in length to clear the supporting column or beams. This has the benefit of avoiding the need for notching or trimming the supported beam. In these cases further design checks must be carried out to avoid lateral torsional buckling i.e. out-of-plane buckling of the fin plate.

With laterally unrestrained beams, long fin plates can behave in an extremely complex way. **Without experimental evidence to justify their design, they should not be used.**

Erection

The greatest benefit from this connection undoubtedly comes on site. The beam can be quickly swung into position and pulled into line with a podger spanner. The insertion of approximately one third of the total number of bolts in the connection at each end is usually sufficient to allow the crane hook to be released to make the next lift.

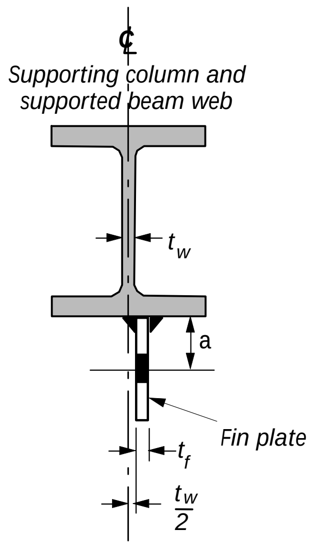


Figure 6.2 Setting out for fin plate

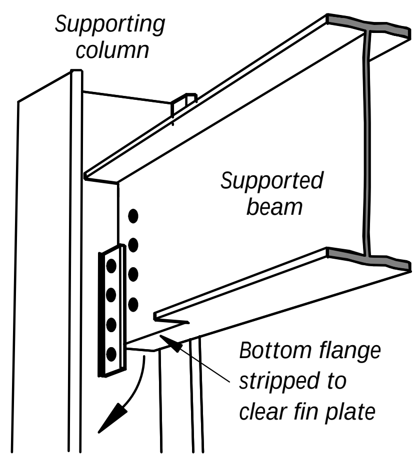
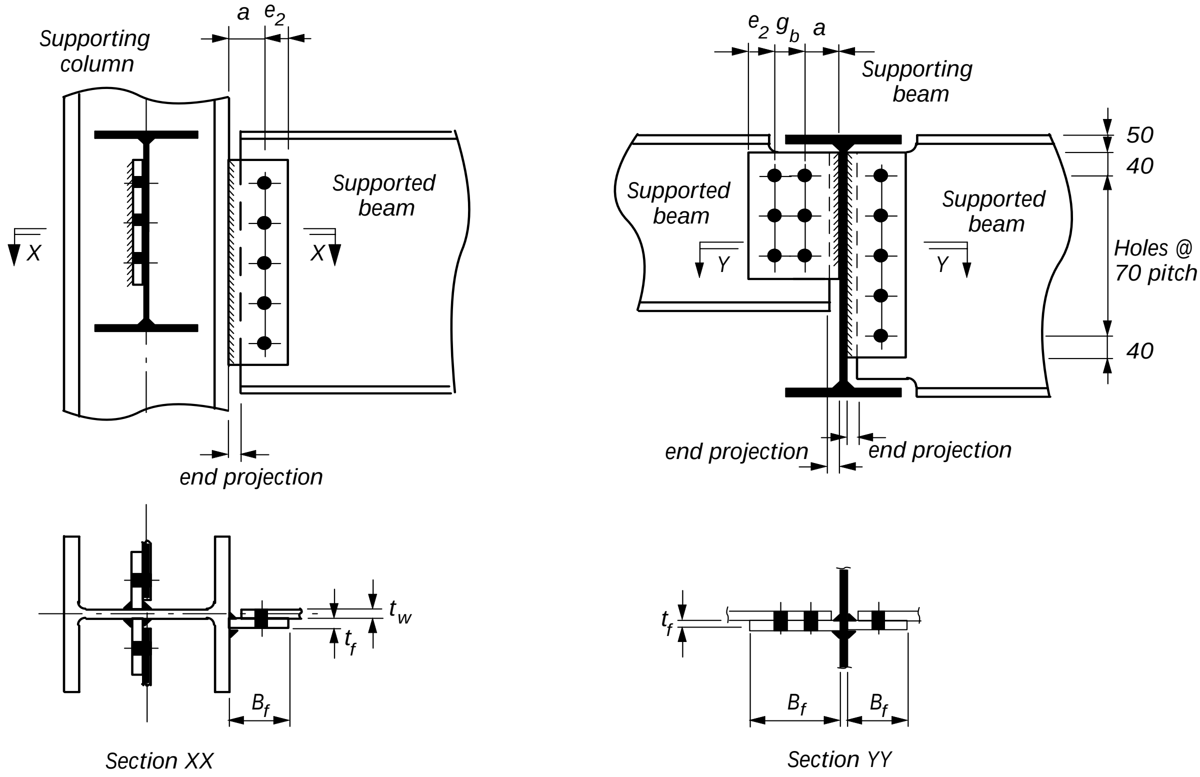


Figure 6.3 Detail to facilitate erection



Beam-to-column **Beam-to-beam**

Supported beam depth mm	Vertical bolt lines	Recommended fin plate size ($B_f \times t_f$) mm	Horizontal bolt spacing (a/e_2) ($a/g_b/e_2$) mm	End projection mm
≤ 610	1	100 x 10	50/50	10
> 610 *	1	120 x 10	60/60	20
≤ 610	2	150 x 10	50/50/50	10
>610 *	2	180 x 10	60/60/60	20

Bolts: M20 8.8 in 22mm diameter holes

Plate: S275 steel, minimum length 0.6D where D is depth of supported beam

Weld: Two 8mm fillets for 10mm thick plates

* For beams over 610mm deep the span to depth ratio of beam should not exceed 20 and the vertical distance between extreme bolts should not exceed 530mm.

Figure 6.4 Standard fin plate connection

6.3 RECOMMENDED GEOMETRY

The design procedures which follow set down a number of recommended details which are intended to achieve the required flexibility for a simple connection.

When detailing the joint, the main requirements are as follows:-

(i) the fin plate is positioned close to the top flange in order to provide positional restraint;

(ii) the fin plate depth is at least 0.6 times the supported beam depth in order to provide the beam with adequate torsional restraint;

(iii) the thickness of the fin plate or the beam web is
≤ 0.42d (for S355 steel) or
≤ 0.50d (for S275 steel)
(d = nominal bolt diameter);

(iv) 8.8 bolts are used, un-torqued and in clearance holes;

(v) all end and edge distances on the plate and the beam web are at least 2d;

(vi) the fillet weld leg length is at least 0.8 times the fin plate thickness.

The first two requirements ensure that in those cases where the beam is laterally unrestrained, it can be designed with an effective length of 1.0L. (BS 5950: Table 13)

The last four requirements ensure a bearing mode of failure in either the fin plate or beam web. This provides the necessary rotational flexibility in the connection.

These requirements, together with the standard geometry presented in Section 2, have been used to create the 'standard connection' shown in Figure 6.4.

6.4 DESIGN

The full design procedure is presented in Section 6.5.

With a single vertical line of bolts, the connection shear capacity will be in the range 25% to 50% of the beam shear capacity - a value similar to that of the double angle web cleat connection. Two vertical lines of bolts do help, but as the eccentricity of the design load also increases, the benefit does not double and the best that can be achieved is around 75% of the beam shear capacity.

For connections using the standard geometry, the capacity will generally be governed by bolt bearing (Check 2) or by shear and bending of the fin plate (Check 3).

An alternative design procedure has been developed in Australia and this is reported in references 26 and 27.

Flexible and rigid supports

For the design of the fin plate and bolts the model presented in Section 6.5 assumes a line of action for the load at the point where the fin plate is welded to the support, i.e. the support is flexible and free to rotate. (Figure 6.5 (a)). As most supports have some rigidity, this will be found to be a slightly conservative approach.

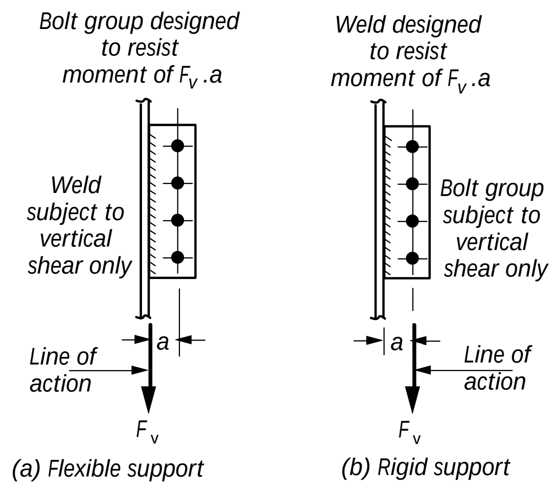


Figure 6.5 Flexible and rigid supports

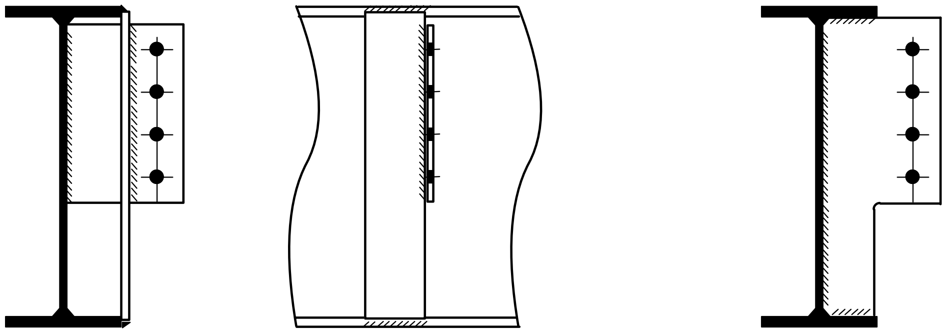
If the fin plate support can be assumed to be rigid, for example a heavy column flange, then there is no reason why the designer should not modify the model so that the line of action is through the bolts (Figure 6.5 (b)).

Clearly the design model must be consistent, and if the above approach is adopted, the fin plate and weld should be designed for combined shear and bending moment. The column should be checked for axial load and bending moment based on the longer of the nominal eccentricity (100mm) or 'a'.

Stiffening

It is possible to improve the performance of a long fin plate by providing some stiffening. This may either stabilise the plate against out of plane buckling or may improve the support provided to the plate by the supporting beam or column web. The same stiffening may achieve both effects, see Figures 6.6 and 6.7.

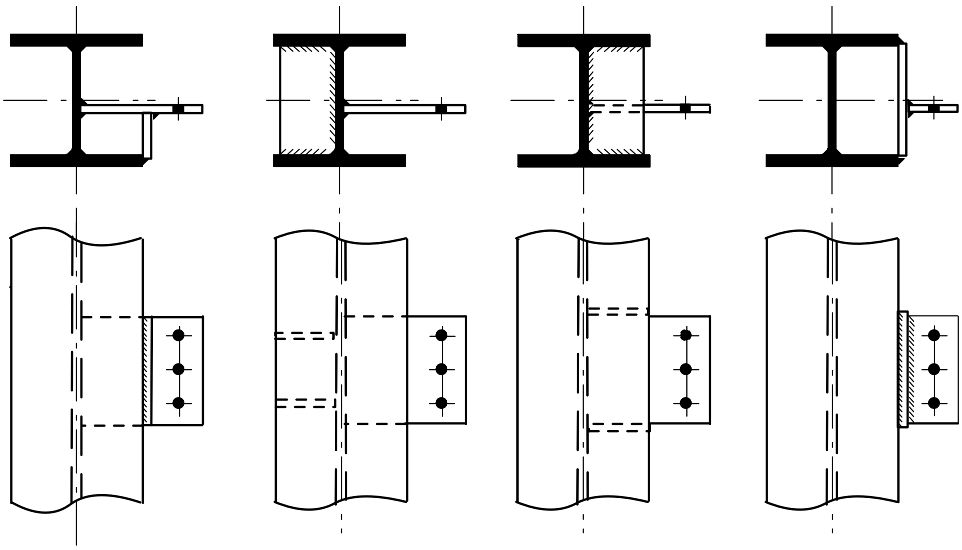
One further minor consideration is the torsion introduced because the fin plate is attached to only one side of the supported beam web. The test programme indicated that these torsions are negligible and may be ignored.



(a) Stability against out of plane buckling

(b) Resistance to web distortion

Figure 6.6 Possible stiffening arrangement for long fin plates to supporting beam



(a) Stability

(b) In-plane stiffness

(c) Stability & stiffness

(d) Substitute short fin plate

Figure 6.7 Stiffening arrangements for long fin plates to column webs

Structural integrity

In certain tall multi-storey buildings there is a requirement for the connections to be designed to carry large tying forces. This is intended to prevent disproportionate collapse.

Fin plate connections detailed in accordance with Figure 6.4 will be found to have tying capacities which generally exceed their shear capacity, and in this respect no further checks will normally be required to the plate, weld or beam. Local checks to the column, particularly if there are any one-sided beam to column connections, must be carried out. (See design procedure Checks 14 to 16.)

Worked examples

Four worked examples are provided in Section 6.6 to illustrate the full set of design checks of Section 6.5.

Connection capacity tables

Two sets of capacity tables for fin plate connections with both one and two vertical rows of bolts, using steel grade S275 and S355 beams, detailed in accordance with the standard geometry presented in Figure 6.4, are included in the yellow pages.

Values of the connection shear and tying capacities are tabulated together with simple aids to check the capacity of the supporting member and the beam notch (If applicable).

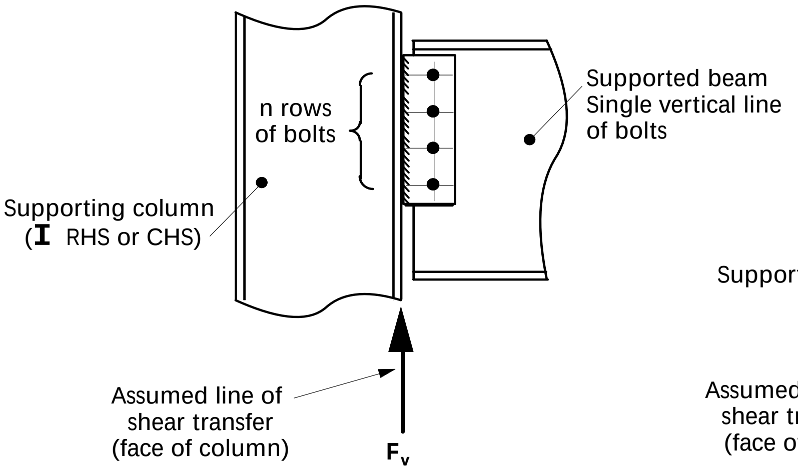
6.5 DESIGN PROCEDURE

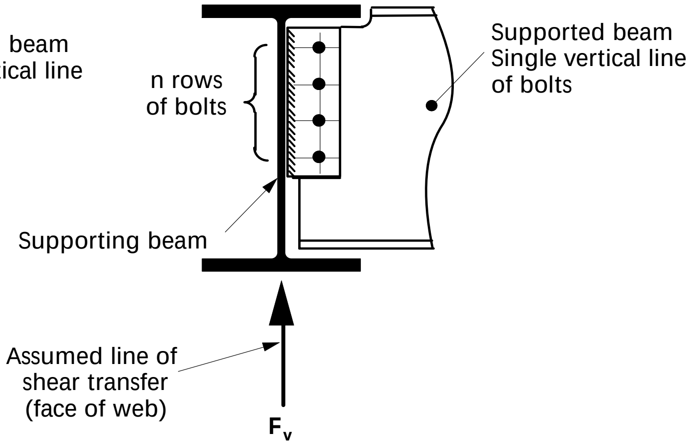
Recommended design model

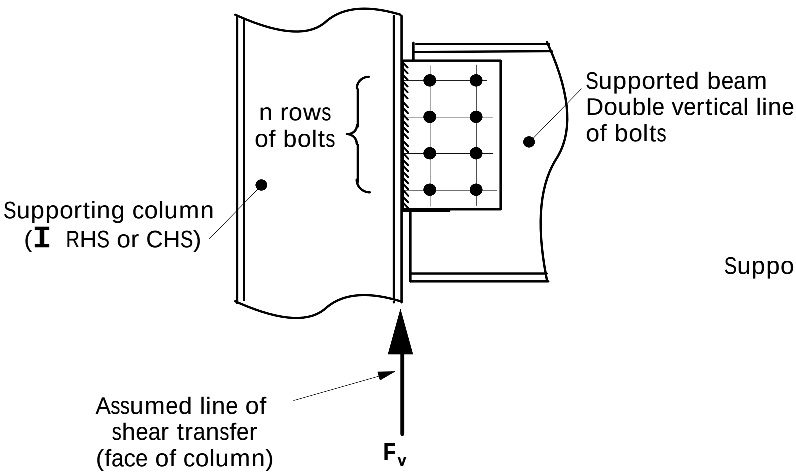
The design procedure applies to beams connected either to the column flange, the column web, the supporting beam web, RHS columns or to CHS columns. Although the figures show partial depth fin plates the design procedures apply equally to full depth fin plates.

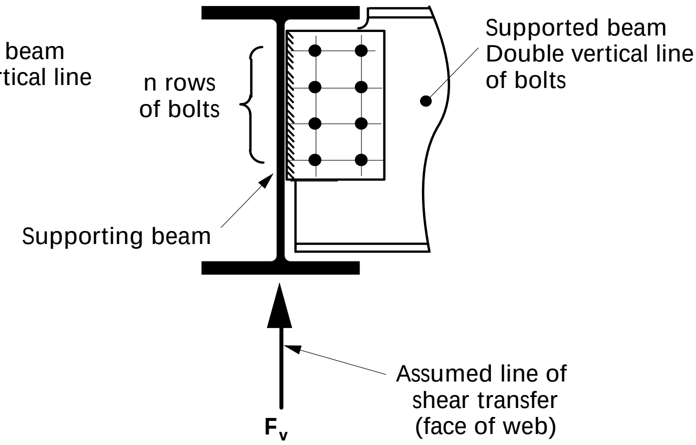
For connections to the column web the d/t ratio of the column should be limited to 40ε for **I** and hot finished RHS and 35ε for cold formed RHS. D/t should be limited to $50\varepsilon^2$ for CHS.

$$\varepsilon = \left(\frac{275}{p_y}\right)^{0.5}$$



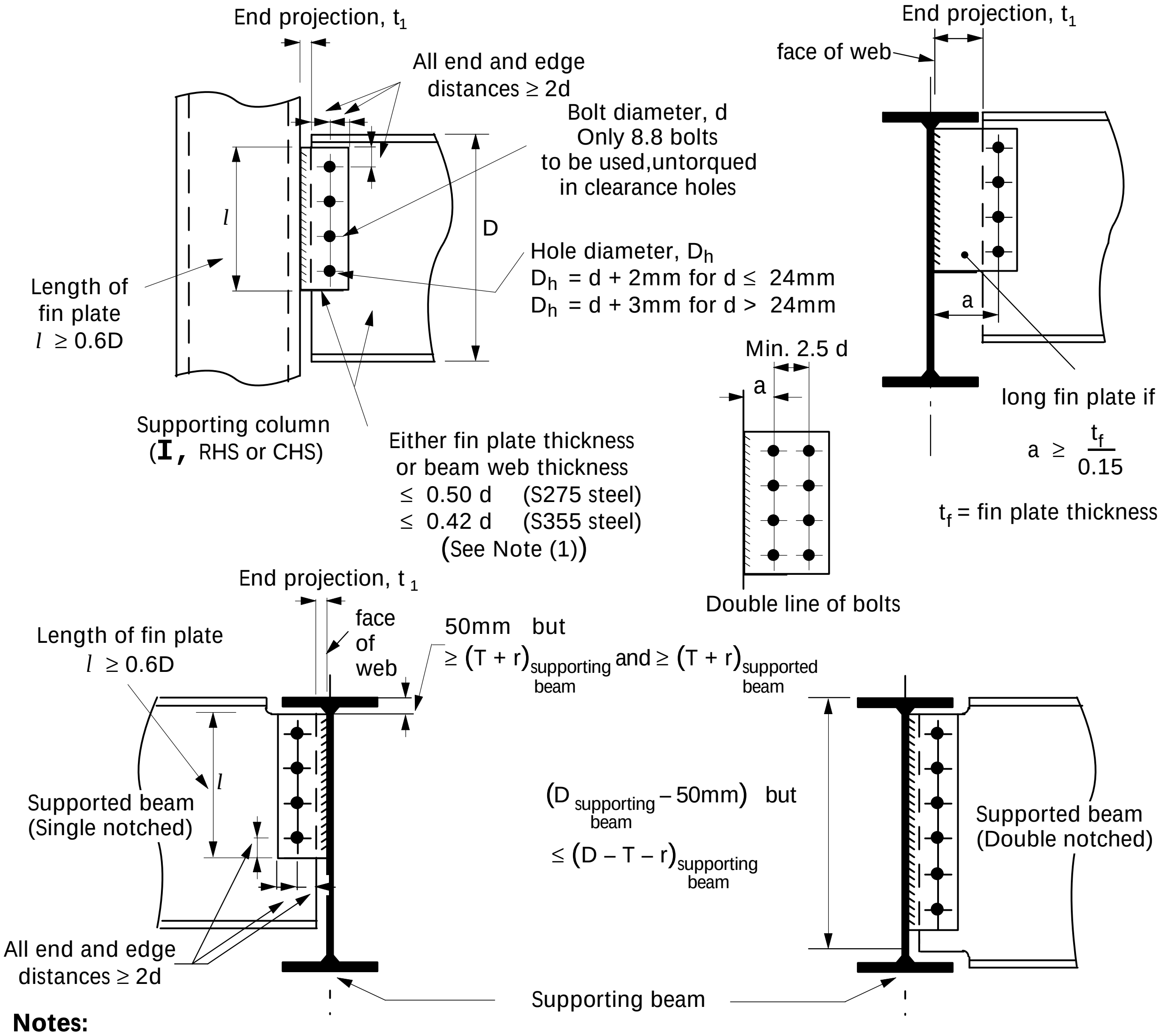






Check 1	**Recommended detailing practice**		
Check 2	**Supported beam**	–	Bolt group
Check 3	**Supported beam**	–	Connecting elements
Check 4	**Supported beam**	–	Capacity at the connection
Check 5	**Supported beam**	–	Capacity at a notch
Check 6	**Supported beam**	–	Local stability of notched beam
Check 7	**Unrestrained supported beam**	–	Overall stability of notched beam
Check 8	**Supporting beam/column**	–	Welds
Check 9	*Not applicable*		
Check 10	**Supporting beam/column**	–	Local capacity
Check 11	**Structural integrity**	–	Connecting elements
Check 12	**Structural integrity**	–	Supported beam
Check 13	*Not applicable*		
Check 14	**Structural integrity**	–	Supporting column web (UC or UB)
Check 15	**Structural integrity**	–	Supporting column wall (RHS)
Check 16	**Structural integrity**	–	Supporting column wall (CHS)

CHECK 1	Recommended detailing practice



Notes:

(1) For the design methods in this guide to be satisfactory, it is essential to comply with the limitation on fin plate or beam web maximum thicknesses to ensure adequate deformation capacity.

(2) The fin plate is generally positioned close to the top flange of the beam to provide adequate positional restraint. Its length should be at least 0.6D to give adequate "nominal torsional restraint" (BS 5950-1 Table 13 and cl. 4.2.2).

(3) For supported beams exceeding a depth of 610mm [25], the design method given here may only be used when the following three conditions are all met:

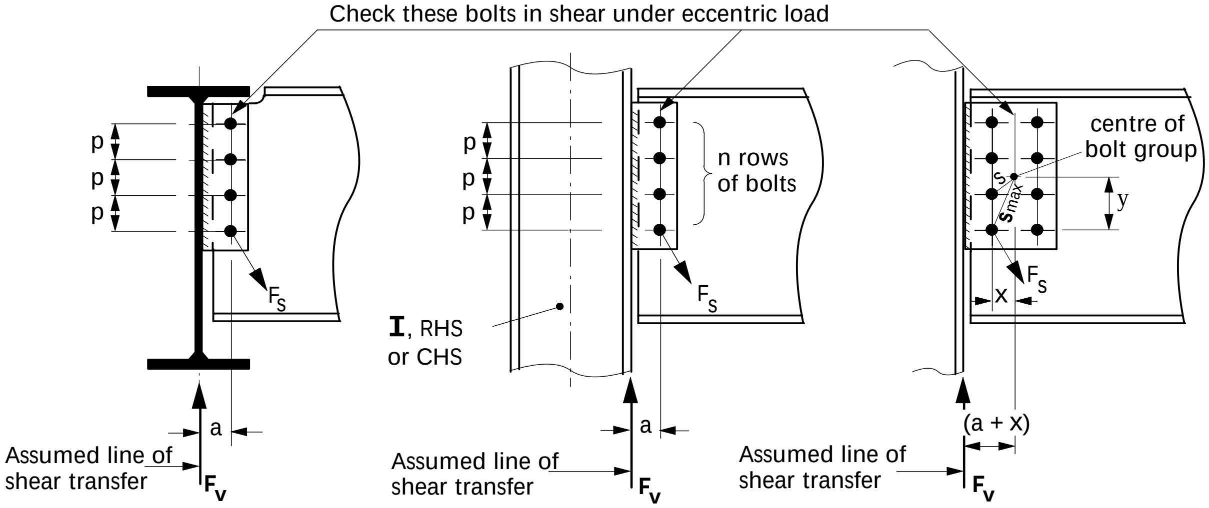
- Supported beam Span/Depth ≤ 20
- End projection t_1 ≥ 20mm
- Vertical distance between extreme bolts (n-1)p ≤ 530mm

(4) Bolt spacing and edge distances should comply with the recommendations of BS 5950-1:2000.

(5) Detailing is similar for long fin plates (i.e. the fin plate thickness t_f is less than 0.15a) except the end projection t_1 will be considerably greater.

> In beam-to-column (flanges of **I**-section) connections where it is required to comply with structural integrity requirements for a tie force of 75kN, the connection must have at least 2 no. M20, 8.8 bolts and the fin plate thickness ≥ 6mm.
>
> For greater tie forces, checks 11 to 16, as appropriate, should be carried out.

CHECK 2	Supported beam – Bolt group



Single line of bolts **Double line of bolts**

Capacity of bolt group connecting fin plates to web of supported beam (taking account of eccentricity 'a' for single line of bolts and (a + x) for double line of bolts.)

Basic requirement (bearing of bolts on fin plate and beam web)*:

$F_s \leq P_{bs}$

F_s = resultant force on outermost bolt due to direct shear and moment

For single line of bolts

$F_s = \left(F_{sv}^2 + F_{sm}^2\right)^{1/2}$

F_{sv} = vertical force on the bolt due to direct shear $= \dfrac{F_v}{n}$

F_{sm} = force on the outermost bolt due to moment $= \dfrac{F_v\, a}{Z_{bg}}$

Z_{bg} = elastic section modulus of bolt group $= \dfrac{n(n+1)p}{6}$

For Double line of bolts

$F_s = \left((F_{sv} + F_{smv})^2 + F_{smh}^2\right)^{1/2}$

F_{sv} = vertical force on the bolt due to direct shear $= \dfrac{F_v}{2n}$

F_{smv} = vertical force on the outermost bolt due to moment $= \dfrac{M\,x}{I_{bg}}$

F_{smh} = horizontal force on the outermost bolt due to moment $= \dfrac{M\,y}{I_{bg}}$

$M = F_v\,(a + x)$

$I_{bg} = \sum s^2$

where:

p = bolt pitch

s = distance from centre of bolt group to each bolt

s_{max} = distance from centre of bolt group to furthest bolt

P_{bs} = bearing capacity per bolt

= min ($d\, t_f\, p_{bs.f}$, $d\, t_w\, p_{bs.b}$)

$p_{bs.f}$ = bearing strength of fin plate

$p_{bs.b}$ = bearing strength of supported beam web

t_f = thickness of fin plate

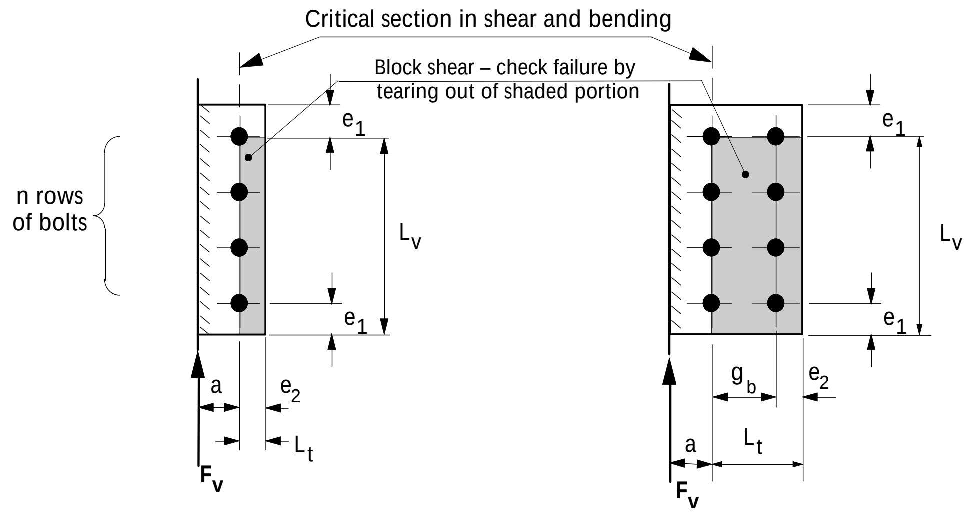
t_w = thickness of supported beam web

d = diameter of bolt

I_{bg} = inertia of bolt group

* Providing that the limitations on fin plate thickness, beam web thickness and end distance in **CHECK 1** are met, bolt capacity will be governed by normal bearing on the fin plate or beam web and not bolt shear or end bearing.

CHECK 3	Supported Beam – Connecting elements



Single Line of bolts **Double Line of bolts**

Shear and bending capacity of fin plate connected to supported beam

(i) For shear:

Basic Requirement:

$F_v \leq P_{v.min}$

$P_{v.min}$ = shear capacity of fin plate

= smaller of plain shear capacity P_v and block shear capacity P_r

Plain shear

$P_v = \min\left(0.6\, p_y A_v\,,\quad 0.7\, p_y K_e A_{v.net}\right)$

$A_v = 0.9\,(2e_1 + (n-1)\,p)\,t_f$

$A_{v.net} = A_v - n\,D_h\,t_f$

Block shear

$P_r = 0.6\,p_y\,t_f\left(L_v + K_e(L_t - kD_h)\right)$

$L_v = e_1 + (n-1)p$

$k = 0.5$ and $L_t = e_2$ (for single line of bolts)

$k = 2.5$ and $L_t = e_2 + g_b$ (for double line of bolts)

(ii) Shear and bending interaction

Basic Requirement:

$F_v\,a \leq M_c$

For low shear (i.e. $F_v \leq 0.75P_{v.min}$)

$$M_c = \frac{p_y\,t_f}{6}\left(2e_1+(n-1)p\right)^2$$

For high shear (i.e. $F_v > 0.75P_{v.min}$)

$$M_c = \frac{p_y\,t_f}{4}\left(2e_1+(n-1)p\right)^2\left(1-\left(\frac{F_v}{P_{v.min}}\right)^2\right)^{1/2}$$

where:

p = bolt pitch

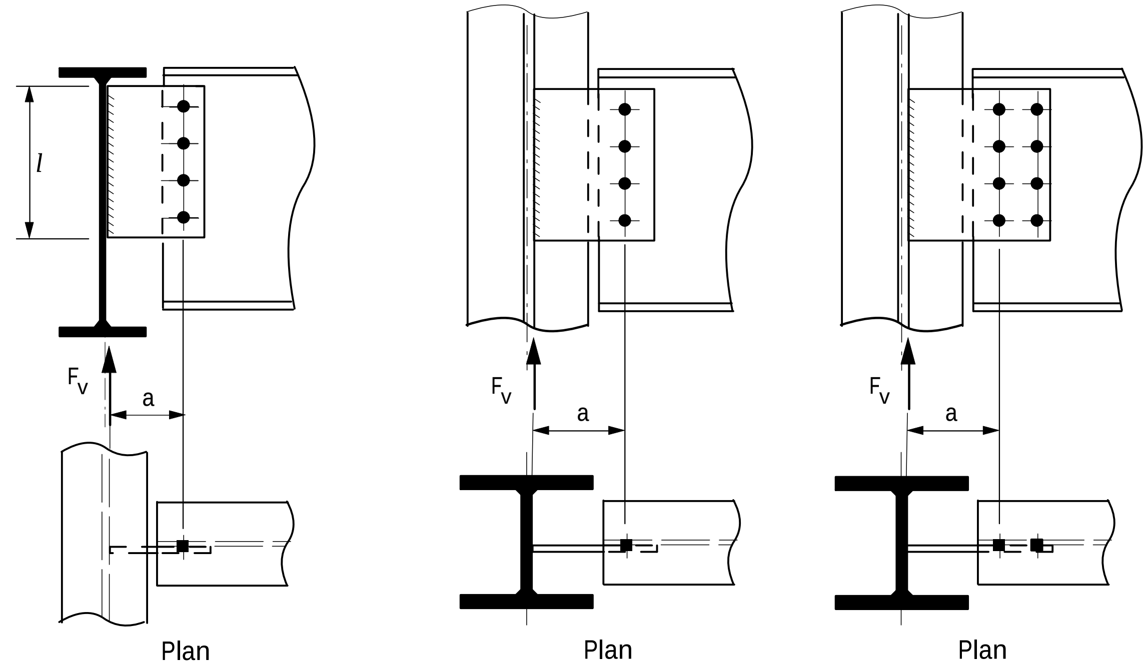
D_h = hole diameter

t_f = fin plate thickness

K_e = 1.2 for S 275 steel

= 1.1 for S 355 steel

CHECK 3 continued	Supported beam – Connecting elements



(iii) Lateral torsional buckling resistance of <u>long</u> fin plates (i.e. where $a > \frac{t_f}{0.15}$)

Basic Requirement:

$F_v\, a \le M_b/m_{LT}$ and $F_v\, a \le p_y Z_x$

M_b = lateral torsional buckling resistance moment of the fin plate

= $p_b Z_x$

Z_x = elastic modulus of fin plate about major axis

= $\frac{t_f\, l^2}{6}$

p_b = bending strength of fin plate obtained from BS 5950-1, Table 17 and based on λ_{LT} as follows:

$$\lambda_{LT} = 2.8 \left(\frac{L_E\, l}{1.5\, t_f^2} \right)^{1/2}$$

(BS5950-1, cl.B.2.7)

where:

l = depth of fin plate

t_f = thickness of fin plate

L_E = 1.0 a

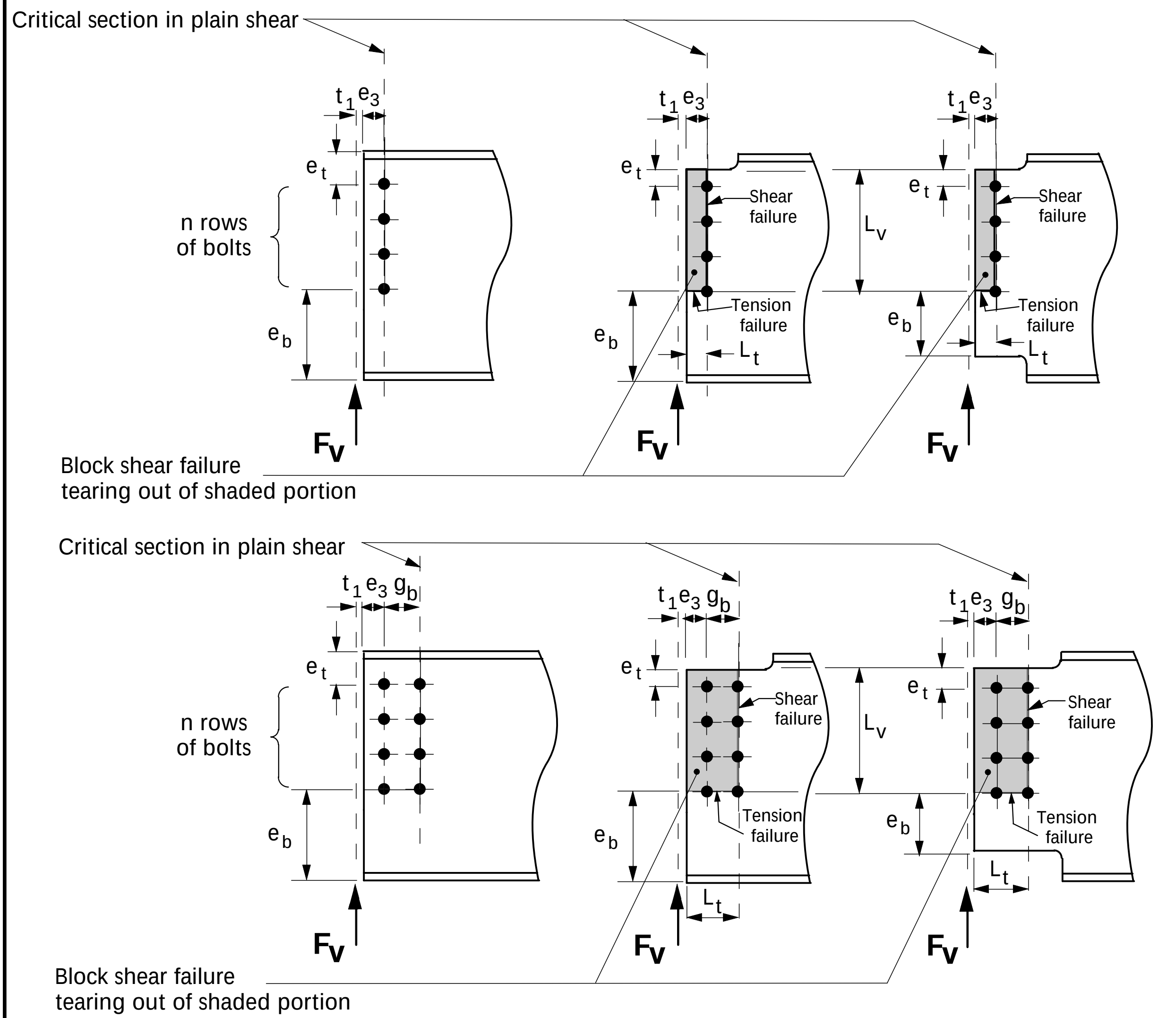
m_{LT} = 0.6 (BS5950-1, Table 18)

Notes:

(1) This check is required for long fin plates (i.e. $a > \frac{t_f}{0.15}$) when the supported beam is laterally restrained at its connection to the fin plate.

(2) Long fin plates should not be used with unrestrained beams without experimental evidence to justify the design.

CHECK 4	Supported Beam – Capacity at the connection



Shear and bending capacity of the supported beam:

(i) For shear:

Basic requirement:

$F_v \leq P_{v.min}$

$P_{v.min}$ = shear capacity of the beam at the connection

= smaller of:
Plain shear capacity P_v and
Block shear capacity P_r

Plain shear:

$P_v = \min\left(0.6\, p_y A_v\, ,\ 0.7\, p_y K_e A_{v.net}\right)$

$A_v = (e_t + (n-1)\,p + e_b)t_w$
(for un-notched and single notch beam)

$= 0.9(e_t + (n-1)p + e_b)t_w$
(for double notched beam)

$A_{v.net} = A_v - n\,D_h\,t_w$

Block shear (applicable to notch beams only)

$P_r = 0.6\, p_y\, t_w\left(L_v + K_e(L_t - kD_h)\right)$

$L_v = e_t + (n-1)p$

$k = 0.5$ and $L_t = e_3$ (for single line of bolts)

$k = 2.5$ and $L_t = e_3 + g_b$ (for double line of bolts)

where:

K_e = 1.2 for S 275 steel
1.1 for S 355 steel

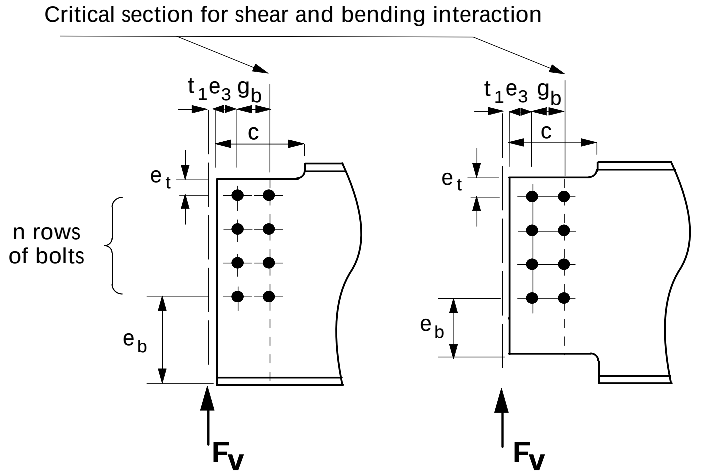
p = bolt pitch

D_h = diameter of hole

t_w = thickness of supported beam web

CHECK 4 continued	Supported beam – Capacity at the connection

Note: *If the notch length c is greater than* $(e_3 + g_b)$*, then shear and bending interaction should be checked at the 2*[nd] *line of bolts. The beam at the end of the notch may also be critical -* *see CHECK 5.*



(ii) Shear and Bending interaction at the 2nd line of bolts, if the notch length c > $(e_3 + g_b)$

Basic requirement:

$F_v (t_1 + e_3 + g_b) \leq M_{cC}$

For single notched beam:

For low shear (i.e. $F_v \leq 0.75P_{v.min}$)

$$M_{cC} = p_y Z$$

For high shear (i.e. $F_v > 0.75P_{v.min}$)

$$M_{cC} = 1.5\, p_y Z \left(1 - \left(\frac{F_v}{P_{v.min}}\right)^2\right)^{1/2}$$

For double notched beam:

For low shear (i.e. $F_v \leq 0.75P_{v.min}$)

$$M_{cC} = \frac{p_y t_w}{6}\left(e_t + (n\text{-}1)p + e_b\right)^2$$

For high shear (i.e. $F_v > 0.75P_{v.min}$)

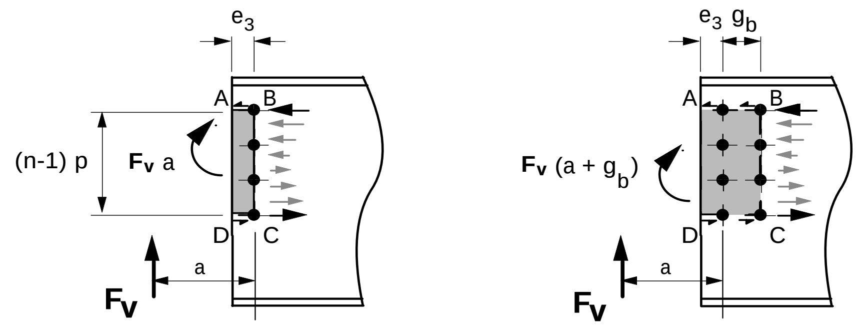
$$M_{cC} = \frac{p_y t_w}{4}\left(e_t + (n-1)p + e_b\right)^2\left(1 - \left(\frac{F_v}{P_{v.min}}\right)^2\right)^{1/2}$$

where:

M_{cC} = moment capacity of the notched beam at the connection in the presence of shear.

Z = elastic section modulus of the gross tee section at the bolt line.

CHECK 4 continued	Supported beam – Capacity at the connection



Shear and bending interaction of the beam web:

For short fin plates (i.e. $a \le \frac{t_f}{0.15}$) background studies have demonstrated that the resistance of the web does not need to be checked.

For long fin plates (i.e. $a > \frac{t_f}{0.15}$) it is necessary to ensure that the section labelled as ABCD above can resist a moment $F_v a$ for single line of bolts or $F_v(a + g_b)$ for double line of bolts. (AB and CD are in shear and BC is in bending.)

(iii) Shear and bending interaction of the beam web section - Long fin plates only (i.e. $a > \frac{t_f}{0.15}$)

Basic requirement:

$F_v a \le M_{cBC} + P_{vAB}(n-1)p$

for single line of bolts

$F_v(a + g_b) \le M_{cBC} + P_{vAB}(n-1)p$

for double line of bolts

For low shear (i.e. $F_{vBC} \le 0.75P_{vBC}$)

$$M_{cBC} = \frac{p_y t_w}{6}\left((n-1)\,p\right)^2$$

For high shear (i.e. $F_{vBC} > 0.75P_{vBC}$)

$$M_{cBC} = \frac{p_y t_w}{4}\left((n-1)p\right)^2\left(1 - \left(\frac{F_{vBC}}{P_{vBC}}\right)^2\right)^{1/2}$$

where:

M_{cBC} = moment capacity of the beam web BC

P_{vAB} = shear capacity of the beam web AB

= $\min\left(0.6\,p_y e_3 t_w,\ 0.7 p_y K_e \left(e_3 - \frac{D_h}{2}\right) t_w\right)$

for single line of bolts

= $\min\left(0.6\,p_y (e_3+g_b) t_w,\ 0.7 p_y K_e \left(e_3 + g_b - \frac{3\,D_h}{2}\right) t_w\right)$

for double line of bolts

P_{vBC} = shear capacity of the beam web BC

= $\min\left(0.6\,p_y (n-1)\,p\,t_w,\ 0.7 p_y K_e \left((n-1)(p - D_h)\right) t_w\right)$

F_{vBC} = shear force on the beam web BC

= $F_v - (P_v - P_{vBC})$ but ≥ 0

t_w = thickness of beam web

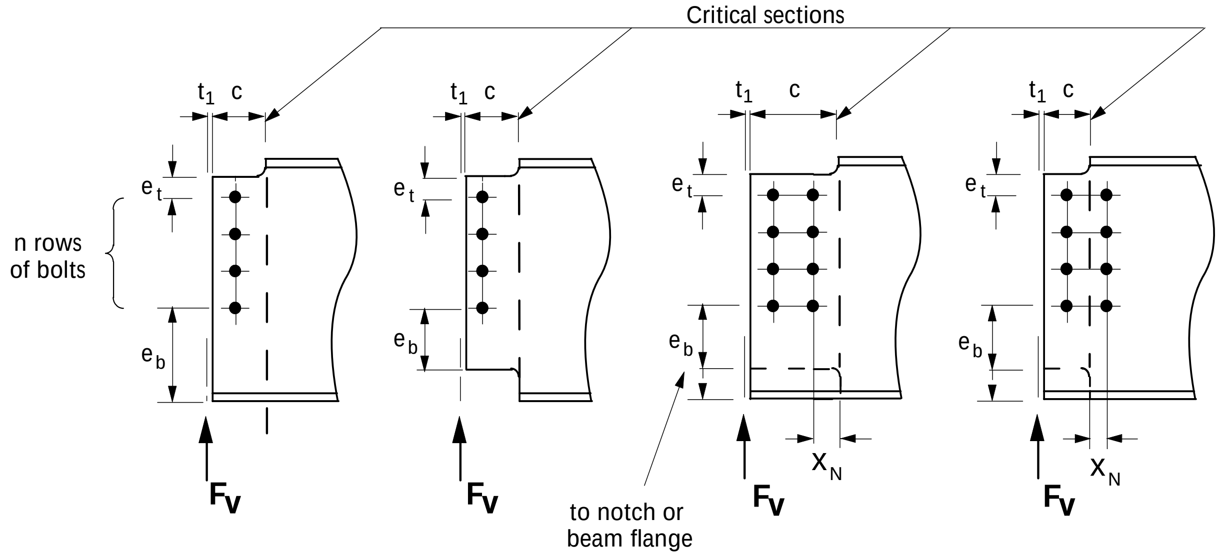
t_f = thickness of fin plate

D_h = diameter of hole

K_e = 1.2 for S275 steel

= 1.1 for S355 steel

CHECK 5	Supported beam – Capacity at a notch



Shear and bending interaction at the notch:

Basic requirement:

(a) For single bolt line or for double bolt lines, if $x_N \geq 2d$:

$$F_v\,(t_1 + c) \leq M_{cN}$$

M_{cN} for Single notched beam:

For low shear (i.e. $F_v \leq 0.75P_{vN}$)

$$M_{cN} = p_y Z_N$$

For high shear (i.e. $F_v > 0.75P_{vN}$)

$$M_{cN} = 1.5 p_y Z_N \left(1 - \left(\frac{F_v}{P_{vN}}\right)^2\right)^{1/2}$$

M_{cN} for Double notched beam:

For low shear (i.e. $F_v \leq 0.75P_{vN}$)

$$M_{cN} = \frac{p_y t_w}{6}\big(e_t + (n-1)p + e_b\big)^2$$

For high shear (i.e. $F_v > 0.75P_{vN}$)

$$M_{cN} = \frac{p_y t_w}{4}\big(e_t + (n-1)p + e_b\big)^2\left(1 - \left(\frac{F_v}{P_{vN}}\right)^2\right)^{1/2}$$

(b) For Double bolt lines, if $x_N < 2d$:

$$\max\big(F_v(t_1+c),\ F_v(t_1+e_3+g_b)\big) \leq M_{cN}$$

$M_{cN} = M_{cC}$ from Check 4

where:

M_{cN} = moment capacity of the beam at the notch in the presence of shear

P_{vN} = shear capacity at the notch
= $0.6\,p_y A_{vN}$

A_{vN} = $\big(e_t + (n-1)\,p + e_b\big)t_w$ (for single notched beam)
= $0.9\big(e_t + (n-1)\,p + e_b\big)t_w$ (for double notch beam)

t_1 = end projection

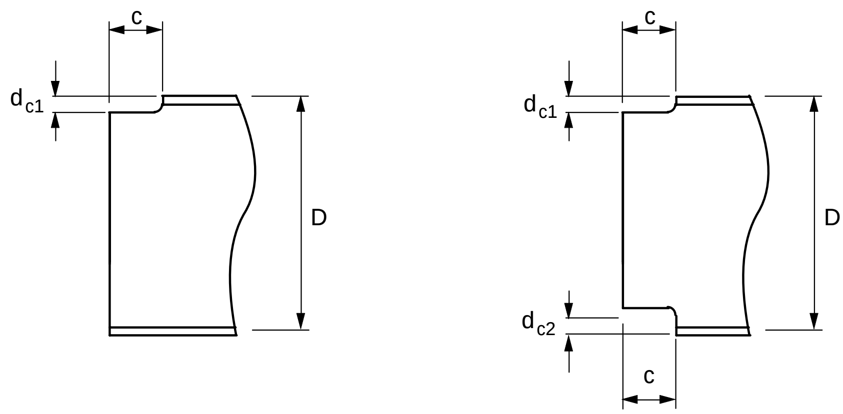
t_w = thickness of supported beam web

c = length of notch

Z_N = elastic section modulus of the gross tee section at the notch

e_3, g_b as per CHECK 4

CHECK 6	Supported beam - Local stability of notched beam



When the beam is restrained against lateral torsional buckling, no account need be taken of notch stability provided the following conditions are met:

For one flange notched [14],[15]

Basic requirement:

d_{c1}	$\leq$	D/2 **and:**			
c	$\leq$	D	for	$D/t_w \leq 54.3$	(S275 steel)
c	$\leq$	$\frac{160000D}{(D/t_w)^3}$	for	$D/t_w > 54.3$	(S275 steel)
c	$\leq$	D	for	$D/t_w \leq 48.0$	(S355 steel)
c	$\leq$	$\frac{110000D}{(D/t_w)^3}$	for	$D/t_w > 48.0$	(S355 steel)

For both flanges notched [15]

Basic requirement:

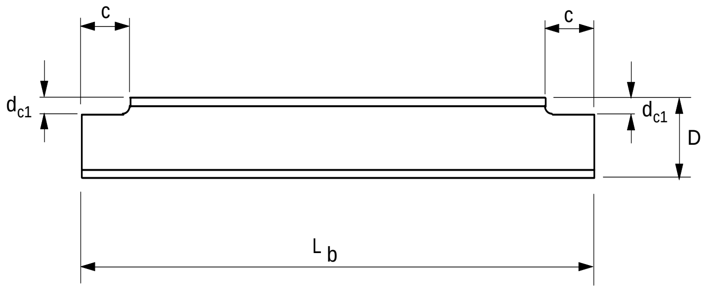
Max (d_{c1} , d_{c2})	$\leq$	D/5 **and:**			
c	$\leq$	D	for	$D/t_w \leq 54.3$	(S275 steel)
c	$\leq$	$\frac{160000D}{(D/t_w)^3}$	for	$D/t_w > 54.3$	(S275 steel)
c	$\leq$	D	for	$D/t_w \leq 48.0$	(S355 steel)
c	$\leq$	$\frac{110000D}{(D/t_w)^3}$	for	$D/t_w > 48.0$	(S355 steel)

where:

t_w = thickness of supported beam web

Where the notch length c exceeds these limits, either suitable stiffening should be provided or the notch should be checked to references 14, 15 and 16.

CHECK 7	Unrestrained Supported Beam Overall stability of notched beam



When a notched beam is <u>unrestrained</u> against lateral torsional buckling, the overall stability of the beam should be checked.

Notes:

(1) This check is only applicable for beams with one flange notched. Guidance on double-notched beams is given in Section 5.12 of Reference 17.

(2) If the notch length c and/or notch depth d_{c1} are different at each end, then the larger values for c and d_{c1} should be used.

(3) Beams should be checked for lateral torsional buckling to BS 5950-1[1], clause 4.3 with a modified effective length (L_E) which takes account of notches.

(4) The solution below gives the modified effective length (L_E) based on references 18, 19 and 20. It is only valid for $c/L_b < 0.15$ and $d_{c1}/D < 0.2$ (beams with notches outside these limits should be checked as tee sections, or stiffened).

$$L_E = L_b \left(1 + \frac{2c}{L_b}(K^2 + 2K)\right)^{1/2}$$

$$K = K_o/\lambda_b$$

$$\lambda_b = \frac{u\,v\,L_b}{r_y}$$

where: x, u, v and r_y are for the un-notched **I** beam section and are defined in BS 5950-1
Conservatively, $u = 0.9$ and $v = 1.0$

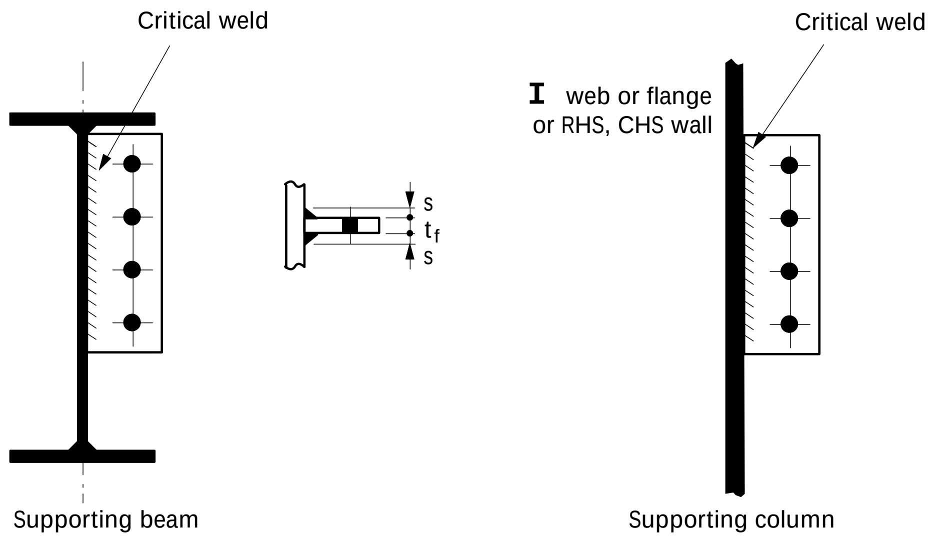
for $\lambda_b < 30$ $\quad K_o = 1.1\, g_o\, x$ but $\leq 1.1\, K_{max}$

for $\lambda_b \geq 30$ $\quad K_o = g_o\, x$ but $\leq K_{max}$

g_o and K_{max} are tabulated below:

$\frac{c}{L_b}$	g_o	K_{max}	
		UB section	UC section
≤ 0.025	5.56	260	70
0.050	5.88	280	80
0.075	6.19	290	90
0.100	6.50	300	95
0.125	6.81	305	95
0.150	7.13	315	100

CHECK 8	Supporting beam/Column – Welds



Strength of weld connecting fin plate to supporting beam or column under bending moment and shear

Basic requirement:

$$s \geq 0.8t_f$$

where:

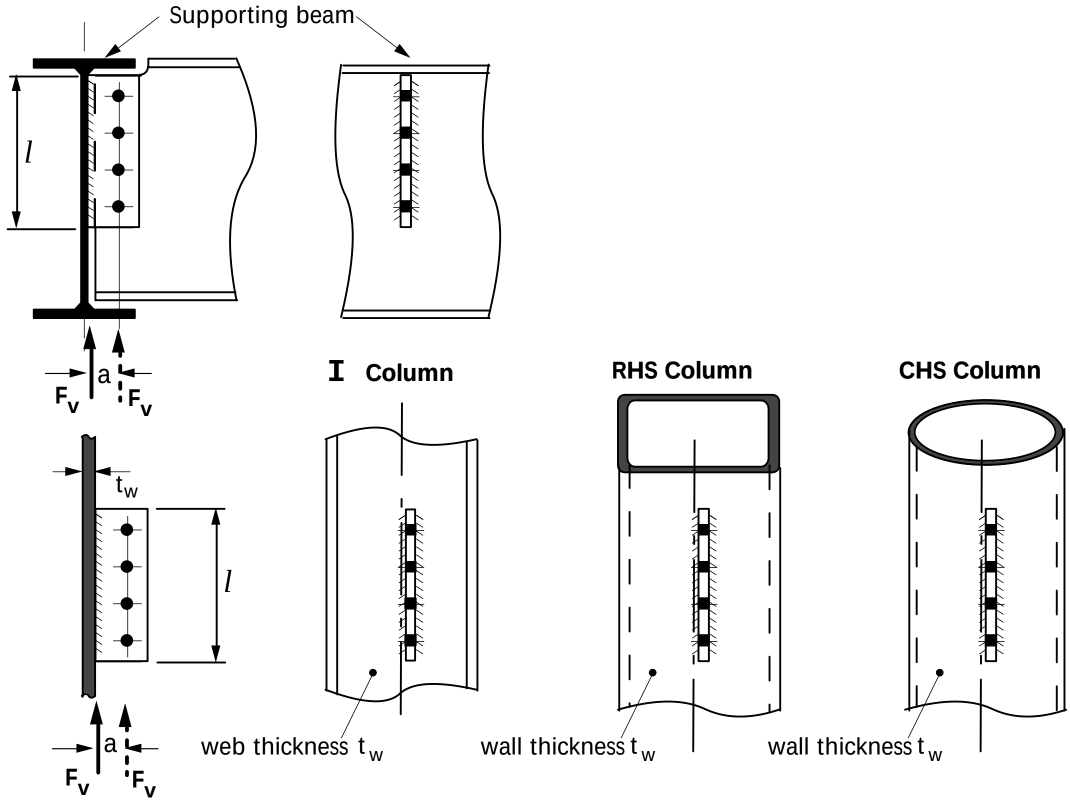
s = leg length of fillet weld

t_f = fin plate thickness

Note: This check ensures that the weld is not the weakest part of the connection.

CHECK 9	***Not applicable (see Table 3.1)***

CHECK 10	Supporting beam/column - Local capacity **(with one supported beam)**



Local shear and punching shear capacity of :

- **beam web supporting one beam**
- **web of I column supporting one beam**
- **wall of RHS or CHS column supporting one beam**

(i) Local shear capacity:

Basic requirement:

$$\frac{F_v}{2} \leq P_v$$

P_v = local shear capacity of supporting beam web or column web or column wall

$= 0.6\, p_y\, A_v$

$A_v = 0.9\, l\, t_w$

(ii) Punching shear capacity:

Provided that either the conservative or rigorous requirement given below is satified, yielding of the fin plate occurs before punching shear failure of the supporting member:

Basic requirement (either will suffice):

$$t_f \leq t_w \left(\frac{U_{sc}}{p_{yf}}\right) \quad \textbf{OR} \quad t_f \leq t_w \left(\frac{U_{sc}}{f_b}\right)$$

(Conservative) (Rigorous)

where:

t_w = thickness of supporting beam web, column web or column wall

t_f = fin plate thickness

U_{sc} = Ultimate tensile strength of supporting member

p_{yf} = design strength of fin plate

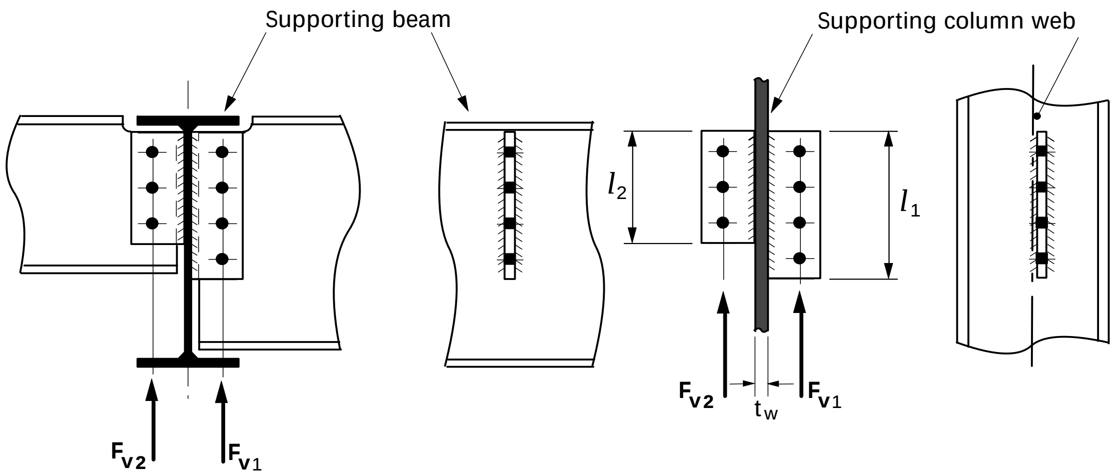
f_b = maximum bending stress in the fin plate

$= \dfrac{F_v\, a}{Z_{gross}}$ but $f_b \leq p_{yf}$

Z_{gross} = elastic modulus of gross section of the fin plate

$= \dfrac{t_f\, l^2}{6}$

CHECK 10 continued	Supporting beam/column - Local capacity **(with two supported beams)**



(iii) Local shear capacity of supporting beam web or column web supporting two beams:

Basic Requirement:

$$\frac{F_{v1A}}{2} + \frac{F_{v2}}{2} \leq P_v$$

where:

$F_{v1A} = F_{v1} \dfrac{l_2}{l_1}$

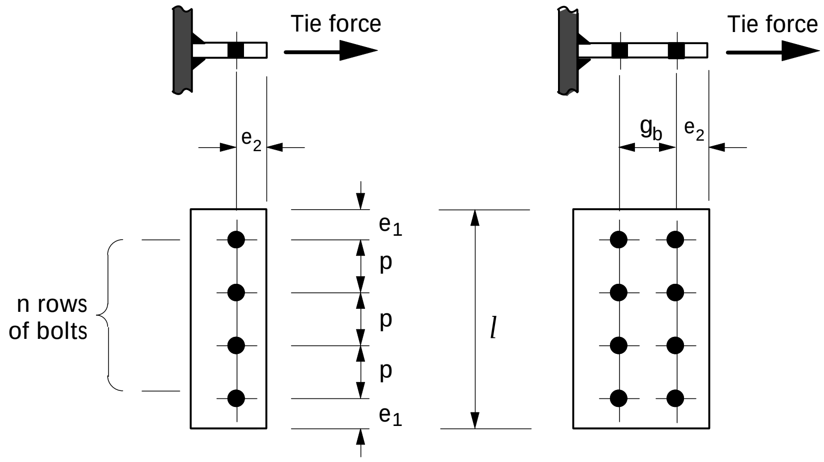
P_v = local shear capacity of supporting beam web or column web

$= 0.6\ p_y\ A_v$

$A_v = 0.9\, l_2\, t_w$

t_w = thickness of supporting beam web or column web

CHECK 11	Structural integrity - connecting elements



Single line of bolts **Double line of bolts**

Note: This check is only needed if it is necessary to comply with structural integrity requirements.

To resist a tie force of 75kN, the connection must have at least 2 no. M20, 8,8 bolts and a fin plate thickness ≥ 6mm.

Structural integrity – tension and bearing capacity of fin plate

(i) For tension:

Basic requirement:

Tie force ≤ Tension capacity of fin plate

Tension capacity of fin plate

$= \min (p_y A\ ,\ K_e\, p_y\, A_{net})$

$A = l\ t_f$

$A_{net} = A - nD_h t_f$

(ii) For bearing:

Basic requirement:

Tie force ≤ Bearing capacity of fin plate

Bearing capacity of fin plate

$= 1.5n\, d\, t_f\, p_{bs}$ but

$\leq 0.5n\, e_2\, t_f\, p_{bs}$

(for single line of bolts)

$= 3n\, d\, t_f\, p_{bs}$ but

$\leq n\left(1.5d\, t_f\, p_{bs} + 0.5\, e_2\, t_f\, p_{bs}\right)$

(for double lines of bolts)

where:

p = bolt pitch

d = diameter of bolt

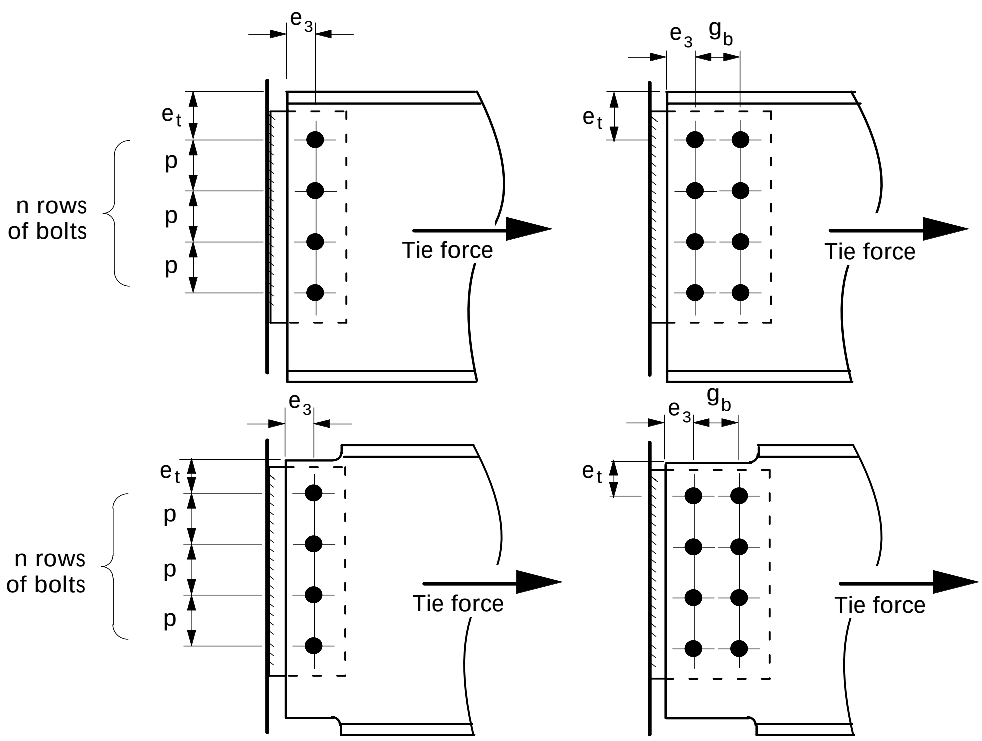
D_h = diameter of hole

t_f = thickness of fin plate

p_{bs} = bearing strength of the plate (BS 5950-1, Table 32)

K_e = 1.2 for S275 steel
1.1 for S355 steel

CHECK 12	Structural Integrity – supported beam



Note: This check is only needed if it is necessary to comply with structural integrity requirements

Structural integrity – tension and bearing capacity of beam web

(i) For tension

Basic requirement:

Tie force $\leq$ Net tension capacity of beam web

Net tension capacity of beam web

$= L_e t_w p_y$

(ii) For bearing

Basic requirement:

Tie force $\leq$ Bearing capacity of beam web.

Bearing capacity of beam web

$= 1.5n\, d\, t_w\, p_{bs}$ but

$\leq 0.5ne_3 t_w\, p_{bs}$

for single line of bolts

$= 3n\, d\, t_w\, p_{bs}$ but

$\leq n\left(1.5d\, t_w\, p_{bs} + 0.5\, e_3\, t_w p_{bs}\right)$

for double line of bolts

where:

L_e = effective net length

$= 2e_e + (n-1)p_e - nD_h$

$e_e = e_3$ but $\leq e_t$ for single line of bolts

$e_e = e_3 + g_b - D_h$ but $\leq e_t$ for double line of bolts

$p_e = p$ but $\leq 2e_3$ for single line of bolts

$p_e = p$ but $\leq 2(e_3 + g_b - D_h)$ for double line of bolts

t_w = beam web thickness

p = bolt pitch

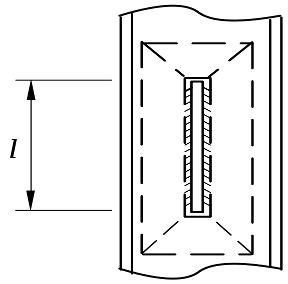
D_h = diameter of hole

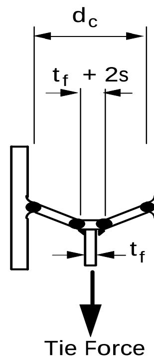
d = diameter of bolt

p_{bs} = bearing strength of beam web (BS 5950–1, Table 32)

CHECK 13	***Not applicable (see Table 3.1)***

CHECK 14	Structural integrity – Supporting column web (UC or UB) **H**





Fin plate connecting to column web

This check is only needed if it is necessary to comply with structural integrity requirements

Structural integrity – Tying capacity of rolled column web, in the presence of axial compression in the column

Basic requirement:

Tie force $\leq$ Tying capacity of column web

Tying capacity of column web $= \dfrac{8\, M_u}{1 - \beta_1}\left(\eta_1 + 1.5(1 - \beta_1)^{0.5}\right)$ *

M_u = moment capacity of column web per unit length

$= \dfrac{p_u\, t_w^{\,2}}{4}$

p_u = design tensile strength of the column

$= U_s / 1.25$ (see inset box)

* Factor 1.5 in the equation includes an allowance for the axial compression in the column

where:

$\eta_1 = \dfrac{l}{d_c}$

$\beta_1 = \dfrac{t_f + 2s}{d_c}$

t_f = thickness of fin plate

l = length of fin plate

d_c = depth of column between fillets

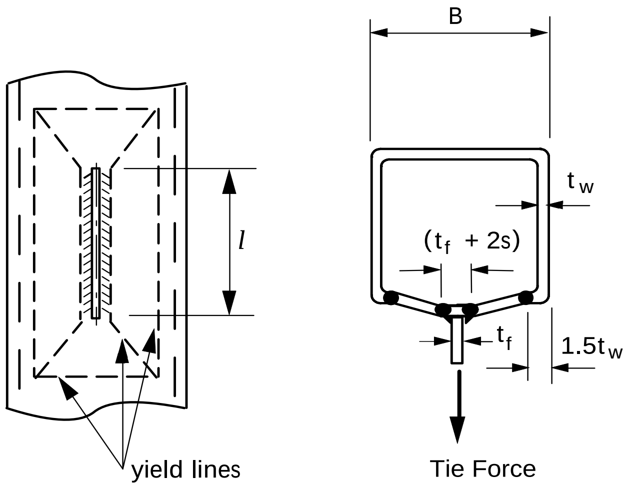
t_w = thickness of column web

s = leg length of weld

Design tensile strength	
	$p_u = U_s / 1.25$
S275	328 N/mm^2
S355	392 N/mm^2
Values of U_s taken from BS EN 10025: 1993[21]	

Note: The check is required for either single-sided connections to the rolled column web or unequally loaded double-sided connections to the rolled column web.

CHECK 15	Structural integrity – supporting column wall (RHS)



Fin plate connecting to RHS column

This check is only needed if it is necessary to comply with structural integrity requirements

Structural integrity – Tying capacity of RHS wall, in the presence of axial compression in the column

Basic requirement:

Tie force $\leq$ Tying capacity of RHS column wall

Tying capacity RHS column wall $= \dfrac{8\,M_u}{1-\beta}\left(\eta + 1.5(1-\beta)^{0.5}\right)$ *

M_u = moment capacity of RHS column wall per unit length

$= \dfrac{p_u\, t_w^{\,2}}{4}$

p_u = design tensile strength of the RHS column

$= U_s/1.25$ (see inset box)

* Factor 1.5 in the equation includes an allowance for the axial compression in the column

where:

$\eta = \dfrac{l}{(B - 3t_w)}$

$\beta = \dfrac{(t_f + 2s)}{(B - 3t_w)}$

t_f = thickness of fin plate

l = length of fin plate

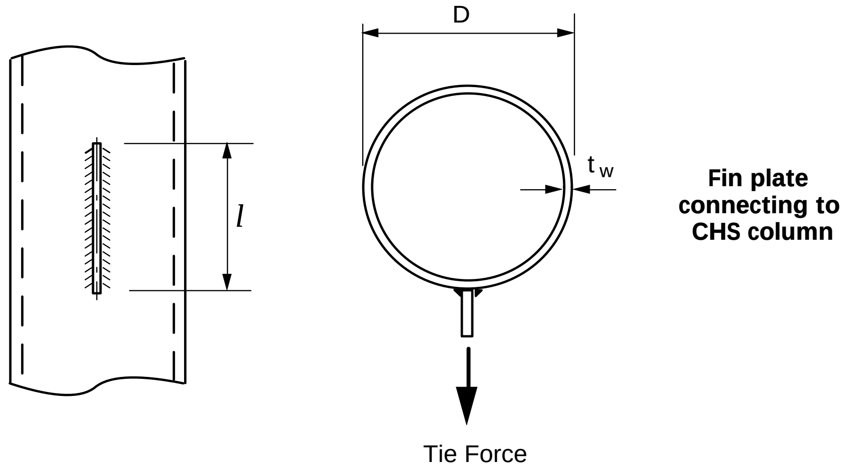
B = overall width of RHS column wall to which the connection is made

t_w = thickness of RHS column wall

s = leg length of weld

Design tensile strength	
	$p_u = U_s/1.25$
S275	328 N/mm²
S355	392 N/mm²
Values of U_s taken from BS EN 10201 - 1:1994[3]	

CHECK 16	Structural integrity – Supporting column wall (CHS)



This check is only needed if it is necessary to comply with structural integrity requirements

Structural integrity – Tying capacity of CHS wall, in the presence of axial compression in the column

Basic requirement:

Tie force $\leq$ Tying capacity of CHS column wall

$$\text{Tying capacity CHS column wall} = 5\, p_u\, t_w^2\, (1 + 0.25\eta) \times 0.67 \quad *$$

p_u = design tensile strength of the CHS column

= $U_s / 1.25$ (see inset box)

* Factor 0.67 in the equation includes an allowance for the axial compression in the column

where:

$\eta = \dfrac{l}{D}$ but ≤ 4

l = length of fin plate

D = diameter of CHS column

t_w = thickness of CHS column wall

Design tensile strength	
	$p_u = U_s / 1.25$
S275	328 N/mm^2
S355	392 N/mm^2
Values of U_s taken from BS EN 10210 - 1:1994[3]	

6.6 WORKED EXAMPLES

The worked examples show design calculations for typical standard connections. Each example demonstrates first the use of the capacity tables (yellow pages) and then full checks according to the procedures in Section 6.5. The full checks will normally only need to be applied to non-standard connections but their application to standard connections demonstrates the validity of the much simpler process when using standard details.

Check 7, dealing with overall stability of an unrestrained beam, should be undertaken by the member designer taking account of any notching required at the ends of the supported beam in order to facilitate the use of a simple connection.

Checks 11 to 15 deal with structural integrity in the presence of an axial tie force required to be developed in some members to ensure the steel frame is sufficiently robust, or in the case of some multi-storey buildings, to localise accidental damage. When tying capacity is not required these checks may be omitted.

Example 1

This example covers the design checks for a two sided beam-to-beam connection with fin plates welded to the web only of the supporting beam. A fin plate with a single line of vertical bolts is employed on one side and another with a double line of bolts is used to connect a larger beam, with a heavier vertical reaction, on the other side.

Example 2

Example 2 demonstrates the additional structural integrity design checks required for a beam-to-column web connection when an axial tie force must be sustained by the connection.

Example 3

Example 3 is a beam-to-RHS column connection using fin plates welded to the column wall. The beam sizes and vertical reactions as in Example 1, so only checks which are different to the those in the first example are shown.

The connection design also considers tie forces in the beams but it should be noted that the tie forces are ignored in checks for vertical reactions and vertical reactions are ignored in checks for tie forces.

Example 4

Example 4 shows the same beam connections as in Example 3 but uses fin plates welded to a CHS column. Tie forces are taken into account separately from the vertical reactions.

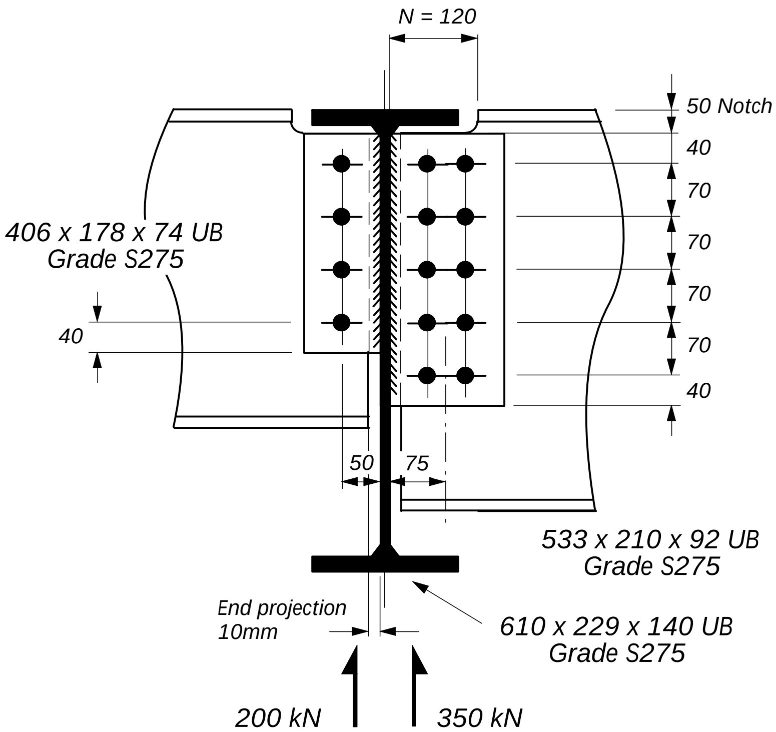
CALCULATION SHEET BCSA	Job No Joints in Steel Construction - Simple Connections		Sheet 1 of 12
	Title Example 1 – Fin Plates - Beam to Beam		
	Client SCI/BCSA Connections Group		
	Calcs by RS	Checked by AM	Date May 2002

DESIGN EXAMPLE 1

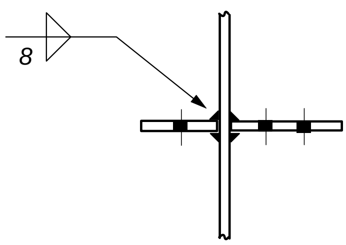
Check the following beam to beam connection for the design forces shown.

Yellow pages used for the initial selection of Fin Plates.

A fin plate with a single vertical line of bolts would be adequate for both connections in this example, but a double vertical line was used for the 533 x 229 x 92 UB in order to demonstrate the design checks required for this configuration.



See Figure 6.4



Fin Plates: *100 x 10 (Single line Type FA4)* *150 x 10 (Double line Type FB5)*

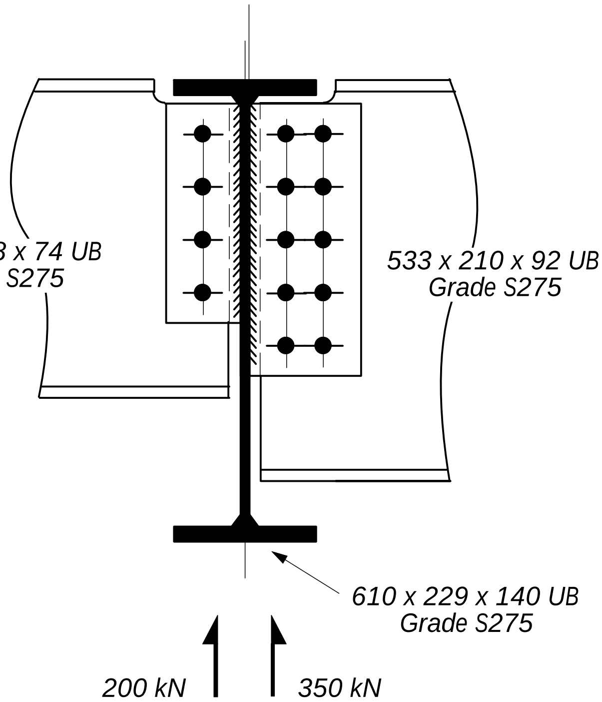
See figure 6.4 and Yellow pages Table H.4

Design Information:

Bolts:	*M20 8.8*
Welds:	*8mm Fillet Weld*
Material:	*All S275*

Title	Example 1 – Fin Plates - Beam to Beam	Sheet	2 of 12

CONNECTION DESIGN USING CAPACITY TABLES FROM YELLOW PAGES



Fin Plate type FA4 Grade S275	Fin Plate type FB5 Grade S275	
Bolts M20 8.8	**Bolts M20 8.8**	
From capacity table H.27 in yellow pages:	*From capacity table H.28 in yellow pages:*	*Yellow pages Table H.27 and H.28*
Connection shear capacity **= 265kN > 200kN**	*Connection shear capacity* **= 476kN > 350kN**	**∴ O.K.**
Maximum notch length $(c + t_1)$ **= 336mm > 120mm**	*Maximum notch length* $(c + t_1)$ **= 345mm > 120mm**	**∴ O.K.**

Web thickness of supporting beam = 13.1mm — *Table H.64*

Minimum support thickness based on the maximum shears of 265kN and 476kN

$$= (6.2 + 8.2) = 14.4\text{mm} \nless 13.1\text{mm}$$ **∴ Fails**

but for shear forces of 200kN and 350 kN the minimum required thickness

$$= 6.2 \times \frac{200}{265} + 8.2 \times \frac{350}{476}$$

$$= 4.7\text{mm} + 6.0\text{mm}$$

$$= \mathbf{10.7mm < 13.1mm}$$ **∴ O.K.**

The connections are adequate.

Title	*Example 1 – Fin Plates - Beam to Beam*	Sheet	*3 of 12*

SUMMARY OF FULL DESIGN CHECKS FOR EXAMPLE 1

Sheet Nos	***CHECK***		***406UB (S275)***		***533UB (S275)****		***610UB (S275)***	
			Capacity	***Applied Load***	***Capacity***	***Applied Load***	***Capacity***	***Applied Load***
4	***CHECK 1*** *- Recommended detailing practice*		*All recommendations adopted*					
4	***CHECK 2*** *Supported beam - Bolt group shear capacity*	*Capacity per bolt (kN)*	***87.4***	***65.9***	*92*	*53.9*	*Not Applicable*	
			CRITICAL CHECK FOR 406 UB					
5 & 6	***CHECK 3*** *Supported beam* *- Connecting elements* *(Strength of fin plate)*	*Shear (kN)*	*400*	*200*	***494***	***350***	*Not Applicable*	
					CRITICAL CHECK FOR 533 UB			
		Bending capacity (kNm)	*38.6*	*10*	*59.4*	*17.5*		
	Lateral torsional buckling resistance of long fin plate		*Plate thickness $t_f > 0.15a$* *$\therefore$ both fin plates are 'short'*					
7 to 9	***CHECK 4*** *Supported Beam* *- Capacity at connection* *(notched beam)*	*Shear (kN)*	*446* *(block shear)*	*200*	*603* *(block shear)*	*350*	*Not Applicable*	
		Bending capacity (kNm)	*N/A*	*N/A*	*164*	*35*		
10	***CHECK 5*** *Supported Beam* *- Capacity at a notch*	*Bending capacity (kNm)*	*89.1*	*24*	*164*	*42*	*Not Applicable*	
11	***CHECK 6*** *Supported Beam* *- Local stablity of notched beam* *(Beam restrained)*	*Notch length (mm)*	*412.8*	*110*	*533.1*	*110*	*Not Applicable*	
			Notch length c < Specified limits					
11	***CHECK 7*** *Supported Beam* *- Lateral torsional buckling* *(Beam restrained)*	*-*	*Not Applicable*				*Not Applicable*	
12	***CHECK 8*** *Supporting Beam* *- Fin plate weld*	*$s \geq 0.8t_f$ (mm)*	*($0.8t_f$)* *8*	*(s)* *8*	*($0.8t_f$)* *8*	*(s)* *8*	*Not Applicable*	
12	***CHECK 9***	*-*	*Not Applicable*				*Not Applicable*	
12	***CHECK 10*** *Supporting Beam* *- Local Capacity of beam web*	*Shear (kN)*	*Not Applicable*				*564*	*241*

** A fin plate with a single vertical line of bolts would be adequate in this example but a double vertical line was used in order to demonstrate the design checks required for this configuration.*

Title	Sheet
Example 1 – Fin Plates - Beam to Beam	*4 of 12*

Check 1: Recommended detailing practice

Fin plate thickness: t_f = 10mm $\leq 0.5d$ (d = 20mm dia)

Fin plate length: l = 290mm (>0.6D for 406 UB)

= 360mm (>0.6D for 533 UB)

Bolts: M20 8.8

Check 2: Supported beam - Bolt group

Basic requirement: $F_s \leq P_{bs}$

For 406 x 178 x 74 UB grade S275

$$P_{bs} = \min(d\, t_f\, p_{bs.f},\ d\, t_w\, p_{bs.b})$$

$$= \frac{20 \times 10 \times 460}{10^3}\ ,\ \frac{20 \times 9.5 \times 460}{10^3}$$

$$= 87.4\text{kN}$$

p_{bs} from BS5950–1 Table 32

For single line of bolts:

Resultant force per bolt, $F_s = \left(F_{sv}{}^2 + F_{sm}{}^2\right)^{1/2}$

Direct force per bolt, $F_{sv} = \dfrac{F_v}{n} = \dfrac{200}{4} = 50\text{kN}$

Eccentric bending moment, $M = F_v\, a = 200 \times 0.05 = 10\text{kNm}$

Modulus of bolt group, $Z_{bg} = \dfrac{n(n+l)\,p}{6}$

$$= \frac{4 \times 5 \times 70}{6} = 233\text{mm}$$

Force on the outermost bolt due to moment, $F_{sm} = \dfrac{F_v\, a}{Z_{bg}} = \dfrac{10 \times 10^3}{233} = 42.9\text{kN}$

Resultant force per bolt, $F_s = \left(50^2 + 42.9^2\right)^{1/2} = 65.9\text{kN}$

$$\mathbf{F_s = 65.9kN < 87.4kN} \qquad \therefore \textbf{ O.K.}$$

For 533 x 210 x 92 UB grade S275

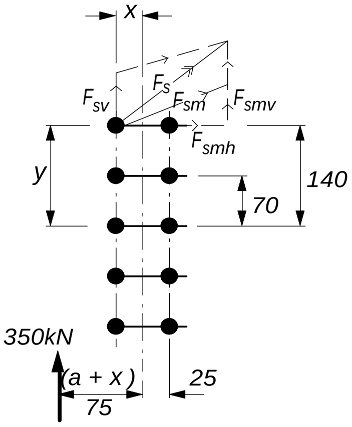
$$P_{bs} = \min\left(\frac{20 \times 10 \times 460}{10^3}\ ,\ \frac{20 \times 10.1 \times 460}{10^3}\right) = 92\text{kN}$$

For double line of bolts:

Resultant force per bolt, $F_s = \left((F_{sv} + F_{smv})^2 + F_{smh}{}^2\right)^{1/2}$

Direct force per bolt, $F_{sv} = \dfrac{F_v}{2n} = \dfrac{350}{10} = 35\text{kN}$

Eccentric bending moment, $M = F_v\,(a + x) = 350 \times 0.075 = 26.3\text{kNm}$



Inertia of bolt group,

$$I_{bg} = \Sigma S^2 = 4(25^2 + 140^2) + 4(25^2 + 70^2) + 2 \times 25^2$$

$$= 104250\text{mm}^4$$

$$F_{smh} = \frac{M\,y}{I_{bg}} = \frac{26.3 \times 140 \times 10^3}{104250} = 35\text{kN}$$

$$F_{smv} = \frac{M\,x}{I_{bg}} = \frac{26.3 \times 25 \times 10^3}{104250} = 6\text{kN}$$

$$F_s = \left((35 + 6)^2 + 35^2\right)^{1/2} = 53.9\text{kN}$$

$$\mathbf{F_s = 53.9kN < 92kN} \qquad \therefore \textbf{ O.K.}$$

Title *Example 1 – Fin Plates - Beam to Beam*	Sheet *5 of 12*

Check 3: Supported Beam - Connecting Elements

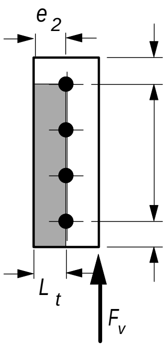
(i) For shear:

Basic requirement: $F_v < P_{v.min}$

For 406 x 178 x 74 UB Side

Shear capacity of fin plate $P_{v.min}$ is the smaller of Plain shear capacity P_v and Block shear capacity P_r

Plain shear capacity of fin plate, $P_v = \min(0.6\, p_y A_v\, ,\ 0.7\, p_y k_e A_{v.net})$



Shear area, $A_v = 0.9\big(2e_1 + (n - l)p\big)\, t_f$

$= 0.9\big(80 + 210\big) \times 10$

$= 2610\text{mm}^2$

Net area, $A_{v.net} = A_v - n\, D_h\, t_f$

$= 2610 - (4 \times 22 \times 10) = 1730\text{mm}^2$

$0.6\, p_y A_v = \dfrac{0.6 \times 275 \times 2610}{10^3} = 431\text{kN}$

$0.7\, p_y K_e\, A_{v.net} = \dfrac{0.7 \times 275 \times 1.2 \times 1730}{10^3} = 400\text{kN}$

$\therefore P_v = 400\text{kN}$

Block shear capacity of fin plate, $P_r = 0.6\, p_y\, t_f \big(L_v + K_e\, (L_t - kD_h)\big)$ *See note below*

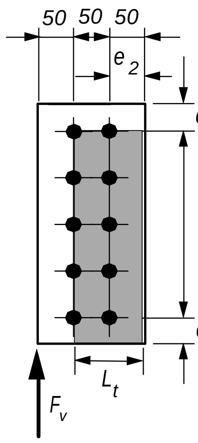
$\big($*where* $k = 0.5,\ L_t = e_2$ *and* $L_v = e_1 + (n - 1)p\big)$

$P_r = \dfrac{0.6 \times 275 \times 10\big(250 + 1.2(50 - 0.5 \times 22)\big)}{10^3} = 490\text{kN}$

$\therefore P_{v.min} = \min(P_v\, ,\ P_r) = 400\text{kN}$

$\mathbf{F_v = 200kN < 400kN}$ **∴ O.K.**

For 533 x 210 x 92 UB Side



$A_v = 0.9\big(80 + 280\big)10 = 3240\text{mm}^2$

$A_{v.net} = 3240 - (5 \times 22 \times l0) = 2140\text{mm}^2$

$0.6\ p_y\ A_v = \dfrac{0.6 \times 275 \times 3240}{10^3} = 535\text{kN}$

$0.7\, p_y K_e\ A_{v.net} = \dfrac{0.7 \times 275 \times 1.2 \times 2140}{10^3} = 494\text{kN}$

Block shear capacity of fin plate, $P_r = 0.6\, p_y\, t_f\big(L_v + K_e(L_t - kD_h)\big)$

(where: $k = 2.5,\ L_t = 100,$ *and* $L_v = e_1 + (n - 1)p$*) see note below*

$P_r = \dfrac{0.6 \times 275 \times 10\big(250 + 1.2(100 - 2.5 \times 22)\big)}{10^3} = 617\text{kN}$

$\therefore P_{v.min} = \min(P_v\, ,\ P_r) = 494\text{kN}$

$\mathbf{F_v = 350kN < 494kN}$ **∴ O.K.**

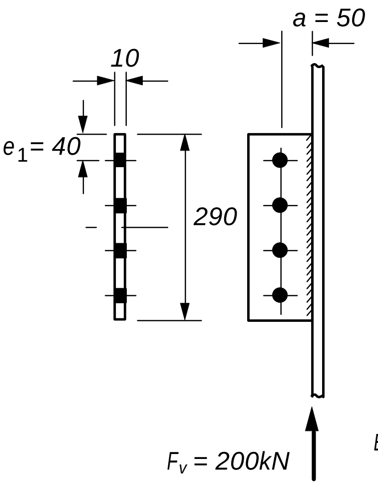
NOTE: *Block shear checks have been shown here, but they are never critical for well proportioned fin plates. However, if the bolt spacing is concentrated at one part of the plate then these checks may be critical.*

Title *Example 1 – Fin Plates - Beam to Beam*	Sheet *6 of 12*

(ii) Basic requirement for shear and bending interaction:

$$F_v a \leq M_c$$

For 406 x 178 x 74 UB Side



$0.75\, P_{v.min} = 0.75 \times 400 = 300kN$

$F_v = 200kN < 300kN$

$\therefore$ low shear criteria for bending applies

$$\therefore M_c = \frac{p_y t_f}{6}\left(2e_1 + (n-1)p\right)^2$$

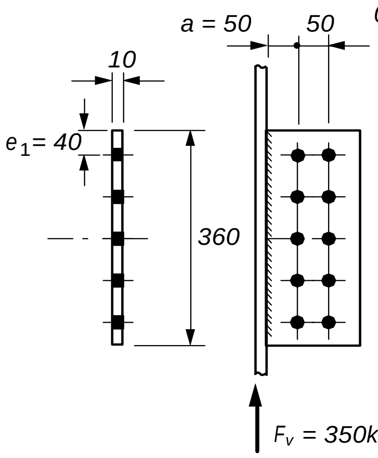
$$= \frac{275 \times 10}{6 \times 10^6}\left(2 \times 40 + 210\right)^2$$

$$= 38.6kNm$$

Eccentric moment, $F_v a = 200 \times 0.05 = 10kNm$

$\boldsymbol{F_v a = 10kNm < 38.6kNm}$ **∴ O.K.**

For 533 x 210 x 92 UB Side



$0.75\, P_{v.min} = 0.75 \times 494 = 371kN$

$F_v = 350kN < 371kN$

$\therefore$ low shear criteria for bending applies

$$\therefore M_c = \frac{275 \times 10}{6 \times 10^6}\left(2 \times 40 + 280\right)^2$$

$$= 59.4kNm$$

Eccentric moment $F_v a = 350 \times 0.05 = 17.5kNm$

$\boldsymbol{F_v a = 17.5kNm < 59.4kNm}$ **∴ O.K.**

(iii) Lateral Torsional Buckling Resistance of Long Fin Plates:

This check is only required where $t_f < 0.15a$

For 406 x178 x 74 UB and 533 x 210 x 92 UB:

$0.15a = 0.15 \times 50$

$= 7.5mm$

$t_f = 10mm$ $\therefore$ *Lateral Torsional Buckling check not required*

Note: *For a double vertical line of bolts the eccentricity is taken from the support to the first vertical line of bolts.*

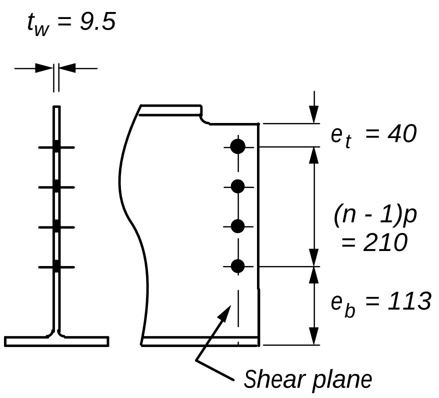
Title Example 1 – Fin Plates - Beam to Beam	Sheet 7 of 12

Check 4: Supported Beam - Capacity at the connection

(i) Basic requirement for shear: $F_v \le P_{v.min}$

where $P_{v.min}$ is the smaller of the plain shear capacity, P_v or the block shear capacity, P_v of the supported beam

For 406 x 178 x 74 UB grade S275



Plain shear capacity, P_v $= \min\left(0.6\ p_y A_v\ ,\ 0.7 p_y K_e A_{v.net}\right)$

$A_v = (e_t + (n-1)p + e_b)t_w$ for single notched/plain beam

$A_v = (40 + 210 + 113) \times 9.5$

$= 3449mm^2$

$A_{v.net} = A_v - n D_h t_w$

$= 3449 - (4 \times 22 \times 9.5)$

$= 2613mm^2$

Bolt diameter, d = 20mm

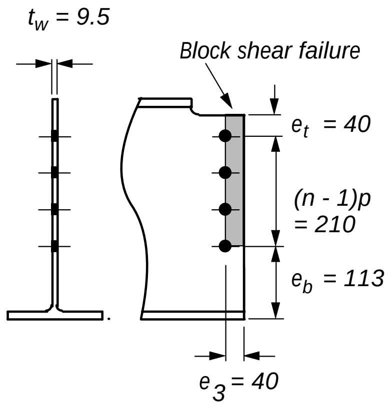
Hole diameter, D_h = 22mm

$0.6 p_y A_v = \dfrac{0.6 \times 275 \times 3449}{10^3} = 569kN$

$0.7 p_y K_e A_{v.net} = \dfrac{0.7 \times 275 \times 1.2 \times 2613}{10^3} = 604kN$

$\therefore P_v = 569kN$

Block shear capacity, P_r $= 0.6 p_y\ t_w (L_v + K_e(L_t - kD_h))$



$L_v = e_t + (n-1)p$

$= 40 + 210$

$= 250mm$

$K_e = 1.2$ (for S275 steel)

$k = 0.5$ (for single line of bolts)

$L_t = e_3$ (for single line of bolts)

$\therefore P_r = \dfrac{0.6 \times 275 \times 9.5(250 + 1.2(40 - 11))}{10^3}$

$= 446kN$

Shear capacity of supported beam $P_{v.min} = \min(P_v\ ,\ P_r) = 446kN$

Applied shear F_v **= 200kN < 446kN** **∴ O.K.**

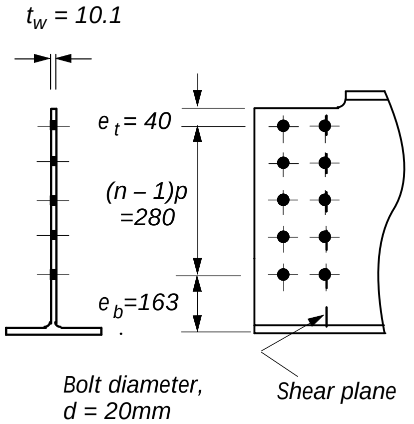
Title	Example 1 – Fin Plates - Beam to Beam	Sheet	8 of 12

For 533 x 210 x 92 UB grade S275

Plain shear capacity $P_v = \min\left(0.6\ p_y A_v\ ,\ 0.7\, p_y K_e A_{v.net}\right)$

$A_v = \left(e_t + (n-1)\,p + e_b\right)t_w$

for single notched beam



Bolt diameter, d = 20mm

Hole diameter, D_h = 22mm

$A_v = (40 + 280 + 163) \times 10.1$

$= 4878mm^2$

$A_{v.net} = A_v - n\,D_h\,t_w$

$= 4878 - (5 \times 22 \times 10.1)$

$= 3767mm^2$

$0.6\,p_y A_v = \dfrac{0.6 \times 275 \times 4878}{10^3} = 805kN$

$0.7\,p_y K_e A_{v.net} = \dfrac{0.7 \times 275 \times 1.2 \times 3767}{10^3} = 870kN$

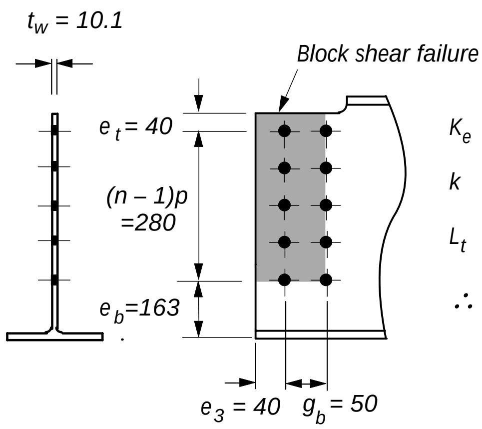
$\therefore P_v = 805kN$

Block shear capacity $P_r = 0.6\,p_y\ t_w\left(L_v + K_e\left(L_t - kD_n\right)\right)$

$L_v = e_t + (n-1)p$

$= 40 + 280$

$= 320mm$



$K_e = 1.2$ (for S275 steel)

$k = 2.5$ (for double line of bolts)

$L_t = e_3 + g_b$ (for double line of bolts)

$\therefore p_r = \dfrac{0.6 \times 275 \times 10.1\left(320 + 1.2(90 - 55)\right)}{10^3}$

$= 603kN$

Shear capacity of supported beam $P_{v.min} = \min\left(P_v\ ,\ P_r\right)$

$\therefore P_v = 603kN$

Applied shear $\boldsymbol{F_v = 350kN < 603kN}$ **$\therefore$ O.K.**

Title	Example 1 – Fin Plates - Beam to Beam	Sheet	9 of 12

(ii) Shear and bending interaction at the 2nd line of bolts is required if $c > e_3 + g_b$

For 533 x 210 x 92 UB grade S275

$c = 110mm > 90mm$

$\therefore$ Interaction check required

$0.75\ P_{v.min} = 0.75 \times 603 = 452kN$

$350kN < 452kN$

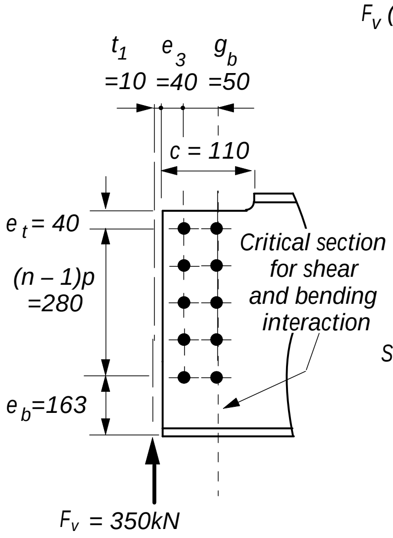
$\therefore$ low shear criteria for bending applies

Basic requirement for bending:

$$F_v\ (t_1 + e_3 + g_b) \leq M_{cC}$$

$$F_v\ (t_1 + e_3 + g_b) = \frac{350\ (10 + 40 + 50)}{10^3} = 35kNm$$

$$M_{cC} = p_y Z$$



For Gross tee Section:

Taking moments of area about bottom flange:

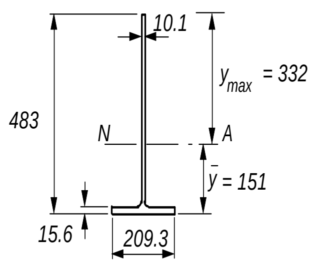
$$(209.3 \times 15.6 \times 7.8) + (467.4 \times 10.1 \times 249.3) = \left[(209.3 \times 15.6) + (467.4 \times 10.1)\right] \times \bar{y}$$

$$\bar{y} = 151mm$$

Second moment of area about neutral axis:

$$I_{xx} = \frac{1}{10^4}\left[\frac{209.3 \times 15.6^3}{12} + (209.3 \times 15.6 \times 143.2^2)\right] + \frac{1}{10^4}\left[\frac{10.1 \times 467.5^3}{12} + (467.4 \times 10.1 \times 98.3^2)\right]$$

$$= 19858cm^4$$



$$Z = \frac{I_{xx}}{y_{max}} = \frac{19858}{33.2} = 598\ cm^3$$

$$M_{cC} = \frac{275 \times 598}{10^3} = 164kNm$$

$$\mathbf{F_v\ (t_1 + e_3 + g_b) = 35kNm < 164kNm}$$

$\therefore$ O.K.

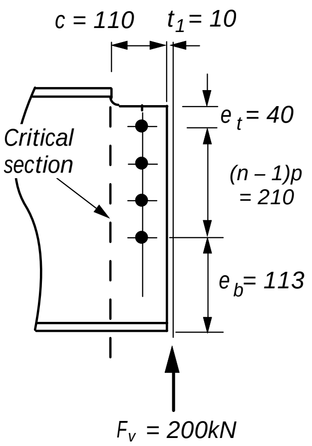
Title	Example 1 – Fin Plates - Beam to Beam	Sheet	10 of 12

CHECK 5: Supported Beam – Capacity at a notch

Shear and Bending interaction at the notch

For 406 x 178 x 74 UB grade S275

(a) Single line of bolts



Basic requirement: $F_v\,(t_1 + c) \le M_{cN}$

$A_{vN} = (e_t + (n-1)p + e_b)\,t_w$

$= (40 + 3 \times 70 + 113) \times 9.5 = 3448\text{mm}^2$

$P_{vN} = 0.6\,p_y\,A_{vN}$

$= \dfrac{0.6 \times 275 \times 3448}{10^3} = 569\text{kN}$

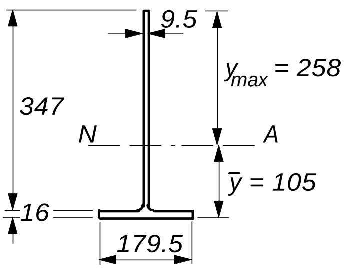
$0.75\,P_{vN} = 427\text{kN}$

$F_v = 200\text{kN} < 427\text{kN}$ $\therefore$ low shear criteria for bending applies

$\therefore M_{cN} = p_y\,Z_N$

For gross tee section:

Taking moments of area about bottom flange:



$(179.5 \times 16 \times 8) + (347 \times 9.5 \times 189.5) = \left[(179.5 \times 16) + (347 \times 9.5)\right] \times \bar{y}$

$\therefore \bar{y} = 105\text{mm}$

Second moment of area about neutral axis:

$$I_{xx} = \frac{1}{10^4}\left[\frac{179.5 \times 16^3}{12} + (179.5 \times 16 \times 97^2)\right] + \frac{1}{10^4}\left[\frac{9.5 \times 347^3}{12} + (347 \times 9.5 \times 84.5^2)\right]$$

$= 8370\text{cm}^4$

$Z_N = \dfrac{I_{xx}}{y_{max}} = \dfrac{8370}{25.8} = 324\text{ cm}^3$

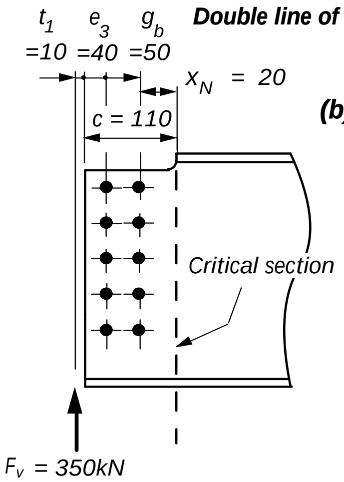
Moment capacity, $p_y Z_N = \dfrac{275 \times 324}{10^3} = 89.1\text{ kNm}$

$F_v\,(t_1 + c) = \dfrac{200\,(10 + 110)}{10^3} = 24\text{kNm}$

$\boldsymbol{F_v\,(t_1 + c) = 24\text{kNm} < 89.1\text{kNm}}$ **$\therefore$ O.K.**

For 533 x 210 x 92 UB grade S275 (c = 110mm)

Double line of bolts



$x_N = 20\text{mm} < 2d\ (40\text{mm})$

(b) Basic requirement: $\max\left(F_v\,(t_1 + c),\ F_v\,(t_1 + e_3 + g_b)\right) \le M_{cN}$

$M_{cN} = M_{cC}$ from Check 4

$M_{cN} = 164\text{kNm}$

$F_v\,(t_1 + c) = \dfrac{350\,(10 + 110)}{10^3} = 42\text{kNm}$

$F_v\,(t_1 + e_3 + g_b) = \dfrac{350\,(10+40+50)}{10^3} = 35\text{kNm}$

Eccentric moment = $\boldsymbol{F_v\,(t_1 + c) = 42\text{kNm} < 164\text{kNm}}$ **$\therefore$ O.K.**

Title Example 1 – Fin Plates - Beam to Beam	Sheet 11 of 12

CHECK 6: Supported Beam - Local Stablity of notched beam

When the beam is restrained against lateral torsional buckling no account need be taken of notch stability provided that:

For one flange notched beam in S275 steel

Basic requirements:

$$\text{Notch depth } d_{c1} \leq \frac{D}{2}$$

$$\text{and } c \leq D \quad \text{for } \frac{D}{t_w} \leq 54.3$$

$$c \leq \frac{160000D}{(D/t_w)^3} \quad \text{for } \frac{D}{t_w} > 54.3$$

For 406 x 178 x 74 UB Grade S275 *(c = 110mm d_{c1} = 50mm t_w = 9.5mm D = 412.8mm)*

$$\text{Notch depth } d_{c1} = 50mm < \frac{412.8}{2} = 206.4mm$$ ∴ O.K.

$$\frac{D}{t_w} = \frac{412.8}{9.5} = 43.5 < 54.3$$

$$c = 110mm < 412.8mm$$ ∴ O.K.

For 533 x 210 x 92 UB *(c =110 mm d_{c1} = 50mm t_w = 10.1mm D = 533.1mm)*

$$\text{Notch depth } d_{c1} = 50mm < \frac{533.1}{2} = 266.6mm$$ ∴ O.K..

$$\frac{D}{t_w} = \frac{533.1}{10.1} = 52.8 < 54.3$$

$$c = 110mm < 533.1mm$$ ∴ O.K.

CHECK 7: Not applicable

(because the supported beam is being considered as restrained against lateral torsional buckling)

Title *Example 1 – Fin Plates - Beam to Beam*	Sheet *12 of 12*

CHECK 8: Supporting Beam - Welds

Basic requirement: $s > 0.8\,t_f$

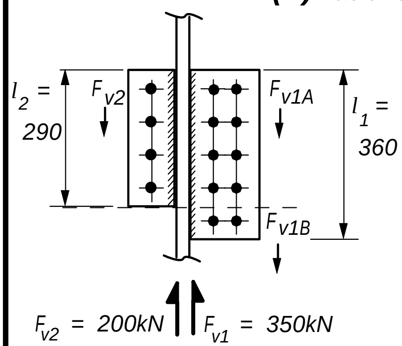
$0.8\,t_f = 0.8 \times 10 = 8mm$

∴ Use 8mm fillet welds

CHECK 9: Not applicable

CHECK 10: Supporting Beam - Local Capacity (two supported beams)

(iii) Basic requirement: $\frac{F_{v1A}}{2} + \frac{F_{v2}}{2} \leq P_v$



$F_{v1A} = \frac{F_{v1}\, l_2}{l_1} = \frac{350 \times 290}{360} = 282kN$

$F_{v1B} = 350 - 282 = 68kN$

Web thickness of supporting member, $t_w = 13.1mm$

Shear capacity, $P_v = 0.6\, P_y\, A_v$

Shear area, $A_v = 0.9 l_2\, t_w = 0.9 \times 290 \times 13.1$

$= 3419mm^2$

$P_v = 0.6 p_y A_v = \frac{0.6 \times 275 \times 3419}{10^3}$

$= 564kN$

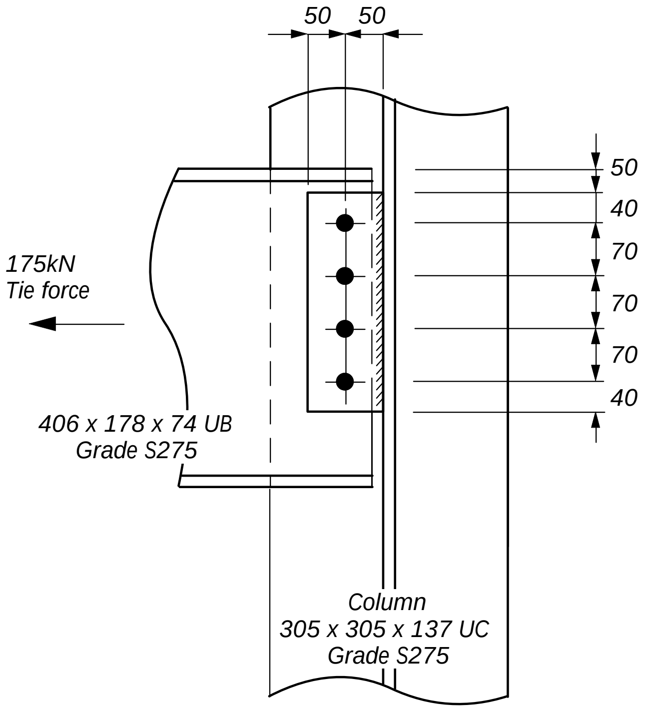
$\frac{F_{v1A}}{2} + \frac{F_{v2}}{2} = \frac{282}{2} + \frac{200}{2} = 241kN$

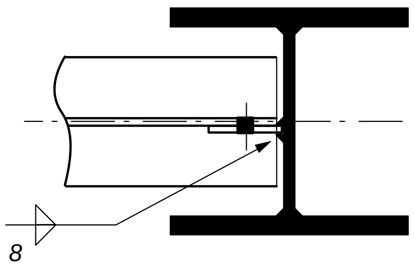
$$\frac{F_{v1A}}{2} + \frac{F_{v2}}{2} = 241kN < 564kN$$ **∴ O.K.**

CALCULATION SHEET BCSA	Job No: Joints in Steel Construction - Simple Connections		Sheet: 1 of 6
	Title: Example 2 - Fin Plates - Beam to UC column web - Structural Integrity		
	Client: SCI/BCSA Connections Group		
	Calcs by: RS	Checked by: AM	Date: May 2002

DESIGN EXAMPLE 2

Check the following beam to column connection for the tie force shown.





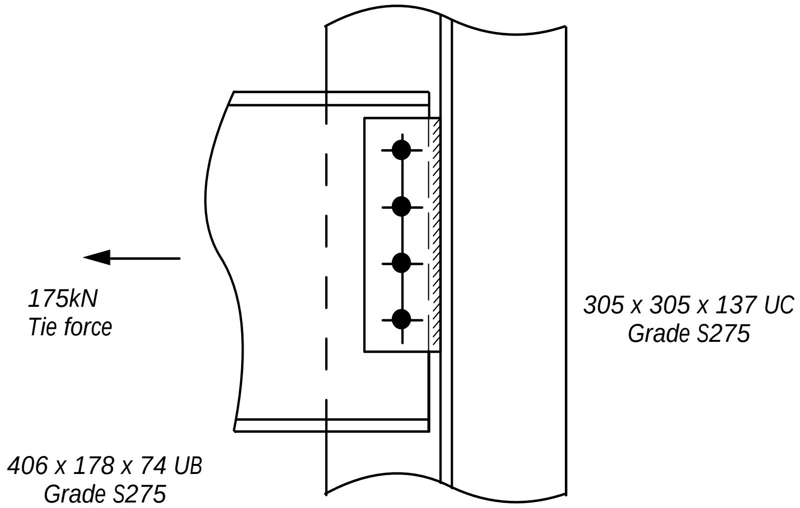
Design Information:

See figure 6.5 and Yellow pages Table H.4

Bolts:	*M20 8. 8*
Material:	*All S275*
Fin Plate:	*100 x 10 thk*
Welds:	*8mm Fillet Weld*

Title	*Example 2 - Fin Plates - Beam to UC column web – Structural Integrity*	Sheet	*2 of 6*

CONNECTION DESIGN USING CAPACITY TABLES FROM YELLOW PAGES



Yellow pages Table H.4

Fin plate type FA4 Grade S275

Bolts M20 8.8

8mm fillet weld

Yellow pages Table H.27

From capacity table H.27 in yellow pages

Connection tying capacity = 350kN > 175kN — **∴ O.K.**

∴ Beam side of connection is adequate.

Note:

(1) The tying capacity of the connection given in the table in the yellow pages is the lesser of the values obtained from CHECKS 11 & 12.

(2) Beams connecting into a column web must also be checked for column web bending as shown in CHECK 14 on sheet 6.

Title	*Example 2 - Fin Plates - Beam to UC column web – Structural Integrity*	Sheet	*3 of 6*

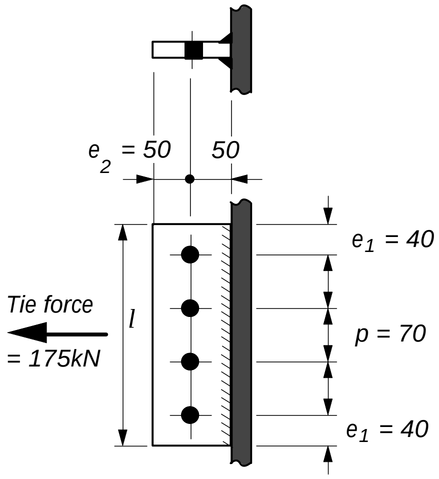
SUMMARY OF FULL DESIGN CHECKS FOR EXAMPLE 2

Note: *If the value of the tying force is greater than the vertical shear force, then the shear strength of the bolts should also be checked using the values given in Bolt capacity tables in the yellow pages.*

Sheet Nos	CHECK		406UB S275 Beam		305UC S275 Column	
			Capacity	Applied Load	Capacity	Applied Load
4	**CHECK 11** Structural Integrity - Connecting Elements Tension and bearing capacity of fin plate	Tension (kN) Bearing (kN)	667 460	175 175	Not Applicable	
5	**CHECK 12** Structural integrity - Supported Beam Tension and bearing capacity of beam web	Tension (kN) Bearing (kN)	528 350	175 175	Not Applicable	
5	**CHECK 13**	-	Not Applicable		Not Applicable	
6	**CHECK 14** Structural integrity - Tying capacity of column web	(kN)	Not Applicable		**362** **CRITICAL CHECK**	**175**

Title	Example 2 - Fin Plates - Beam to UC column web – Structural Integrity	Sheet 4 of 6

CHECK 11: Structural Integrity - Connecting Elements



Hole diameter $D_h = 22mm$

Fin plate thickness, $t_f = 10mm$

(i) For tension:

Basic requirement: **Tie force** $\leq$ **Tension capacity of fin plate**

Tension capacity of fin plate $= \min\left(p_y A\ ,\ K_e\, p_y\, A_{net}\right)$

$A = l\ t_f = 290 \times 10 = 2900mm^2$

$A_{net} = A - n\, D_h\, t_f = 2900 - 4 \times 22 \times 10 = 2020mm^2$

$p_y A = \dfrac{275 \times 2900}{10^3} = 798kN$

$K_e\, p_y\, A_{net} = \dfrac{1.2 \times 275 \times 2020}{10^3} = 667kN$

Tie Force = 175kN < 667kN $\therefore$ **O.K.**

(ii) For bearing:

Basic requirement: **Tie force** $\leq$ **Bearing capacity of fin plate**

Bearing capacity of fin plate $= 1.5n\ d\ t_f\ p_{bs}$ but $\leq 0.5n\ e_2\ t_f\, p_{bs}$ — P_{bs} from BS 5950–1 Table 32

$1.5n\ d\ t_f\ p_{bs} = \dfrac{1.5 \times 4 \times 20 \times 10 \times 460}{10^3} = 552kN$

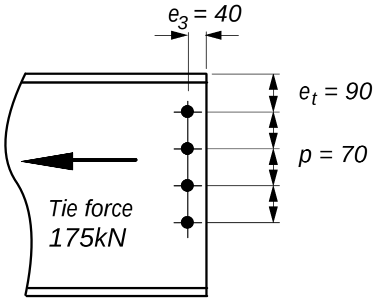
$0.5n\ e_2\ t_f\ p_{bs} = \dfrac{0.5 \times 4 \times 50 \times 10 \times 460}{10^3} = 460kN$

$\therefore$ Bearing capacity of fin plate = 460kN

Tie Force = 175kN < 460kN $\therefore$ **O.K.**

Title *Example 2 - Fin Plates - Beam to UC column web – Structural Integrity* Sheet 5 of 6

CHECK 12: Structural Integrity of supported beam



(i) For Tension:

Basic requirement: Tie force ≤ Net tension capacity of beam web

Net tension capacity of beam web = $L_e \; t_w \; p_y$

Effective net length, L_e = $2e_e + (n-1)\,p_e - n\,D_h$

e_e = e_3 (but ≤ e_t) = 40mm

p_e = p (but ≤ $2e_3$) = 70mm

D_h = hole diameter = 22mm

Beam web thickness, t_f = 9.5mm

∴ L_e = $(2 \times 40) + [(4-1)70] - (4 \times 22)$ = 202mm

∴ Net tension capacity of beam web = $\dfrac{202 \times 9.5 \times 275}{10^3}$ = 528kN

Tie Force = 175kN < 528kN **∴ O.K.**

(ii) For Bearing:

Basic requirement: Tie force ≤ Bearing capacity of beam web

Bearing capacity of beam web = $1.5n\,d\,t_w\,p_{bs}$ but ≤ $0.5n\,e_3\,t_w\,p_{bs}$

p_{bs} from BS 5950-1 Table 32

t_w = web thickness = 9.5mm

p_{bs} = 460N/mm^2

$1.5n\,d\,t_w\,p_{bs}$ = $\dfrac{1.5 \times 4 \times 20 \times 9.5 \times 460}{10^3}$ = 524kN

$0.5n\,e_3\,t_w\,p_{bs}$ = $\dfrac{0.5 \times 4 \times 40 \times 9.5 \times 460}{10^3}$ = 350kN

Bearing capacity of beam web = 350kN

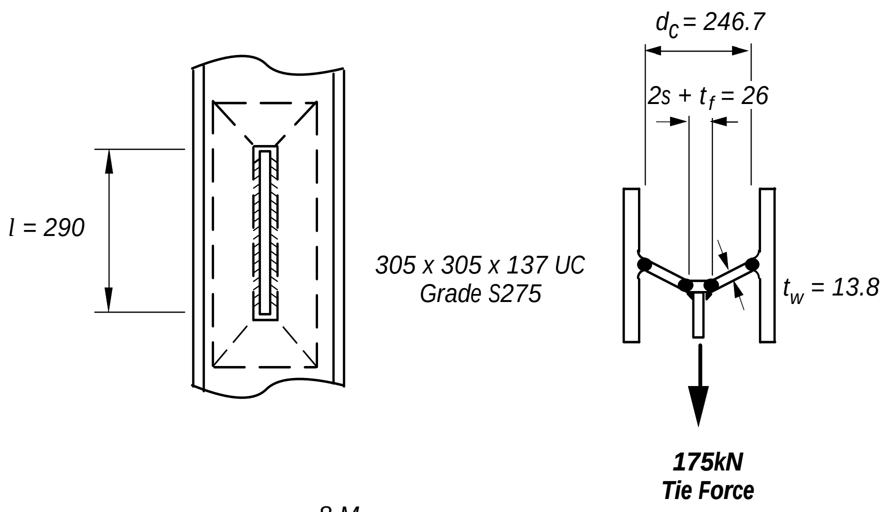
Tie Force = 175kN < 350kN **∴ O.K.**

CHECK 13: Not applicable (see Table 3.1)

Title *Example 2 - Fin Plates - Beam to UC column web – Structural Integrity*	Sheet *6 of 6*

CHECK 14: Structural Integrity - Supporting column web

Basic requirement: ***Tie force*** $\leq$ ***Tying capacity of column web***



Tying capacity of column web $= \dfrac{8\,M_u}{1-\beta_1}\left(\eta_1 + 1.5(1-\beta_1)^{0.5}\right)$

M_u = *moment capacity of column web per unit length*

$= \dfrac{p_u\, t_w^2}{4}$

$= \dfrac{328 \times 13.8^2}{4 \times 10^3} = 15.6\text{kNm/mm}$

$\eta_1 = \dfrac{l}{d_c} = \dfrac{290}{246.7} = 1.176$

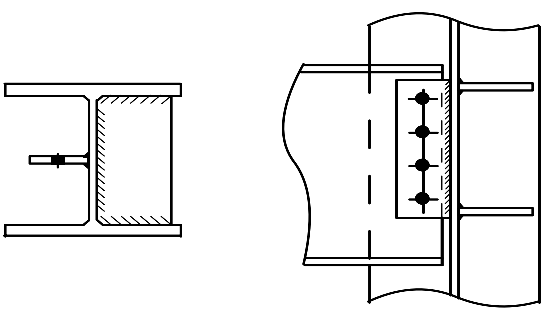
$\beta_1 = \dfrac{t_f + 2s}{d_c} = \dfrac{10 + (2 \times 8)}{246.7} = 0.105$

Tying capacity of column web $= \dfrac{8 \times 15.6}{1 - 0.105}\left(1.176 + 1.5(1 - 0.105)^{0.5}\right)$

$= 139.4\,(1.176 + 1.419) = 362\text{kN}$

Tie Force = 175kN < 362kN ***∴ O.K.***

Note: *If column web fails to satisfy the above criteria then stiffeners fillet welded on one side to the web and flanges would be required thus:*

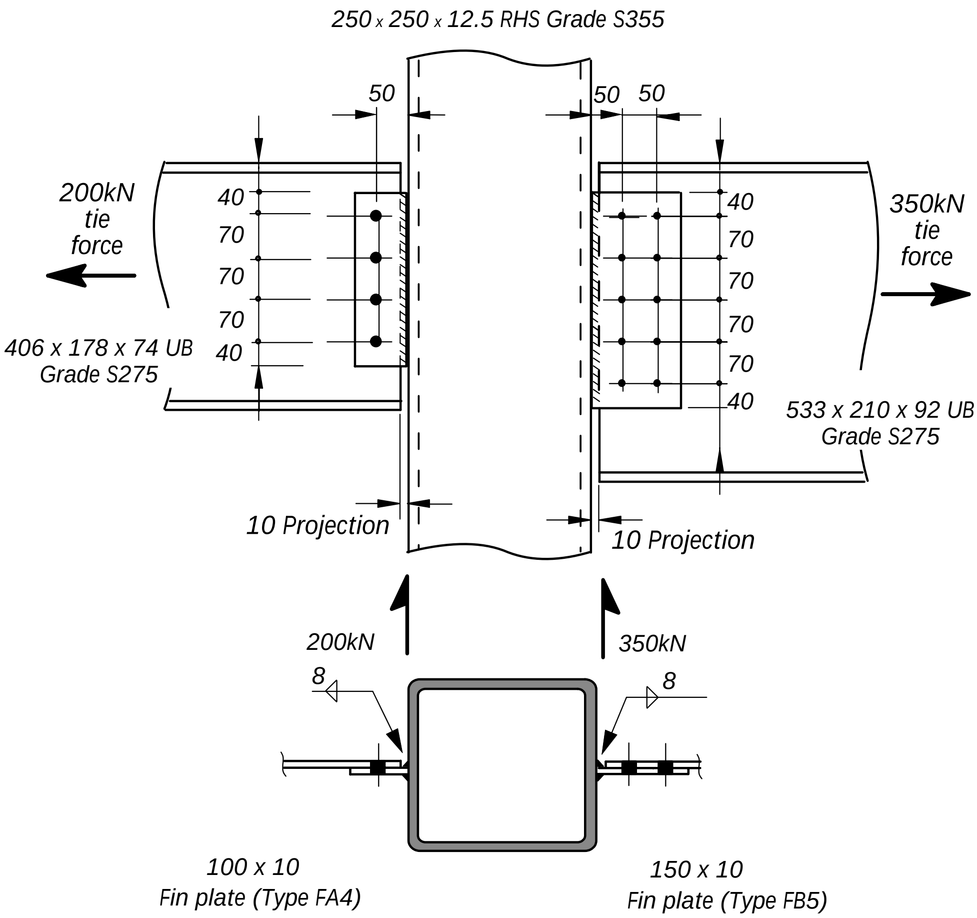


CALCULATION SHEET BCSA	Job: Joints in Steel Construction - Simple Connections		Sheet: 1 of 9
	Title: Example 3 - Fin Plates - Beam to RHS column		
	Client: SCI/BCSA Connections Group		
	Calcs by: RS	Checked by: AW / AM	Date: May 2002

DESIGN EXAMPLE 3

Check the following beam to RHS column connection for the design forces shown.
In this example it is assumed that the tying force is equal to the end reaction. However, depending on how the floor beams are arranged, the tying force given by the formula in BS 5950-1 clause 2.4.5.3 can sometimes be less.

Note: The connections should be checked independently for (i) shear forces and (ii) Tie forces and NOT for both forces at the same time.



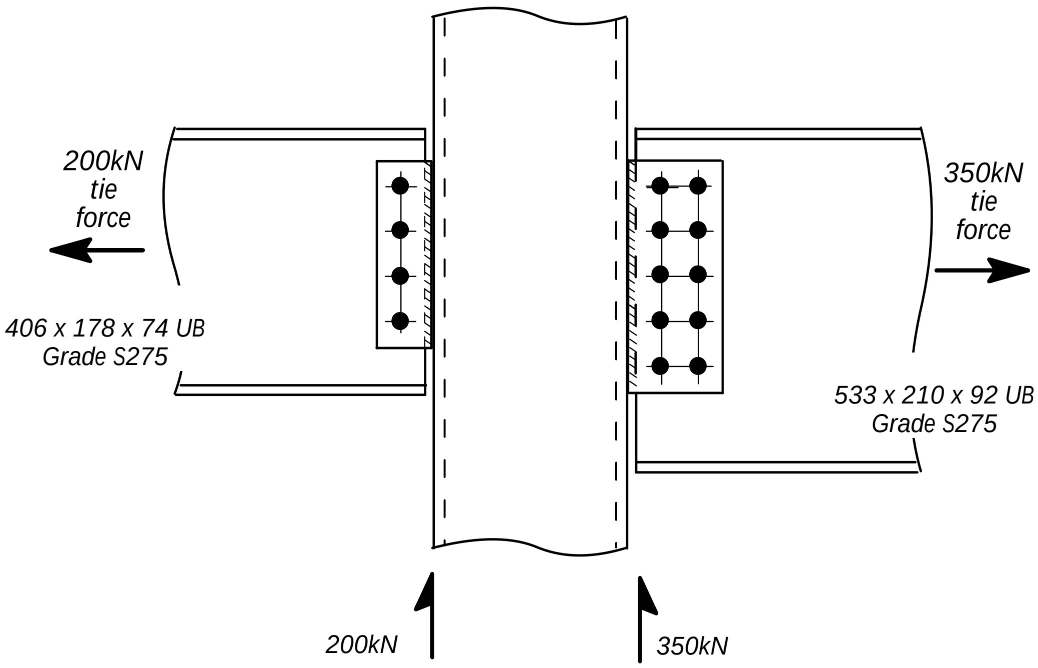
See figure 6.4 and Yellow pages Table H.4

Design Information:

Bolts:	*M20 8.8*
Welds:	*8mm fillet weld*
Column:	*S355*
Beams:	*S275*
Fin plates:	*S275*

Title	Example 3 - Fin Plates - Beam to RHS column	Sheet	2 of 9

CONNECTION DESIGN USING CAPACITY TABLES FROM YELLOW PAGES



Fin Plate type FA4 Grade S275
Bolts M20 8.8

From capacity table H.27 in Yellow pages:

Connection shear capacity
= 265kN > 200kN

Max notch length
$(c + t_1)$ = 336 > Zero

Minimum support thickness
= 4.8mm < 12.5mm

Connection tying capacity
= 350kN > 200kN

Beam side of connection is adequate

Fin Plate type FB5 Grade S275
Bolts M20 8.8

From capacity table H.28 in Yellow pages:

Connection shear capacity
476kN > 350kN

Max notch length
$(c + t_1)$ = 345 > Zero

Minimum support thickness
= 6.3mm < 12.5mm

Connection tying capacity
694kN > 350kN

Beam side of connection is adequate

Yellow pages Tables H.27 and H.28

∴ O.K.

∴ O.K.

∴ O.K.

∴ O.K.

Title	Example 3 - Fin Plates - Beam to RHS column	Sheet	3 of 9

SUMMARY OF FULL DESIGN CHECKS FOR EXAMPLE 3

Notes (i) *CHECKS 1 to 9, where applicable, are all as shown in Example 1 and are not repeated in this example, but the calculated capacities are summarised below. CHECK 4 capacities are higher because there are no notched beams in Example 3.*

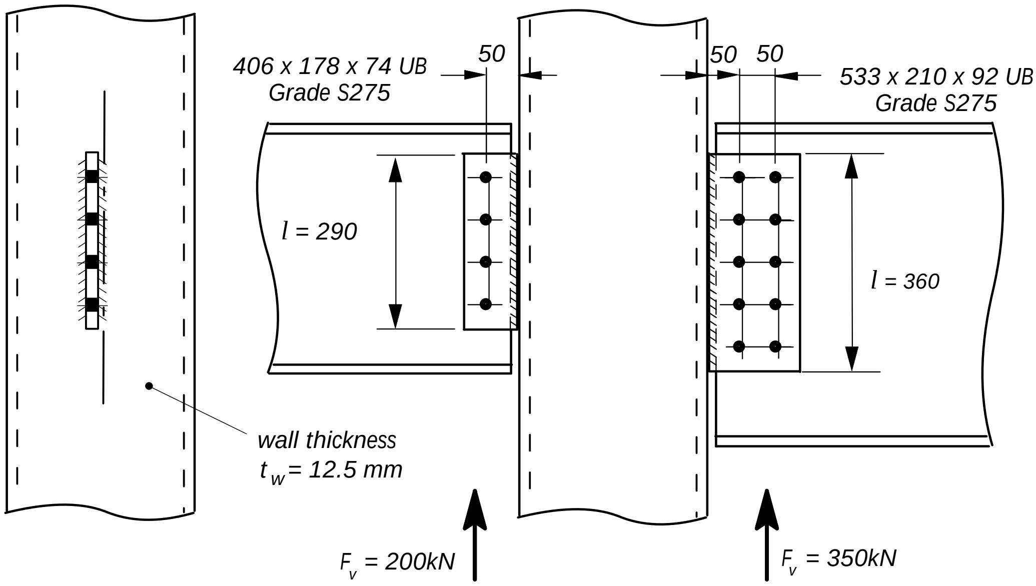
(ii) *In accordance with BS 5950-1; tie forces are ignored when checking the capacity to resist vertical reactions and vertical reactions are ignored when calculating the capacity to resist tie forces.*

(iii) *Values shown ***** are different from those given in the capacity tables (Tables H.27 and H.28) because a single notch is assumed in the tables, i.e. e_t = 40mm.*

Sheet Nos	CHECK		406UB S275		533UB S275		RHS Column S355			
							406UB Side		533UB Side	
			Capacity	Applied Load	Capacity	Applied Load	Capacity	Applied Load	Capacity	Applied Load
See Example 1 Sheets 4 to 12	**CHECK 1** - Recommended detailing practice		All recommendations adopted							
	CHECK 2 Supported Beam - Bolt Group Shear Capacity	Capacity per bolt (kN)	**87.4** **CRITICAL CHECK**	**65.9**	92	53.9	Not Applicable			
	CHECK 3 Supported Beam - Connecting Elements (Strength of fin plate)	Shear (kN) Bending capacity (kNm)	400 38.6	200 10	**494** **CRITICAL CHECK** 59.4	**350** 17.5	Not Applicable			
	CHECK 4 Supported Beam - Capacity at connection	Shear (kN)	647*	200	888*	350	Not Applicable			
	CHECKS 5, 6 and 7		Not Applicable				Not Applicable			
	CHECK 8 Supporting Column Fin plate weld	s ≥ 0.8 t_f mm	Not Applicable				($0.8t_f$) 8	(s) 8	($0.8t_f$) 8	(s) 8
	CHECK 9		Not Applicable				Not Applicable			
4	**CHECK 10** Supporting Column - Local capacity of RHS wall	Shear (kN) Punching Shear	Not Applicable				695 fin plate thickness is adequate	100	863	175
5 & 6	**CHECK 11** Structural Integrity - Connecting Elements Tension and bearing capacity of fin plates	Tension (kN) Bearing (kN)	667 460	200 200	825 1265	350 350	Not Applicable			
7 & 8	**CHECK 12** Structural Integrity - Supported beam Tension and bearing capacity of beam web	Tension (kN) Bearing (kN)	528 **350** **CRITICAL TIE FORCE CHECK**	200 **200**	850 1160	350 350	Not Applicable			
	CHECKS 13 and 14		Not Applicable				Not Applicable			
9	**CHECK 15** Structural Integrity - Supporting column wall	Tension (kN)	Not Applicable				386	200	**432** **CRITICAL TIE FORCE CHECK**	**350**

Title	Example 3 - Fin Plates - Beam to RHS column	Sheet	4 of 9

CHECK 10: Supporting Column - Local capacity



Shear and punching shear capacity of column wall

(i) Local shear capacity

Basic requirement: $F_v / 2 \leq P_v$

For 406 x 178 x 74 UB side

$F_v /2 = 100kN$

Shear capacity, $P_v = 0.6\ p_y\ A_v$

$A_v = 0.9\ l\ t_w$

$= 0.9 \times 290 \times 12.5 = 3263mm^2$

$$\therefore P_v = \frac{0.6 \times 355 \times 3263}{10^3} = 695kN$$

$\mathbf{F_v /2 = 100kN < 695kN}$ **∴ O.K.**

For 533 x 210 x 92 UB side

$F_v /2 = 175kN$

$A_v = 0.9 \times 360 \times 12.5 = 4050mm^2$

$$\therefore P_v = \frac{0.6 \times 355 \times 4050}{10^3} = 863kN$$

$\mathbf{F_v /2 = 175kN < 863kN}$ **∴ O.K.**

(ii) Punching shear

Basic requirement *(using the conservative method for both beam sides):*

U_{sc} from Table H.45 Yellow pages

$$t_f \leq t_w \times \left(\frac{U_{sc}}{p_{yf}}\right)$$

$U_{sc} = 490N/mm^2$ and $p_{yf} = 275N/mm^2$

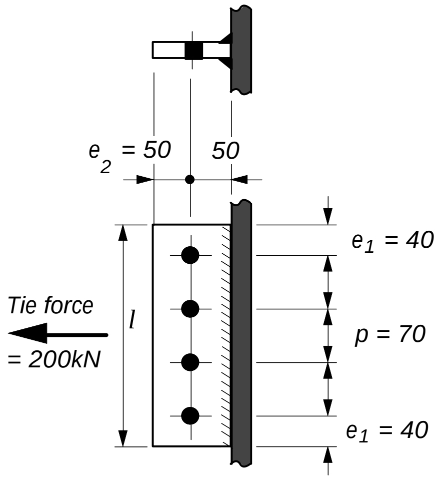
$$t_w \times \frac{U_{sc}}{p_{yf}} = 12.5 \times \frac{490}{275} = 22.3mm$$

$\mathbf{t_f = 10mm < 22.3mm}$ **∴ O.K.**

Title	Sheet
Example 3 - Fin Plates - Beam to RHS column	*5 of 9*

CHECK 11: Structural Integrity - Connecting Elements

406 x 178 x 74 UB Side



Hole diameter $D_h = 22mm$

Fin plate thickness, $t_f = 10mm$

(i) For tension:

Basic requirement: **Tie force** $\leq$ **Tension capacity of fin plate**

Tension capacity of fin plate $= \min\left(p_y A\ ,\ K_e p_y A_{net}\right)$

$A = l\ t_f = 290 \times 10 = 2900mm^2$

$A_{net} = A - n\, D_h\, t_f = 2900 - 4 \times 22 \times 10 = 2020mm^2$

$p_y A = \dfrac{275 \times 2900}{10^3} = 798kN$

$K_e p_y A_{net} = \dfrac{1.2 \times 275 \times 2020}{10^3} = 667kN$

Tie Force = 200kN < 667kN **∴ O.K.**

(ii) For bearing:

Basic requirement: **Tie force** $\leq$ **Bearing capacity of fin plate**

Bearing capacity of fin plate $= 1.5n\ d\ t_f\ p_{bs}$ but $\leq 0.5n\ e_2\ t_f\, p_{bs}$

p_{bs} from BS 5950-1 Table 32

$1.5n\ d\ t_f\ p_{bs} = \dfrac{1.5 \times 4 \times 20 \times 10 \times 460}{10^3} = 552kN$

$0.5n\ e_2\ t_f\ p_{bs} = \dfrac{0.5 \times 4 \times 50 \times 10 \times 460}{10^3} = 460kN$

∴ Bearing capacity of fin plate = 460kN

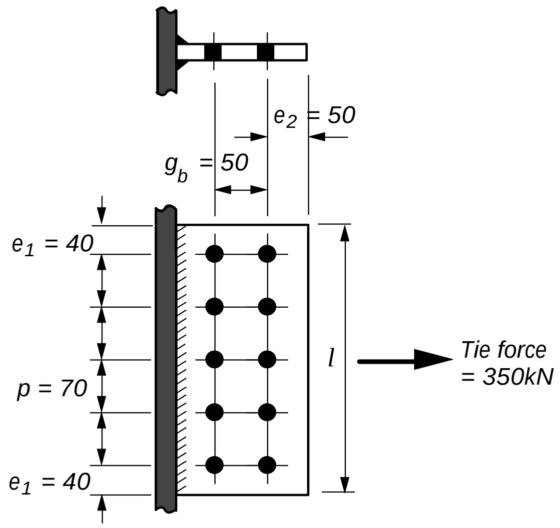
Tie Force = 200kN < 460kN **∴ O.K.**

Title	Example 3 - Fin Plates - Beam to RHS column	Sheet	6 of 9

533 x 210 x 92 UB Side

Hole diameter D_h = 22mm

Fin plate thickness, t_f = 10mm



(i) For tension:

Basic requirement: ***Tie force*** $\leq$ ***Tension capacity of fin plate***

Tension capacity of fin plate $= \min\left(p_y A,\ K_e p_y A_{net}\right)$

$A = l\ t_f = 360 \times 10 = 3600\text{mm}^2$

$A_{net} = A - n D_h t_f = 3600 - 5 \times 22 \times 10 = 2500\text{mm}^2$

$p_y A = \dfrac{275 \times 3600}{10^3} = 990\text{kN}$

$K_e p_y A_{net} = \dfrac{1.2 \times 275 \times 2500}{10^3} = 825\text{kN}$

Tie Force = 350kN < 825kN **∴ O.K.**

(ii) For bearing:

Basic requirement: ***Tie force*** $\leq$ ***Bearing capacity of fin plate***

Bearing capacity of fin plate $= 3n\,d\,t_f\,p_{bs}$ but $\leq n(1.5d\,t_f\,p_{bs} + 0.5\,e_2\,t_f\,p_{bs})$

p_{bs} from BS 5950–1 Table 32

$3n\,d\,t_f\,p_{bs} = \dfrac{3 \times 5 \times 20 \times 10 \times 460}{10^3} = 1380\text{kN}$

$n(1.5d\,t_f\,p_{bs} + 0.5\,e_2\,t_f\,p_{bs}) = 5\left(\dfrac{1.5 \times 50 \times 10 \times 460}{10^3} + \dfrac{0.5 \times 50 \times 10 \times 460}{10^3}\right)$

$= 5\,(138 + 115) = 1265\text{kN}$

∴ Bearing capacity of fin plate = 1265kN

Tie Force = 350kN < 1265kN **∴ O.K.**

Title Example 3 - Fin Plates - Beam to RHS column	Sheet 7 of 9

CHECK 12 Structural Integrity - Supported beam

Tension and bearing capacity of the beam web

For 406 x 178 x 74 UB grade S275

This check is identical to CHECK 12 in Example 2

(i) For Tension

Basic requirement: Tie force ≤ Net tension capacity of beam web

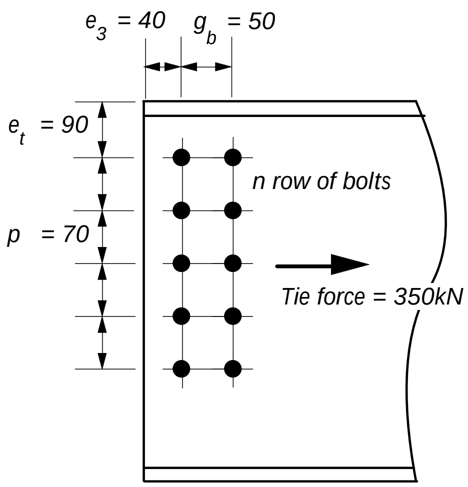
Tie force = 200kN < 528kN — **∴ O.K.**

(ii) For Bearing

Basic requirement: Tie force ≤ Bearing capacity of beam web

Tie force = 200kN < 350kN — **∴ O.K.**

For 533 x 210 x 92 UB grade S275



(i) For Tension

Basic requirement: Tie force ≤ Net tension capacity of beam web

Net tension capacity of beam web = $L_e\, t_w\, p_y$

Effective net length L_e = $2\, e_e + (n-1)p_e - n\, D_h$

e_e = $e_3 + g_b - D_h$ but $\leq$ e_t

= 68mm

p_e = p but $\leq$ $2(e_3 + g_b - D_h)$

= 70mm

L_e = $2 \times 68 + 4 \times 70 - 5 \times 22$

= 306mm

Net tension capacity = $\dfrac{306 \times 10.1 \times 275}{10^3}$ = 850kN — *See note.*

Tie Force = 350kN < 850kN — **∴ O.K**

Note: *This value may not be the same as that given in the capacity tables, where a single notched beam is assumed (i.e. e_t = 40mm)*

Title	*Example 3 - Fin Plates - Beam to RHS column*	Sheet	*8 of 9*

(ii) For Bearing

Basic requirement: ***Tie Force*** $\leq$ ***Bearing capacity of beam web***

Bearing capacity of the beam web $= 3\,n\,d\,t_w\,p_{bs}$

but $\leq n\,(1.5\,d\,t_w\,p_{bs} + 0.5\,e_3\,t_w\,p_{bs})$

$$3\,n\,d\,t_w\,p_{bs} = \frac{3 \times 5 \times 20 \times 10.1 \times 460}{10^3} = 1394kN$$

$$n\,(1.5\,d\,t_w\,p_{bs} + 0.5\,e_3\,t_w\,p_{bs}) = 5\left(\frac{1.5 \times 20 \times 10.1 \times 460}{10^3} + \frac{0.5 \times 40 \times 10.1 \times 460}{10^3}\right)$$

$$= 5\,(139 + 93) = 1160kN$$

Bearing capacity of beam web = *1160kN*

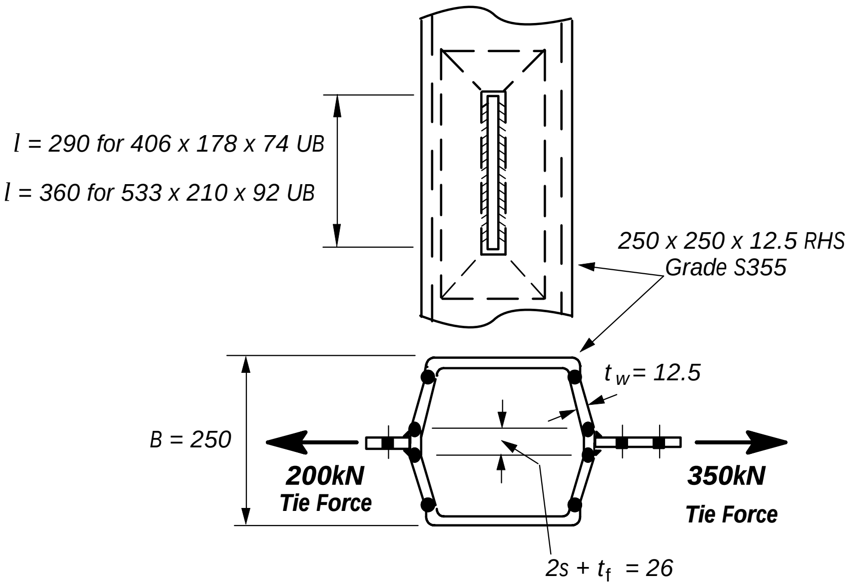
∴ Tie force = 350kN < 1160kN ***∴ O.K.***

CHECK 13: Not applicable ***see Table 3.1***

CHECK 14: Not applicable ***see Table 3.1***

Title	Sheet
Example 3 - Fin Plates - Beam to RHS column	9 of 9

CHECK 15: Structural Integrity – Capacity of Supporting Column Wall (RHS)



Basic requirement: Tie force $\leq$ Tying capacity of RHS column wall

Tying capacity of column web $= \dfrac{8\,M_u}{1-\beta}\left(\eta + 1.5(1-\beta)^{0.5}\right)$

M_u = moment capacity of RHS column wall per unit length

$= \dfrac{p_u\,t_w^2}{4} = \dfrac{392 \times 12.5^2}{4 \times 10^3} = 15.3\text{kNm/mm}$ — p_u from procedures CHECK 15

$\beta = \dfrac{t_f + 2s}{B - 3t_w} = \dfrac{10 + (2 \times 8)}{250 - (3 \times 12.5)} = 0.122$

$\eta = \dfrac{l}{B - 3t_w}$

For 406 x 178 x 74 UB side

$\eta = \dfrac{290}{250 - (3 \times 12.5)} = 1.364$

Tying capacity of column wall $= \dfrac{8 \times 15.3}{1 - 0.122}\left(1.364 + 1.5(1 - 0.122)^{0.5}\right)$

$= 139.41\,(1.364 + 1.406) = 386\text{kN}$

Tie force = 200N < 386kN **∴ O.K.**

For 533 x 210 x 92 UB side

$\eta = \dfrac{360}{250 - (3 \times 12.5)} = 1.694$

Tying capacity of column wall $= \dfrac{8 \times 15.3}{1 - 0.122}\left(1.694 + 1.5(1 - 0.122)^{0.5}\right)$

$= 139.41\,(1.694 + 1.406) = 432\text{kN}$

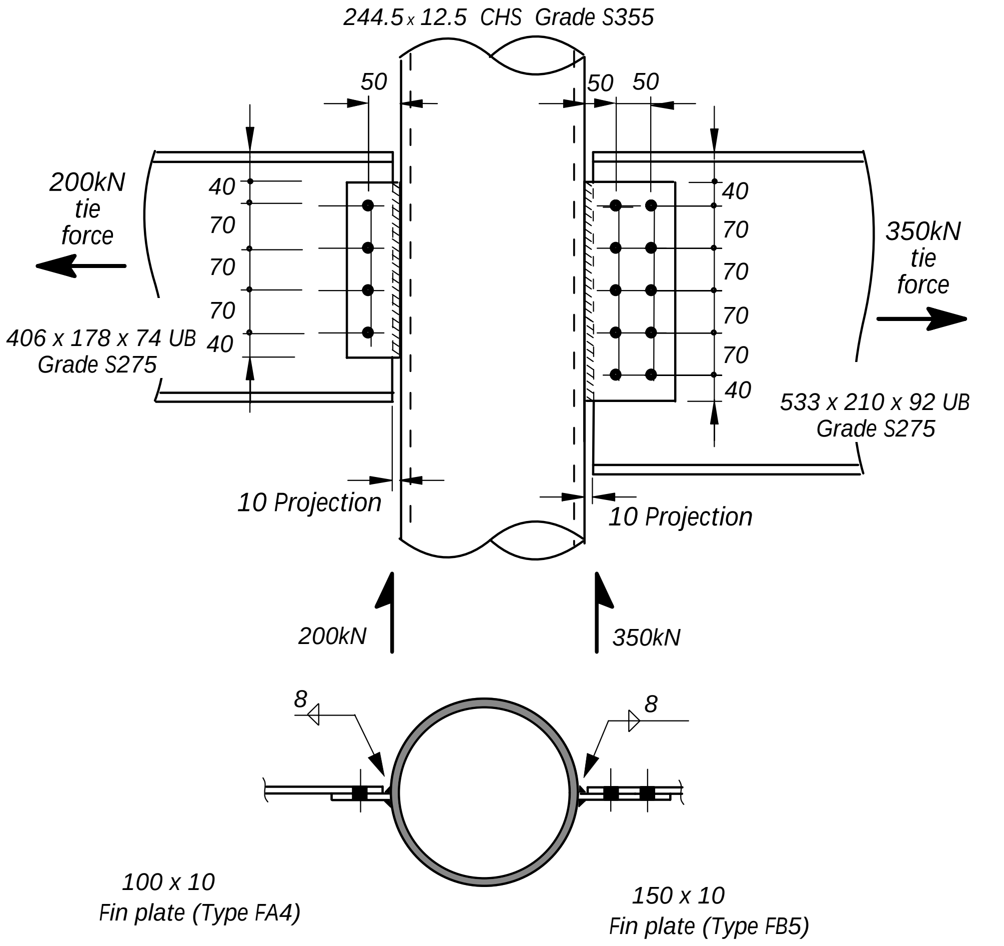
Tie force = 350kN < 432kN **∴ O.K.**

CALCULATION SHEET BCSA	Job: Joints in Steel Construction - Simple Connections		Sheet: 1 of 6
	Title: Example 4 - Fin Plates - Beam to CHS column		
	Client: SCI/BCSA Connections Group		
	Calcs by: RS	Checked by: AW / AM	Date: May 2002

DESIGN EXAMPLE 4

Check the following beam to CHS column connection for the design forces shown.
In this example it is assumed that the tying force is equal to the end reaction. However, depending on how the floor beams are arranged, the tying force given by the formula in BS 5950-1 clause 2.4.5.3 can sometimes be less.

Note: The connections should be checked independently for (i) Shear forces and (ii) Tie forces and NOT for both forces acting at the same time.



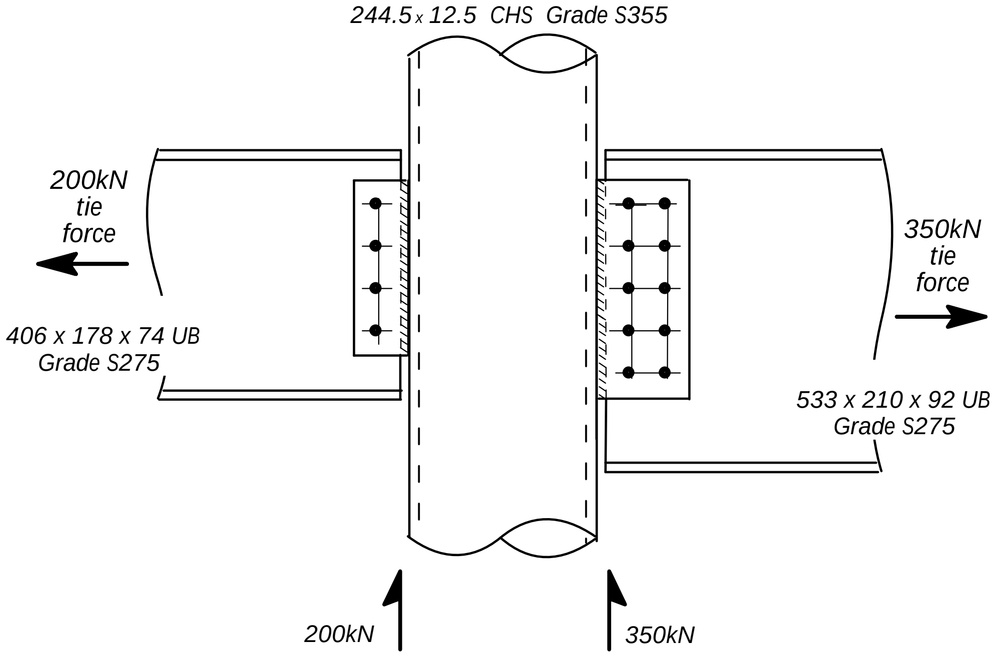
See figure 6.4 and Yellow pages Table H.4

Design Information:

Bolts:	M20 8.8
Welds:	8mm Fillet weld
Column:	S355
Beams:	S275
Fin Plates:	S275

Title	*Example 4 - Fin Plates - Beam to CHS column*	Sheet	*2 of 6*

CONNECTION DESIGN USING CAPACITY TABLES FROM YELLOW PAGES



Fin Plate type FA4 Grade S275
Bolts M20 8.8

From capacity table H.27 in yellow pages:

Connection shear capacity
= 265kN > 200kN

Max notch length
(c + t_1) = 336mm > Zero

Minimum support thickness

= 4.8mm < 12.5mm

Connection tying capacity

= 350kN > 200kN

Beam side of connection is adequate

Fin Plate type FA4 Grade S275
Bolts M20 8.8

From capacity table H.28 in yellow pages:

Connection shear capacity
= 476kN > 350kN

Max notch length
(c + t_1) = 345mm > Zero

Minimum support thickness

= 6.3mm < 12.5mm

Connection tying capacity

= 694kN > 350kN

Beam side of connection is adequate

Yellow pages Table H.27 and H.28

∴ O.K.

∴ O.K.

∴ O.K.

∴ O.K.

Title	Example 4 - Fin Plates - Beam to CHS column	Sheet	3 of 6

SUMMARY OF FULL DESIGN CHECKS FOR EXAMPLE 4

Notes (i) *Checks 1 to 9, where applicable, are all as shown in Example 1 and are not repeated in this example, but the calculated capacities are summarised below. Check 4 capacities are higher because there are no notched beams in Example 4. Similarly, Checks 11 and 12 are as shown in Example 3.*

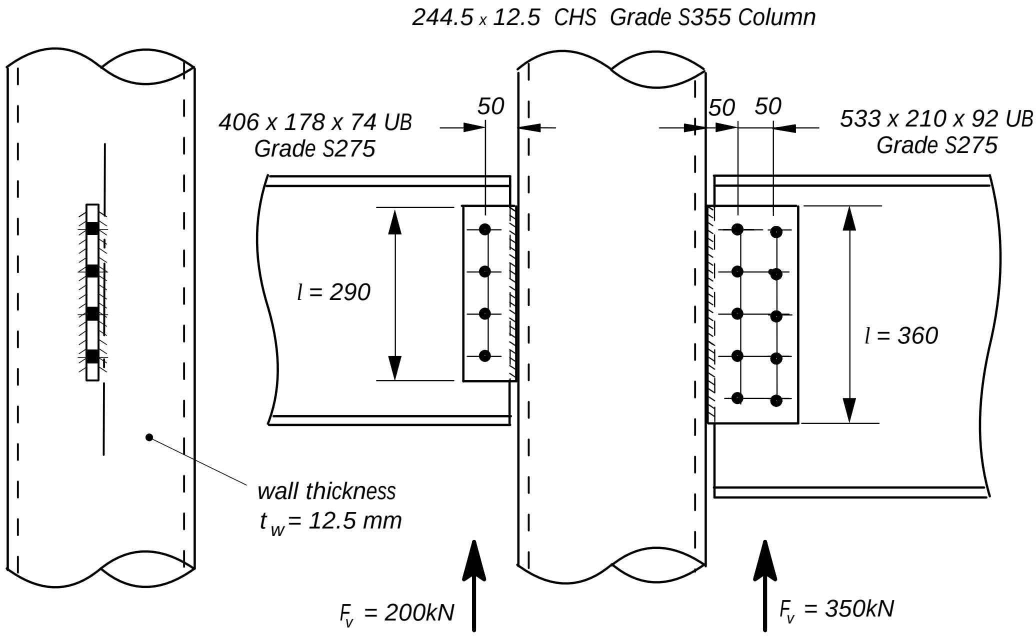
(ii) *In accordance with BS 5950-1; tie forces are ignored when checking the capacity to resist vertical reactions and vertical reactions are ignored when calculating the capacity to resist tie forces.*

(iii) *Values shown * are different from those given in the capacity tables (Tables H.27 and H.28) because a single notch is assumed in the tables, i.e. e_t = 40mm.*

Sheet Nos	CHECK		406UB S275		533UB S275		CHS Column S355			
							406UB Side		533UB Side	
			Capacity	Applied Load	Capacity	Applied Load	Capacity	Applied Load	Capacity	Applied Load
See Example 1 Sheets 4 to 11	**CHECK 1** - Recommended detailing practice		All recommendations adopted							
	CHECK 2 Supported Beam - Bolt Group Shear Capacity	Capacity per bolt (kN)	**87.4** CRITICAL CHECK	**65.9**	92	53.9	Not Applicable			
	CHECK 3 Supported Beam - Connecting Elements (Strength of fin plate)	Shear (kN)	400	200	**494** CRITICAL CHECK	**350**	Not Applicable			
		Bending capacity (kNm)	38.6	10	59.4	17.5				
	CHECK 4 Supported Beam - Capacity at connection	Shear (kN)	647*	200	888*	350	Not Applicable			
	CHECKS 5, 6 and 7		Not Applicable				Not Applicable			
	CHECK 8 Supporting Column - Fin plate weld	$s \geq 0.8\ t_f$ mm	Not Applicable				($0.8t_f$) 8	(s) 8	($0.8t_f$) 8	(s) 8
	CHECK 9		Not Applicable				Not Applicable			
4	**CHECK 10** Supporting Column - Local capacity (of CHS wall)	Shear(kN)	Not Applicable				695	100	863	175
		Punching Shear					Fin plate thickness is adequate			
5	**CHECK 11** Structural Integrity -Connecting Elements Tension and bearing capacity of fin plates	Tension (kN)	667	200	825	350	Not Applicable			
		Bearing (kN)	460	200	1265	350				
5	**CHECK 12** Structural Integrity - Supported beam Tension and bearing capacity of beam web	Tension (kN)	528	200	850	350	Not Applicable			
		Bearing (kN)	350	200	1160	350				
5	**CHECK 13, 14 and 15**		Not Applicable				Not Applicable			
6	**CHECK 16** Structural Integrity - Supporting column wall	Tension (kN)	Not Applicable				**266** CRITICAL TIE FORCE CHECK	**200**	**281** FAILS	**350**

Title	*Example 4 - Fin Plates - Beam to CHS column*	Sheet	*4 of 6*

CHECK 10 : Supporting Column - Local capacity



Shear and punching shear capacity of column wall

(i) Local shear capacity

Basic requirement: $F_v/2 \leq P_v$

For 406 x 178 x 74 UB side

$F_v/2 = 100kN$

Shear capacity, $P_v = 0.6\, p_y\, A_v$

$A_v = 0.9\, l\, t_w$

$= 0.9 \times 290 \times 12.5 = 3263mm^2$

$\therefore P_v = \dfrac{0.6 \times 355 \times 3263}{10^3} = 695kN$

$\boldsymbol{F_v/2 = 100kN < 695kN}$ **∴ O.K.**

For 533 x 210 x 92 UB side

$F_v/2 = 175kN$

$A_v = 0.9 \times 360 \times 12.5 = 4050mm^2$

$\therefore P_v = \dfrac{0.6 \times 355 \times 4050}{10^3} = 863kN$

$\boldsymbol{F_v/2 = 175kN < 863kN}$ **∴ O.K.**

(ii) Punching shear

Basic requirement *(using the conservative method for both beam sides):*

$t_f \leq t_w \times \left(\dfrac{U_{sc}}{p_{yf}}\right)$

$U_{sc} = 490N/mm^2$ and $p_{yf} = 275N/mm^2$ — *U_{sc} from Yellow pages Table H.45*

$t_w \times \dfrac{U_{sc}}{p_{yf}} = 12.5 \times \dfrac{490}{275} = 22.3mm$

$\boldsymbol{t_f = 10mm < 22.3mm}$ **∴ O.K.**

Title	Example 4 - Fin Plates - Beam to CHS column	Sheet	5 of 6

CHECK 11 Structural Integrity - Connecting Elements

(This check is identical to CHECK 11 in Example 3.)

406 x 178 x 74 UB side

(i) For Tension

Basic requirement:	**Tie force**	$\leq$	**Tension capacity of fin plate**	*Example 3 Fin Plates Sheet 5 of 9*
	200kN	$\leq$	**667kN**	**∴ O.K.**

(ii) For Bearing

Basic requirement:	**Tie force**	$\leq$	**Bearing capacity of fin plate**	*Example 3 Fin Plates Sheet 5 of 9*
	200kN	$\leq$	**460kN**	**∴ O.K.**

533 x 210 x 92 UB Grade S275

(i) For Tension

Basic requirement:	**Tie force**	$\leq$	**Tension capacity of fin plate**	*Example 3 Fin Plates Sheet 6 of 9*
	350kN	$\leq$	**825kN**	**∴ O.K.**

(ii) For Bearing

Basic requirement:	**Tie force**	$\leq$	**Bearing capacity of fin plate**	*Example 3 Fin Plates Sheet 6 of 9*
	350kN	$\leq$	**1265kN**	**∴ O.K.**

CHECK 12 Structural Integrity - Supported beam

(This check is identical to CHECK 12 in Example 3.)

406 x 178 x 74 UB Grade S275

(i) For Tension

Basic requirement:	**Tie force**	$\leq$	**Net tension capacity of beam web**	*Example 3 Fin Plates Sheet 7 of 9*
	200kN	$\leq$	**528kN**	**∴ O.K.**

(ii) For Bearing

Basic requirement:	**Tie force**	$\leq$	**Bearing capacity of beam web**	*Example 3 Fin Plates Sheet 7 of 9*
	200kN	$\leq$	**350kN**	**∴ O.K.**

533 x 210 x 92 UB Grade S275

(i) For Tension

Basic requirement:	**Tie force**	$\leq$	**Net tension capacity of beam web**	*Example 3 Fin Plates Sheet 7 of 9*
	350kN	$\leq$	**850kN**	**∴ O.K.**

(ii) For Bearing

Basic requirement:	**Tie force**	$\leq$	**Bearing capacity of beam web**	*Example 3 Fin Plates Sheet 8 of 9*
	350kN	$\leq$	**1160kN**	**∴ O.K.**

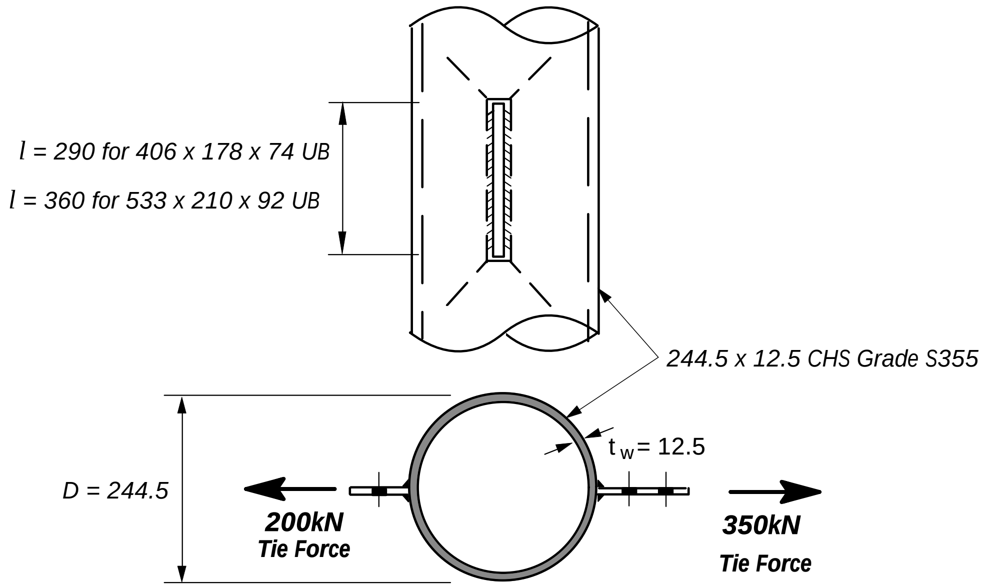
CHECK 13 Not applicable See Table 3.1

CHECK 14 Not applicable See Table 3.1

CHECK 15 Not applicable See Table 3.1

Title	Example 4 - Fin Plates - Beam to CHS column	Sheet	6 of 6

CHECK 16: Structural Integrity – Capacity of Supporting Column Wall (CHS)



Basic requirement: **Tie force** $\leq$ **Tying capacity of CHS column wall**

Tying capacity of CHS column wall $= 5\, p_u\, t_w^2\, (1 + 0.25\, \eta) \times 0.67$

p_u = design tensile strength of column wall

$$= \frac{U_s}{1.25} = 392\ N/mm^2$$

p_u from procedures CHECK 16

$$\eta = \frac{l}{D} \quad \text{but} \leq 4$$

For 406 x 178 x 74 UB side

$$\eta = \frac{290}{244.5} = 1.186$$

Tying capacity of column wall $$= \frac{5 \times 392 \times 12.5^2\,(1 + (0.25 \times 1.186)) \times 0.67}{10^3}$$

= 266kN

Tie force = 200kN < 266kN

∴ O.K.

For 533 x 210 x 92 UB side

$$\eta = \frac{360}{244.5} = 1.472$$

Tying capacity of column wall $$= \frac{5 \times 392 \times 12.5^2\,(1 + (0.25 \times 1.472)) \times 0.67}{10^3}$$

= 281kN

Tie force = 350kN $\nless$ 281kN

∴ FAILS

Use a deeper fin plate for 533 UB Side.

7. COLUMN SPLICES

7.1 INTRODUCTION

Column splices in multi-storey construction are usually provided every two or three storeys and are located just above floor level. This results in convenient lengths for fabrication, transport and erection, and gives easy access from the adjacent floor for bolting up on site. The provision of splices at each storey level is seldom economical since the saving in column material is generally far outweighed by the material, fabrication and erection costs of making the splice.

Typical bolted column splices used for rolled I-section and hollow section members are shown in Figure 7.1.

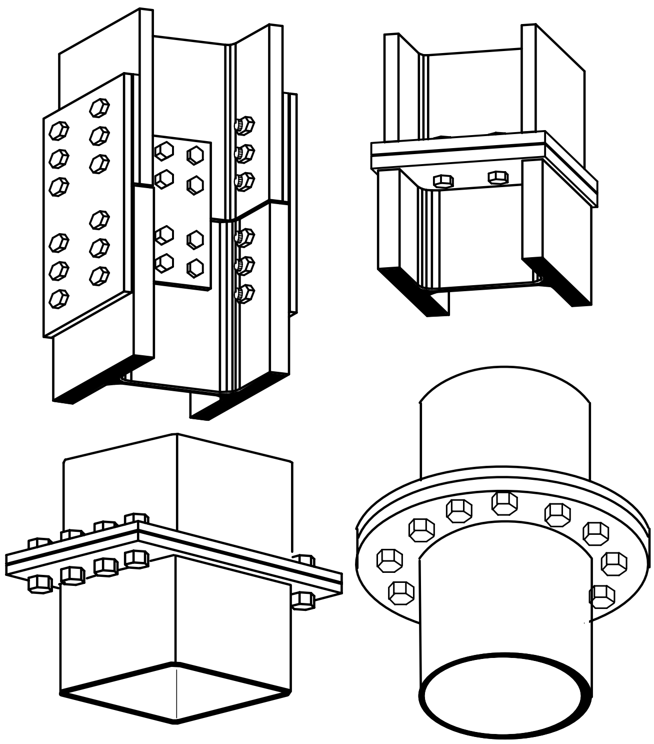


Figure 7.1 Splice connections

Bolted cover plate splices for I-sections:

There are two categories for this type of connection:

- **Bearing type** (see Figure 7.3). Here the loads are transferred in direct bearing from the upper shaft either directly or through a division plate.

  The 'bearing type' is the simpler connection, usually having far fewer bolts than the non-bearing splice, and is therefore the one most commonly used in practice.

- **Non-bearing type** (see Figure 7.4). In this case loads are transferred via the bolts and splice plates. Any bearing between the members is ignored, the connection sometimes being detailed with a physical gap between the two shafts.

  For more heavily loaded splices the end result can be expensive, involving a large amount of fabrication and site bolting

Splices are generally provided just above floor levels (typically 500mm above) hence moment due to strut action is considered insignificant. The moments induced in splices placed at other positions should be checked.

Column splices should hold the connected members in line and wherever practical the members should be arranged so that the centroidal axis of the splice material coincides with the centroidal axis of the column sections above and below the splice.

Bolted 'cap and base' or 'end plate' splices for tubular and rolled I-sections

This type of splice, consisting of 'cap and base' or end plates which are welded to the ends of the lower and upper columns and then simply bolted together on site, is commonly used in tubular construction but has also gained in popularity in recent years for rolled I-section.

The most simple form of the connection is as shown in figure 7.2 and is satisfactory as long as the ends of each shaft are prepared in the same way as for a bearing type splice and also due attention is paid to other considerations such as load reversal making the splice a tension connection, stability during erection and structural integrity. This type of connection can then be both simple and cost effective.

In a splice connection with end plates it is generally more difficult to achieve the same stiffness as the member than it is for the cover plate splices. An accurate elastic analysis of the connection should be used to verify that it is at least as stiff as the member. It is likely that relatively thick end plates will be needed. Extended end plates may be required if there is a significant moment. Even where a splice connecion is entirely in compression, it is advisable to maintain full continuity of stiffness through the connection to safeguard the robustness of the structure.

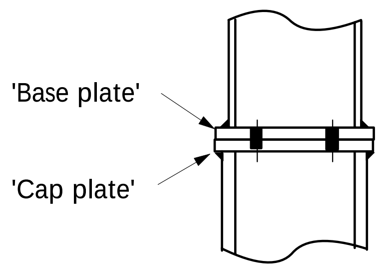


Figure 7.2 'Cap and base' or 'End plate' splice

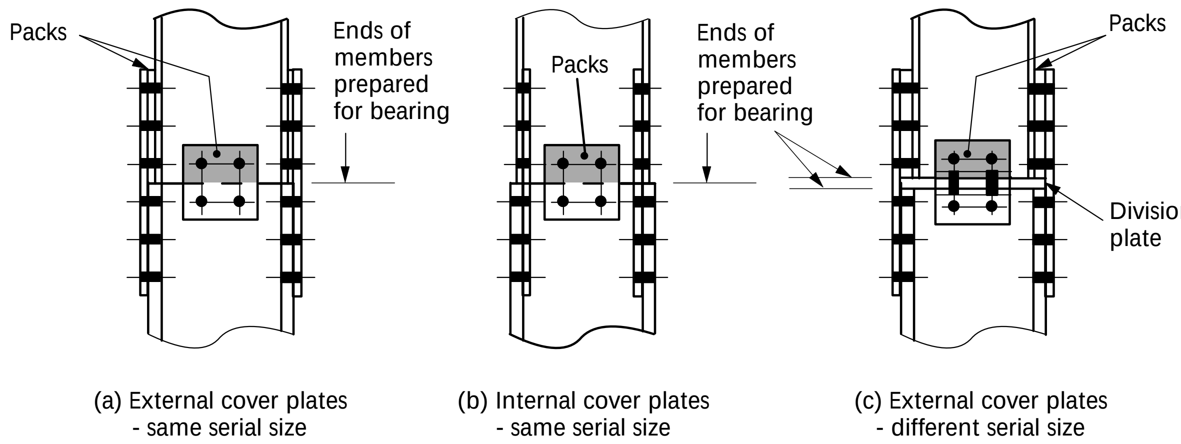


Figure 7.3 Bearing column splices for rolled I-sections

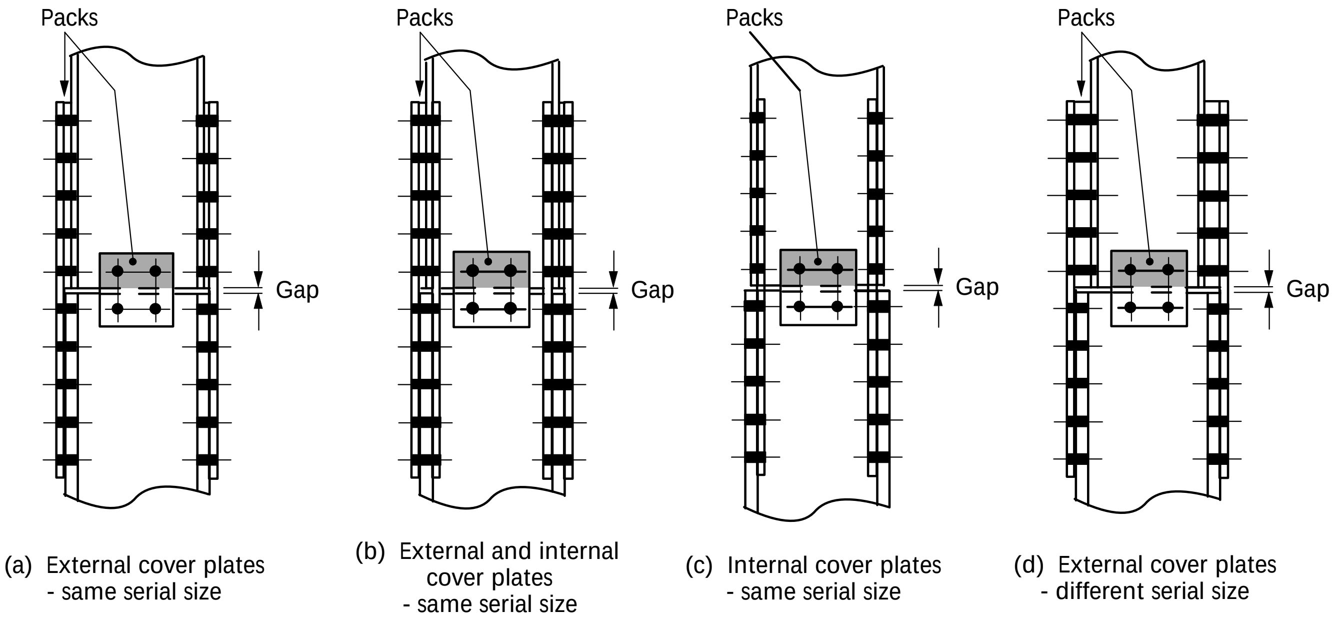


Figure 7.4 Non-bearing column splices for rolled I-sections

7.2 PRACTICAL CONSIDERATIONS

Rolled I-section column connections

The types of splices used for rolled column connections are shown in Figures 7.3 and 7.4.

The normal preparation for a rolled I-section column required to transmit compression by direct bearing is by saw cutting square to the axis of the member. A good quality saw in proper working order is adequate for this purpose.

It must be emphasised that direct bearing does not necessitate the machining or end milling of the columns and that full contact over the whole column area is not essential. Guidance on the allowable tolerances between bearing surfaces can be found in BS 5950-2[1] and the National Structural Steelwork Specification[8].

CHS and RHS tubular connections

'Cap and base' or 'end plate' splice connections are suitable for tubular members in both compression and tension. For simple designs the recommendations included here are those in the Cidect Guides[28][29].

Implicit in the design procedures is an allowance for prying forces; the 'exact method bolt tension capacity' can therefore be used.

Preferred splice types

Practical considerations lead to bearing type splices being recommended in most cases. However if constructional difficulties are encountered (e.g. the need to splice to existing steelwork) then a non-bearing splice may be the only alternative. In such cases a physical gap should be detailed between the columns, and the splice components should be designed accordingly.

In non-bearing splices joining members of different serial size, multiple packs are necessary to take up the dimensional variations. In order to limit the packing to reasonable proportions no more than one jump in the column serial size should be taken at each splice. Further rules on the packing follow in the design checks. These restrictions on packing do not apply when high strength friction grip bolts are used.

For architectural reasons, it will often be necessary to keep the width of the connection to a minimum. If this is the case, then either countersunk bolts, or narrow flange plates bolted to the inside of the flanges can be used as shown in Figure 7.3(b) and 7.4(c). By using both these options, it is possible to detail a splice that is no wider than the column section itself.

Fasteners

Generally the capacities and design strengths of fasteners and fittings should be based on the rules given in BS 5950-1, clause 6.3. The fastener spacing and edge distances should comply with the requirements of clause 6.2 of that code.

For the majority of cases 8.8, M20 or M24, bolts will be adequate for cover plate splices; the flange bolts are inserted with the heads on the outside. Countersunk bolts may be used if flush surfaces are required, modifying the checks as appropriate.

Untorqued bolts in clearance holes are normally used, except in cases where significant net tension may be present or where slip is unacceptable. Situations where joint slip may be unacceptable include splices in a braced bay subjected to large load reversals. In cases where significant net tension may be present, either general grade HSFG bolts can be used, or alternatively the connection could be detailed with 'cap and base' plates as shown in Figure 7.2. As a guide, net tension is considered significant when it exceeds 10% of the design strength (p_y) of the upper column.

Holes

Holes in fittings may be punched full size, using semi-automatic equipment, within the limits laid down In BS 5950-2[1] and in accordance the National Structural Steelwork Specification[8]. Holes through thicker fittings and column flange and web are usually drilled.

Splice fittings

Splice fittings are usually fabricated using S275 material and standard flats. Appropriate sizes may not be available for the heavier splices and in such cases the fittings will be cut from plate.

If a division plate is necessary, it will either be nominally welded to the column or bolted to the web using angle cleats. The latter option gives the opportunity for the plate to be easily removed on site if any adjustments are necessary.

Division plates will normally be sized approximately 5mm smaller than the lower column size to permit easy fitting, and will normally be flat enough for tight bearing without the need for machining or flattening.

Packing shims are usually detailed 5mm short of the bearing surfaces to avoid the possibility of slip and cutting tolerances resulting in the ends being trapped.

7.3 RECOMMENDED GEOMETRY

The main aims when detailing column splices are as follows:

- to provide a connection that is capable of carrying the design loads:
- to ensure that members are held accurately in position relative to each other;
- to provide a degree of continuity of stiffness about both axes:
- to provide sufficient rigidity to hold the upper shaft safely in position during erection[30].

Detailing requirements for splices outlined in this Section are based mainly on past experience, and these guidelines have been used to produce the standard bearing splices for rolled sections included in the yellow pages.

7.4 DESIGN

The design check procedures for column coverplate splices in rolled sections are described in detail for bearing splices in Section 7.5 and for non-bearing splices in Section 7.6.

The design check procedures for RHS and CHS bolted 'cap and base' splices, when subject to tensile forces, are described in Sections 7.7 and 7.8 respectively.

Bearing splices

The procedures require bearing splices to be initially checked to establish whether the design forces and moments induce net tension in any part of the connection. If net tension does occur then further checks must be carried out on the flange cover plates and bolts. With no net tension a standard connection such as shown in the yellow pages can be used without further checks.

It is not necessary to achieve an absolutely perfect fit over the entire bearing surface. Normally, ends of columns that are sawn cut are adequately smooth and flat for bearing without machining. The actual bearing of the ends of each column is dependent upon the accuracy of erection; after erection the ends bed down as successive dead loads are applied to the structure.

It can be assumed that full contact in bearing has been achieved, provided that the gap between the bearing surfaces, after erection, complies with clause 9.5.5 of the National Structural Steelwork Specification.[8]

Non-bearing splices

The design of a non-bearing splice is more involved, as all forces and moments must be transmitted through the bolts and splice plates. The connection must be checked both for compression and for any net tension that may occur.

Shear force

The horizontal shear force arising from the moment gradient is normally resisted by friction across the contact bearing surfaces and/or by the web cover plates. Wind forces on the external elevations of buildings are normally taken directly into the floor slabs. If instead they are taken into the steel columns it is usual to position the connections between the wall and the column such that the splice is not subject to wind shear. It is rare for column splices in simple construction be expected to transmit wind shears, and thus no design methods are presented in this guide.

Structural integrity

The Building Regulations require that certain tall multi-storey buildings must be designed so that accidental damage does not lead to disproportionate collapse. Clause 2.4.5 of BS 5950-1[1] lays down a set of rules for steel frames that are deemed to satisfy these requirements.

In such cases, the flange cover plates and associated bolt groups should be checked for a total tensile force equal to the largest factored vertical dead and imposed load reaction applied to the column at a single floor level located between that column splice and the next column splice down.

Worked examples

Five worked examples are provided in Section 7.9; three examples illustrate the design checks for rolled section cover plate splices and two design checks for hollow section splices in full tension.

Column splice tables

A fully detailed set of bearing-type coverplate splices for common combinations of UC columns is included in the yellow pages (Tables H.32 and H.33). Rationalised bolt spacing and fittings sizes have been adopted and these may be used for most splices. Tension capacities are given for bolted cover plates on each flange and these values can be used either for the net tension or for the structural integrity checks.

For the use of preloaded HSFG bolts see Table H.31 note 6.

Non-bearing splices are infrequently used for rolled columns so tables are **not** provided for this type of connection. However, Tables H.36, H.37 and H.38 are provided for bolted splices in CHS and RHS members in tension.

7.5 DESIGN PROCEDURE
COVER PLATE SPLICES FOR I- COLUMNS - BEARING TYPE

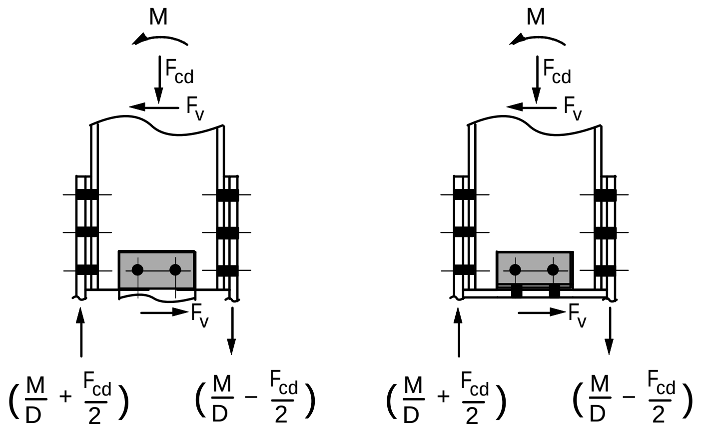
Recommended design model

When the splice is being made between columns of the same serial size, the transfer of axial forces from the upper shaft to the lower shaft can be in direct bearing because the inner profile of the lighter section will always align with the inner profile of the heavier section. The flange cover plates can be arranged to connect either to the external faces of the column (when pairs of packing plates will be required with a thickness equating to the difference in the depth dimension of the two sizes) or alternatively to the inner flanges by using split cover plates.

When the splice is being made between columns which are of different serial sizes, the transfer of axial forces from the upper shaft to the lower shaft is made through a horizontal division plate provided between the shafts to ensure an adequate load path. Flange cover plates then connect to the external faces of the column shafts.

In normal circumstances the horizontal shear force arising from the moment gradient is resisted by friction across the contact bearing surfaces and/or by the web cover plate, but no specific checks are presented here. See further advice in Section 7.4.

Design procedures are summarised below. CHECK 1 gives the size parameters for the splice, CHECKS 2 to 4 are strength checks required when tension is present and CHECK 5 is required for structural integrity design.

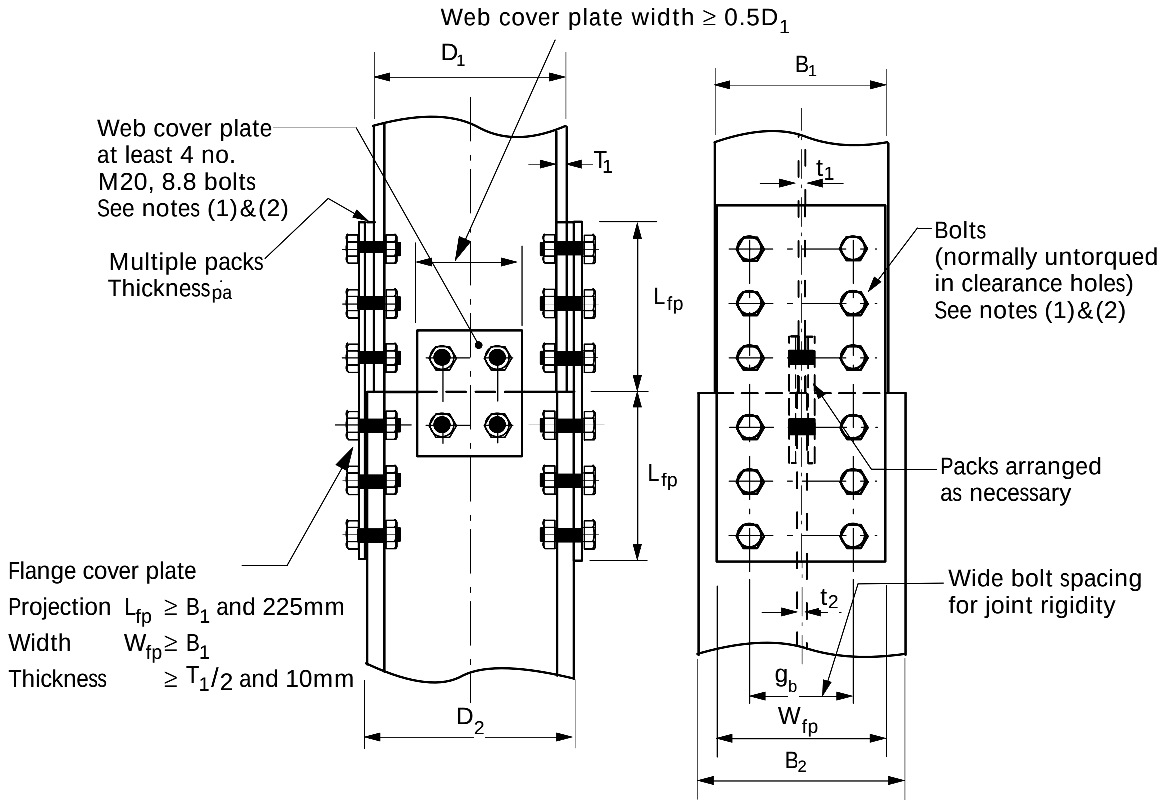


where

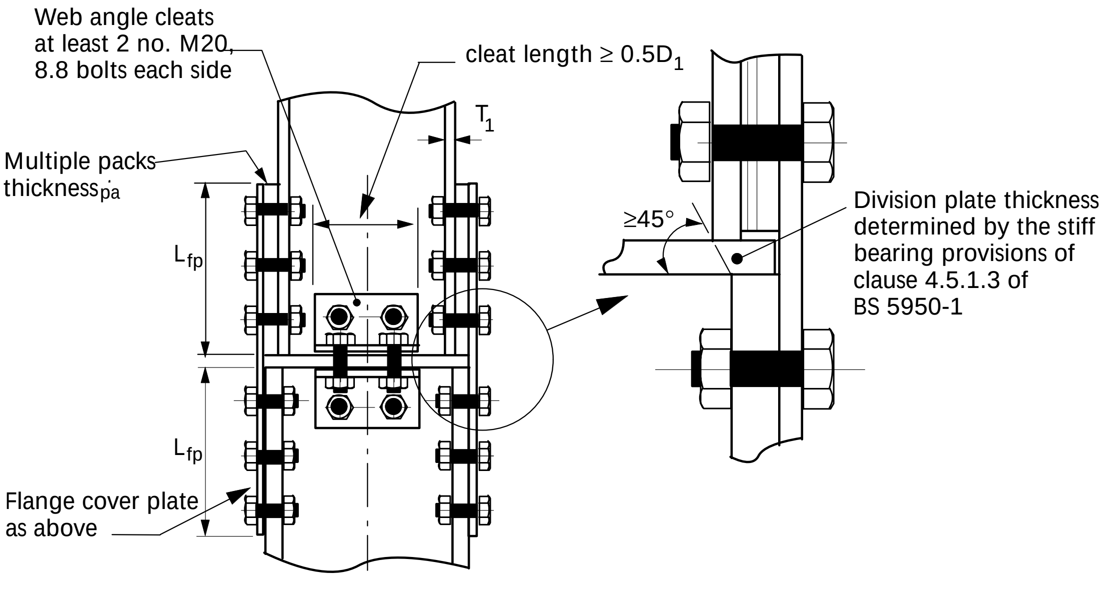
M	=	nominal moment due to factored dead and imposed loads (ie. column design moment) at the floor level immediately below the splice.
F_{cd}	=	axial compressive force due to factored dead load only.
F_v	=	shear force.
D	=	conservatively, the overall depth of the smaller column (for external flange cover plates) or the centreline to centreline distance between the flange cover plates (for internal flange cover plates).

CHECK 1	**Recommended detailing practice**		
CHECK 2	**Flange cover plates**	–	Determine whether net tension is present
CHECK 3	**Flange cover plates**	–	Tensile capacity of cover plates
CHECK 4	**Flange cover plates**	–	Bolt group
CHECK 5	**Structural integrity**	–	Cover plates and bolt group

CHECK 1	Recommended detailing practice Columns (UC or UB) – Bearing splice External flange cover plates	H



Butting surfaces of sections in direct bearing



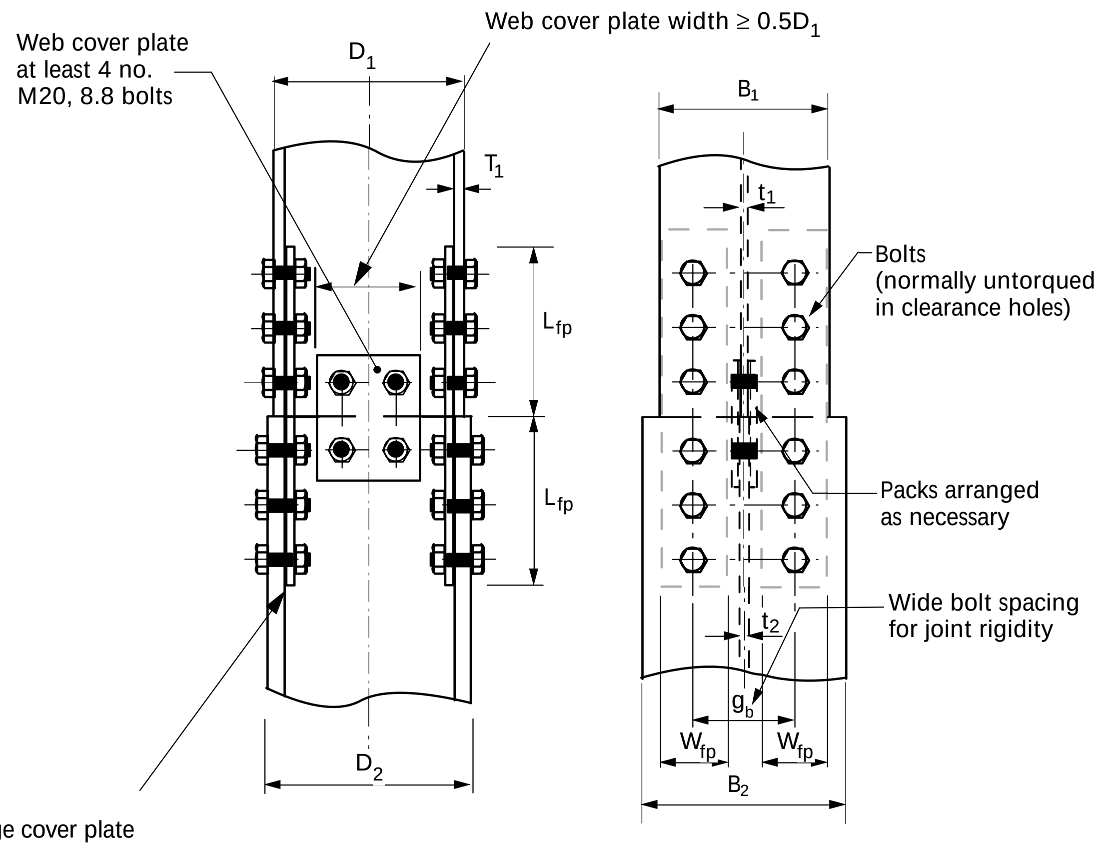
Direct bearing onto a division plate

Notes:

(1) Bolt spacing and edge distances should comply with the recommendations of BS 5950-1:2000.

(2) If there is significant net tension (see CHECK 3) then the notes from CHECK 1 for Non-bearing splices should be adhered to.

CHECK 1 (continued)	Recommended detailing practice Columns (UC or UB) – Bearing splice Internal flange cover plates	H



Flange cover plate

Projection $L_{fp} \geq B_1$ and 225mm

Width $W_{fp} \geq (B_1 - t_2 - 2r_2)/2$ (r_2 = root radius)

Thickness $\geq T_1/2$ and 10mm

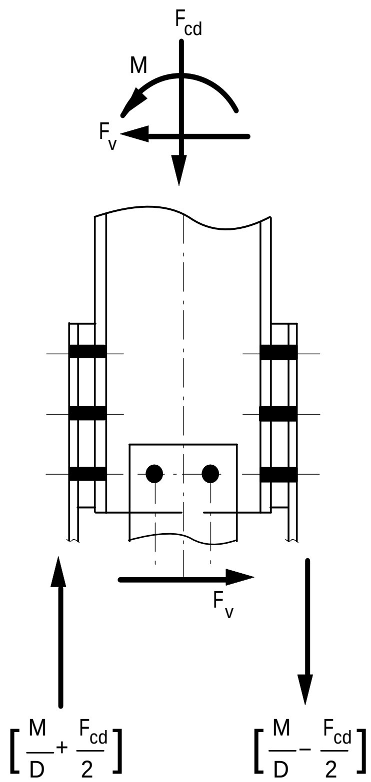
Internal flange cover plates

Notes:

(1) Bolt spacing and edge distances should comply with the recommendations of BS 5950-1:2000.

(2) If there is significant net tension (see CHECK 3) then the notes from CHECK 1 for Non-bearing splices should be adhered to.

CHECK 2	Flange cover plates – Presence of net tension due to axial load and bending moment	H



Basic requirement:

If $\quad M \leq \dfrac{F_{cd}\, D}{2}$

Net tension does not occur and so the splice need only be detailed to transmit axial compression by direct bearing (Check 1)

If $\quad M > \dfrac{F_{cd}\, D}{2}$

Net tension does occur and the flange cover plates and their fasteners should be checked for a tensile force, F_2 (Check 3 and 4)

Preloaded HSFG bolts should be used when net tension induces stress in the upper column flange > 10% of the design strength of that column.

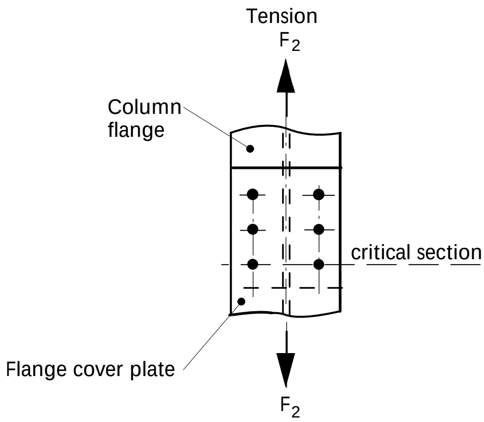
$$F_2 = \frac{M}{D} - \frac{F_{cd}}{2}$$

where: M = nominal moment due to factored dead and imposed loads (i.e. column design moment) at the floor level immediately below the splice.

F_{cd} = axial compressive force due to factored dead load only.

D = conservatively, the overall depth of the smaller column (for external flange cover plates) or the centreline to centreline distance between the flange cover plates (for internal flange cover plates).

CHECK 3	Flange cover plate Tensile capacity of cover plate	H



Basic requirement:

$$F_2 \leq \min\left(p_y A_{fp},\ K_e p_y A_{fp.net}\right)$$

where

F_2 is as given by CHECK 2

A_{fp} = gross area of flange cover plate(s) attached to one flange

$A_{fp.net}$ = net area of flange cover plate(s) attached to one flange

K_e = 1.2 for S275 steel , = 1.1 for S355 steel {See BS 5950-1 clause 3.4.3}

Check for significant net tension:

If $\dfrac{F_2}{T_1 B_1 p_{y1}}$ > 0.1 then preloaded HSFG bolts should be used

where:

T_1 = Upper column flange thickness

B_1 = Upper column flange width

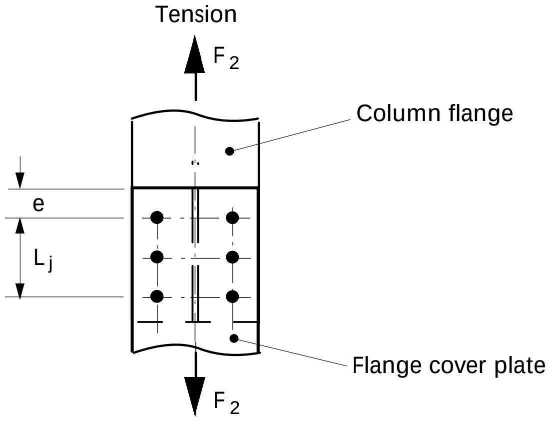
p_{y1} = Upper column design strength

Note:

(1 It is sufficiently accurate to base this calculation on the gross area of the flange.

(2) When the tension is due to structural integrity requirements it is not necessary to use preloaded HSFG bolts.

CHECK 4	Flange cover plate -bolt group	H



Bolts in shear

Basic requirement:

$F_2 \leq$ Reduction factor x ΣP_s

P_s = shear capacity of single bolt = $p_s A_s$

(But for the top pair of bolts, P_s is the smaller of $p_S A_S$ or 0.5 e t_{fp} p_{bs})

where:

$$\text{Reduction factor} = \min\left(\frac{9\,d}{8d + 3t_{pa}}\ ,\ \left(\frac{5500 - L_j}{5000}\right)\ ,\ 1\right)$$

L_j = joint length (in mm)

p_s = shear strength of the bolt

A_s = shear area of the bolt

t_{fp} = thickness of the flange cover plate

p_{bs} = bearing strength of the flange cover plate

e = end distance

d = bolt diameter

t_{pa} = thickness of the packing

Bolts in bearing

Basic requirement:

$F_2 \leq \Sigma P_{bs}$

P_{bs} = bearing capacity of the flange cover plate

(But for the top pair of bolts, P_{bs} is the smaller of dt_{fp} p_{bs} or 0.5 e t_{fp} p_{bs})

Note: If the thickness of the column flange is less than the thickness of the flange cover plate, then bearing capacity of the column flange should also be checked.

For Preloaded HSFG bolts

$F_2 \leq \Sigma P_{sL}$

For connection designed to be non-slip under factored loads

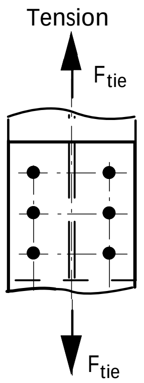
P_{sL} = slip resistance = $0.9\, K_s\, \mu\, P_o$ (BS 5950-1, cl. 6.4.2)

K_s = 1.0 for fasteners in standard clearance holes

μ = slip factor

P_o = minimum shank tension as specified in BS 4604[31]

CHECK 5	Structural integrity of splice	H



If it is necessary to comply with structural integrity requirements, then checks 3 and 4 should be carried out with: $F_2 = F_{tie} / 2$

based on the conservative assumption that the tie force is resisted by the two flange cover plates.

F_{tie} is the tensile force obtained from BS 5950-1, clause 2.4.5.3(c).

7.6 DESIGN PROCEDURE
COVER PLATE SPLICES FOR I- COLUMNS - NON-BEARING TYPE

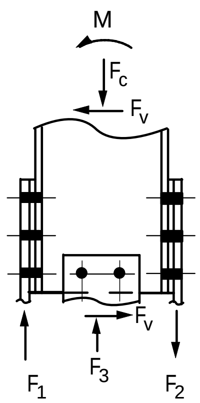
Recommended design model

The transfer of all forces, including the axial force, is made from the upper shaft to the lower shaft via the bolted splice cover plates.

The flange cover plates can be arranged to connect either to the external faces of the column (when pairs of packing plates will be required with a thickness equating to the difference in the depth dimension of the two sizes) or alternatively to the inner flanges by using split cover plates.

In normal circumstances the horizontal shear force arising from the moment gradient is resisted by the web cover plate, but no specific checks are presented here. See further advice in Section 7.4.

Design procedures are summarised below. CHECK 1 gives the size parameters for the splice, CHECKS 2 to 5 are strength checks and CHECK 6 is required for structural integrity design.

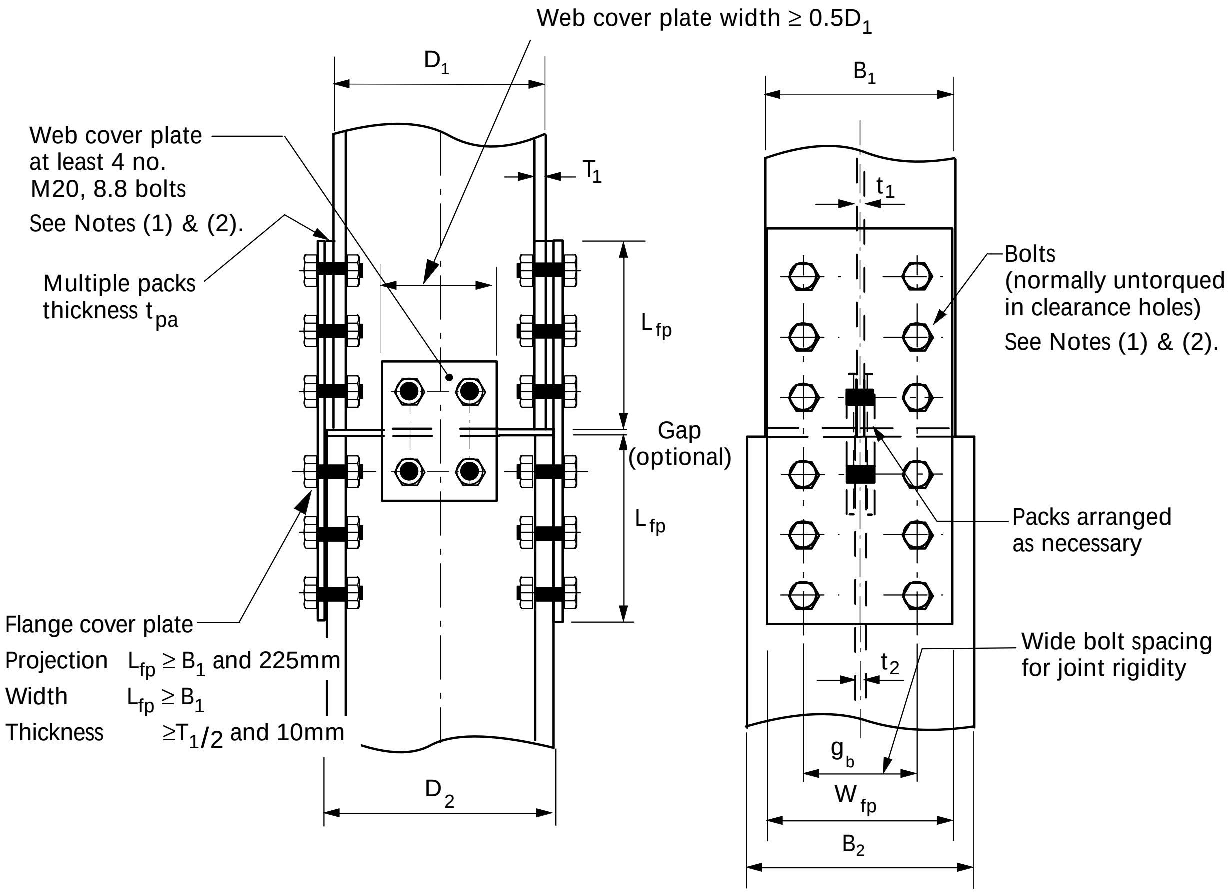


where

M	=	nominal moment due to factored dead and imposed loads (ie. column design moment) at the floor level immediately below the splice.
F_c	=	axial compressive force due to factored dead and imposed loads
F_v	=	shear force.
D	=	conservatively, the overall depth of the smaller column (for external flange cover plates) or the centreline to centreline distance between the flange cover plates (for internal flange cover plates).
F_1	=	maximum compressive force per flange
F_2	=	maximum tensile force per flange
F_3	=	compressive force in web

CHECK 1	**Recommended detailing practice**		
CHECK 2	**Flange cover plates**	–	Compression and tension capacity
CHECK 3	**Flange cover plates**	–	Bolt group
CHECK 4	**Web cover plates**	–	Compression capacity
CHECK 5	**Web cover plates**	–	Bolt group
CHECK 6	**Structural integrity**	–	Cover plates and bolt group

CHECK 1	Recommended detailing practice Columns (UC or UB) – Non-bearing Splice External flange cover plates	H



Axial compression developed in external flange cover plates

Notes:

(1) Preloaded HSFG bolts are required if slip is unacceptable.

(2) Although the minimum width and length for cover plates are similar to those for bearing splices the thickness and bolt design will usually result in larger cover plates.

(3) Bolt diameters must at least 75% of packing thickness t_{pa}.[32]

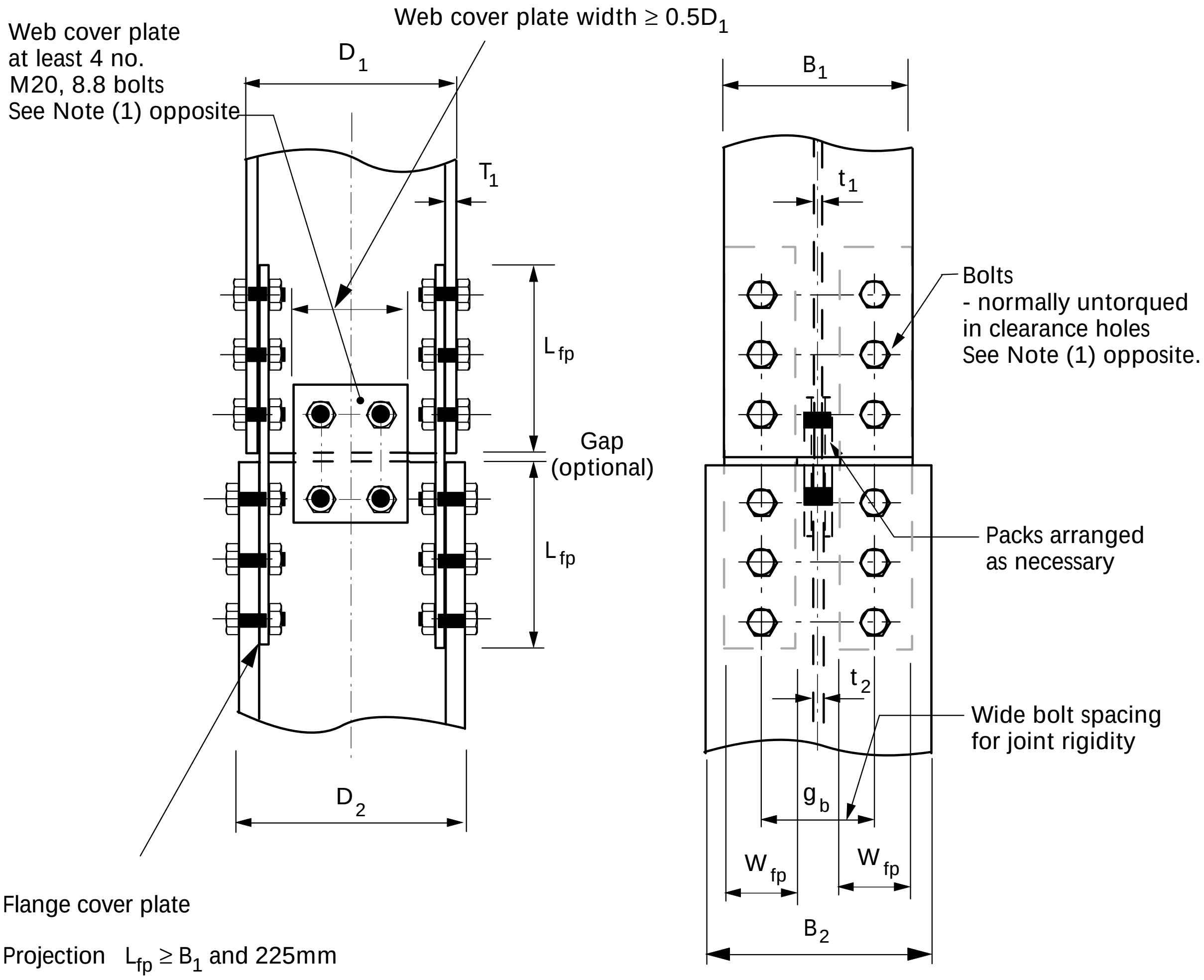
(4) The number of plies in multiple packs should not exceed four.[32]

(5) There should not be more than one jump in serial size of column at the splice.

(6) If external and internal flange covers are to be provided the sizes should be similar to those shown and the combined thickness of the external and internal cover plates must be at least $T_1/2$. All cover plates should be at least 10mm thick.

(7) Bolt spacing and edge distances should comply with the recommendations of BS 5950-1:2000

CHECK 1 (continued)	Recommended detailing practice Columns (UC or UB) – Non-bearing splice Internal flange cover plates	H



Flange cover plate

Projection $L_{fp} \geq B_1$ and 225mm

Width $W_{fp} \geq (B_1 - t_2 - 2r_2)/2$

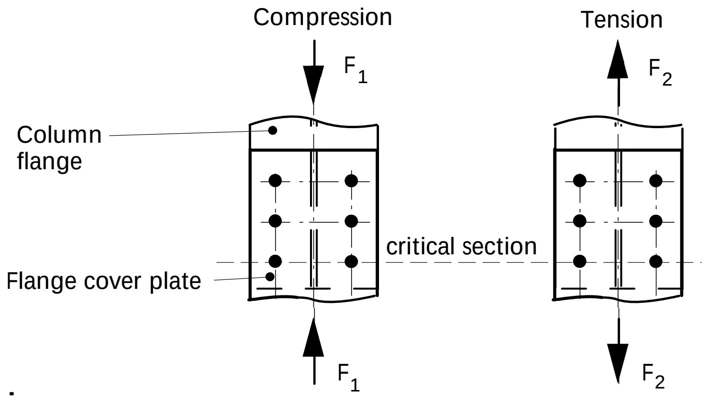
Thickness $\geq T_1/2$ and 10mm

Axial compression developed in internal flange cover plates

Notes:

The notes (1) and (3) to (7) opposite apply.

CHECK 2	Flange cover plates Capacity	H



For compression

Basic requirement:

$F_1 \leq p_y A_{fp}$

where:

$F_1 = \frac{M}{D} + F_c \left(\frac{A_f}{A} \right)$ (conservative)

p_y = Design strength of cover plate

A_{fp} = gross area of flange cover plate(s) attached to one flange

M = nominal moment due to factored dead and imposed loads (i.e. column design moment) at the floor level immediately below the splice

F_c = axial compressive force due to factored dead and imposed loads

D = conservatively, the overall depth of the smaller column (for external flange cover plates) or the centreline to centreline distance between the flange cover plates (for internal flange cover plates)

A = total area of the smaller column

A_f = area of one flange of the smaller column = $B_1 \times T_1$

B_1 = width of flange of the smaller column

T_1 = flange thickness of the smaller column

For tension

Basic requirement:

$F_2 \leq \min \left(p_y A_{fp} , K_e p_y A_{fp.net} \right)$

where:

$F_2 = \frac{M}{D} - F_{cd} \left(\frac{A_f}{A} \right)$ (conservative)

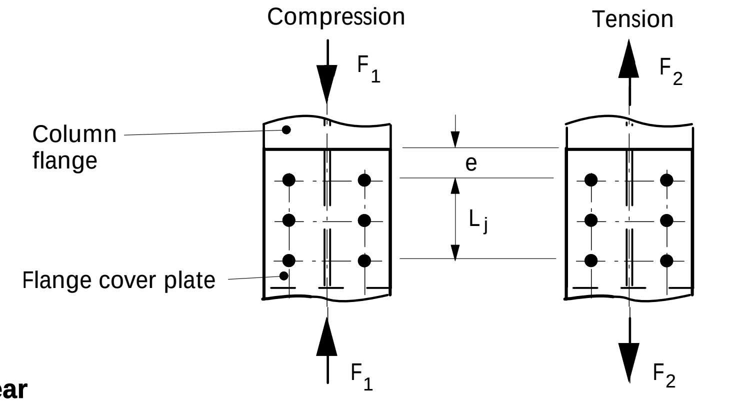
F_{cd} = axial compression force due to factored dead load only

$A_{fp.net}$ = net area of flange cover plate(s) attached to one flange

K_e = 1.2 for S275 steel ,
= 1.1 for S355 steel

{ See BS 5950-1 clause 3.4.3 }

CHECK 3	Flange cover plates Bolt Group	H



Bolts in shear

Basic requirement:

Max.(F_1,F_2) $\leq$ Reduction factor x ΣP_s

P_s = shear capacity of single bolt = $p_s A_s$

(But for the top pair of bolts, P_s is the smaller of $p_S A_S$ and $0.5\ e\ t_{fp}\ p_{bs}$)

where:

$$\text{Reduction factor} = \min\left(\frac{9d}{8d + 3t_{pa}},\ \left(\frac{5500 - L_j}{5000}\right),\ 1\right)$$

L_j = joint length (in mm)

p_s = shear strength of the bolt

A_s = shear area of the bolt

t_{fp} = thickness of the flange cover plate

p_{bs} = bearing strength of the flange cover plate

e = end distance

d = bolt diameter

t_{pa} = thickness of the packing

Bolts in bearing

Basic requirement:

Max.(F_1,F_2) $\leq$ ΣP_{bs}

P_{bs} = bearing capacity of the flange cover plate

(But for the top pair of bolts, P_{bs} is the smaller of $dt_{fp}\ p_{bs}$ and $0.5\ e\ t_{fp}\ p_{bs}$)

Note: If the thickness of the column flange is less than the thickness of the flange cover plate, then bearing capacity of the column flange should also be checked.

For Preloaded HSFG bolts

Max.(F_1,F_2) $\leq$ ΣP_{sL}

For connection designed to be non-slip under factored loads

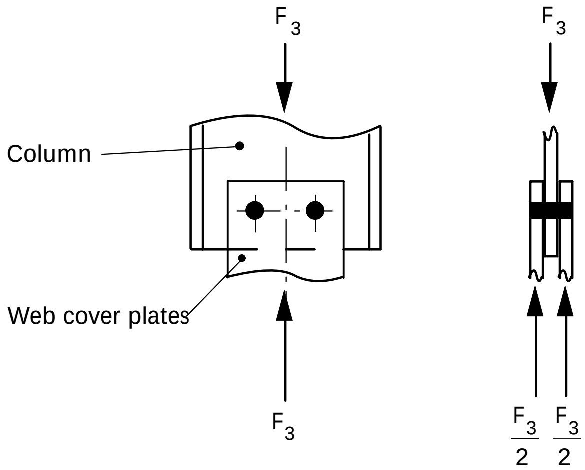
P_{sL} = slip resistance = $0.9\ K_s\ \mu\ P_o$ (BS 5950-1, cl. 6.4.2)

K_s = 1.0 for fasteners in standard clearance holes

μ = slip factor

P_o = minimum shank tension as specified in BS 4604[31]

CHECK 4	Web cover plates Capacity	H



Capacity of web cover plates

Basic requirement:

$$\frac{F_3}{2} \leq p_y A_{wp}$$

conservatively $F_3 = \dfrac{F_c \; A_w}{A}$

where:

F_c = axial compressive force due to factored dead and imposed loads

A = total area of the smaller column

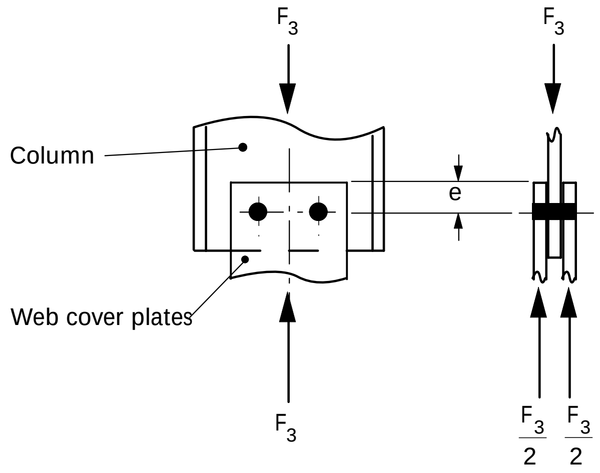
A_w = area of web of the smaller column = $(A - 2A_f)$

A_f = area of one flange of the smaller column

p_y = Design strength of web cover plates

A_{wp} = gross area of one web cover plate

CHECK 5	Web cover plates Bolt group	H



Bolts in shear

Basic requirement:

$$\frac{F_3}{2} \leq \sum P_s$$

where: F_3 = As per Check 4

P_s = shear capacity of single bolt = $p_s\ A_s$
(But for the top pair of bolts, P_s is the smaller of $p_s\ A_s$ or $0.5\ e\ t_{wp}\ p_{bs}$)

p_s = shear strength of the bolt

A_s = shear area of the bolt

t_{wp} = thickness of the web cover plate

p_{bs} = bearing strength of the web cover plate

e = end distance

Bolts in bearing

$$F_3 \leq \sum P_{bs}$$

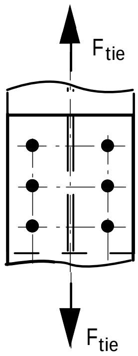
where $\sum P_{bs}$ = bearing capacity of the web cover plate

P_{bs} = bearing capacity of the web cover plate per bolt = $d\ t\ p_{bs}$
(But for the top pair of bolts, P_{bs} is the smaller of $d\ t\ p_{bs}$ or $0.5\ e\ t_{wp}\ p_{bs}$)

d = bolt diameter

Note: If the thickness of the column web is less than the combined thickness of the web cover plates, then the bearing capacity of the column web should also be checked.

CHECK 6	Structural integrity of splice	H



If it is necessary to comply with structural integrity requirements, then the Bearing type splice checks 2 and 3 should be carried out with:

$$F_2 = \frac{F_{tie}}{2}$$

based on the conservative assumption that the tie force is resisted by the two flange cover plates.

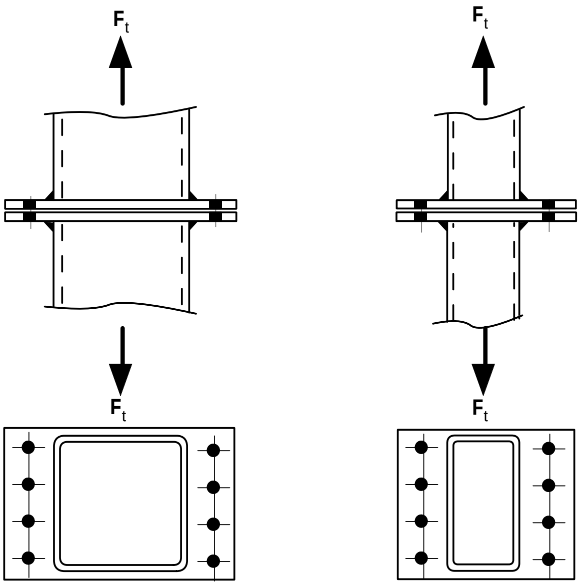
F_{tie} is the tensile force obtained from BS 5950-1, clause 2.4.5.3(c).

7.7 DESIGN PROCEDURE
RHS 'CAP AND BASE' OR 'END PLATE' SPLICE IN TENSION

Recommended design model

The design procedures for tension splices in square and rectangular hollow sections considers tension bolts being placed along two parallel faces as shown below. There are no structural integrity checks given since the principal force is tension.

The semi-empirical rules are based on the recommendations given in the CIDECT Design Guide[29] and take account of prying forces; it is therefore permissible to use the exact method bolt tension capacities of BS 5950-1.



CHECK 1 – **Recommended detailing practice**

CHECK 2 – **Complete end plate yielding** *(not applicable)*

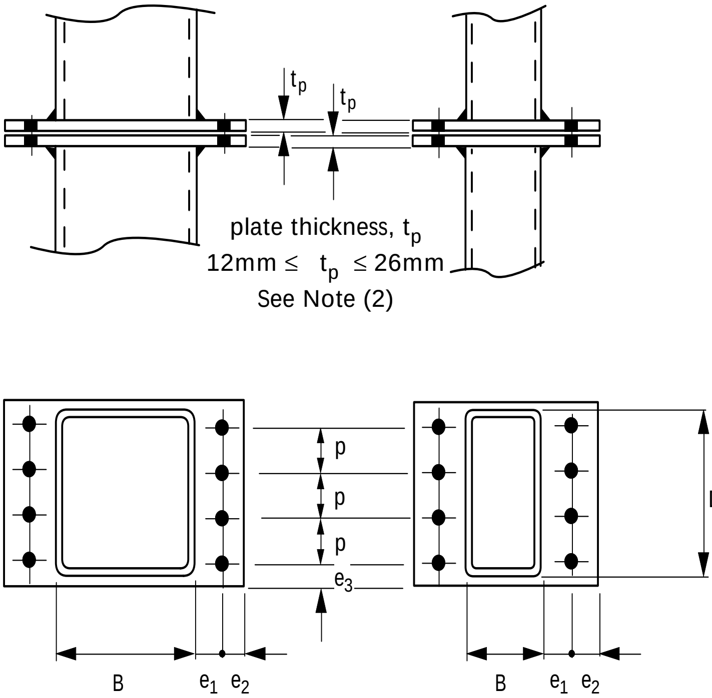
CHECK 3 – **Bolt failure with end plate yielding**

CHECK 4 – **Bolt failure**

CHECK 5 – **Weld capacity**

CHECK 6 – **Member capacity**

CHECK 1	Recommended detailing practice RHS splice in tension Bolted end plates	☐ ☐



Hole diameter D_h

$D_h = d + 2mm$ for $d \le 24mm$

$D_h = d + 3mm$ for $d > 24mm$

Bolt spacing p

$p \ge 2.5\ d$

Edge distance	e_2	$\ge$	$1.4D_h$	
Edge distance	e_3	$\ge$	$1.4D_h$	
Total number of bolts	N	$\le$	$\frac{2D}{p} + 2$	but $\ge$ 4

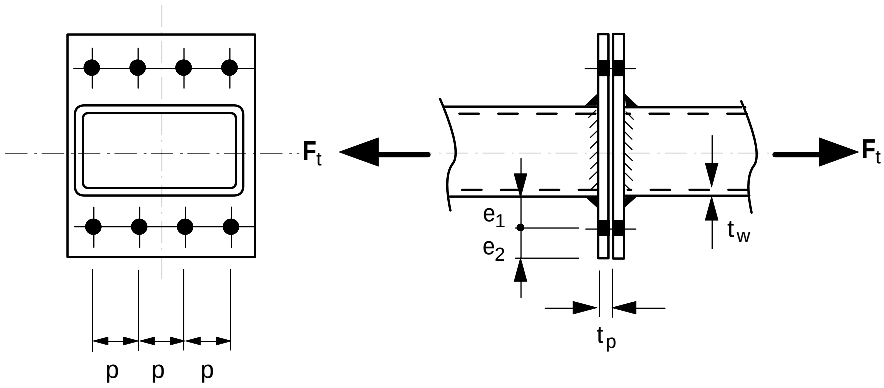
Notes:

(1) Dimension e_1 to be kept to a minimum.

(2) Plate thickness t_p should be limited to between 12mm and 26mm because this is the range for which the design method has been validated experimentally.

CHECK 2	Complete end plate yielding *(Not applicable due to plate thickness limitations)*	☐ ☐

CHECK 3	Bolt failure with end plate yielding



Basic requirement:

$$F_t \leq \frac{t_p^2 (1 + \delta\alpha) N}{K} \qquad \text{(kN)}$$

$$\delta = 1 - \frac{D_h}{p}$$

$$K = \frac{4\left(e_1 - (d/2) + t_w\right)10^3}{p_{yplate}\ \ p}$$

$$\alpha = \left(\frac{K P_t}{t_p^2} - 1\right)\left(\frac{e_{eff} + (d/2)}{\delta\,(e_{eff} + e_1 + t_w)}\right) \quad \text{but} \geq 0$$

where:

t_p = end plate thickness (mm)

N = total number of bolts (both rows)

D_h = bolt hole diameter (mm)

p = bolt pitch (mm)

d = bolt diameter (mm)

t_w = RHS wall thickness (mm)

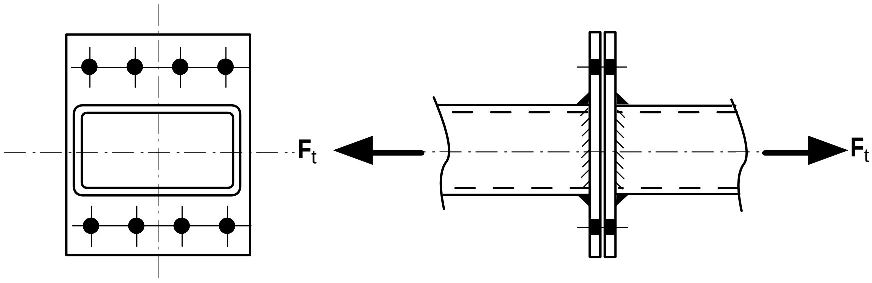
p_{yplate} = design strength of plate (N/mm^2)

P_t = Exact tension bolt capacity (kN) (see inset box)

e_{eff} = minimum of e_2 and $1.25e_1$ (mm)

Tension capacity - 8.8 bolts	
Bolt size	Exact tension capacity P_t
M20	137 kN
M24	198 kN
M30	314 kN

CHECK 4	Bolt failure



Basic requirement:

$$F_t \leq N\,P_t \ \text{(kN)}$$

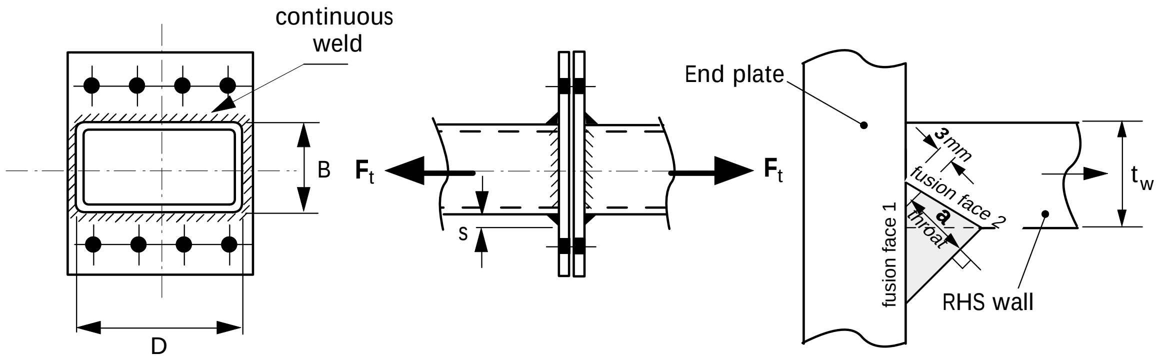
where:

P_t = Exact tension bolt capacity (kN)
(see inset box)

N = total number of bolts (both rows)

Tension capacity - 8.8 bolts	
Bolt size	Exact tension capacity P_t
M20	137 kN
M24	198 kN
M30	314 kN

CHECK 5	Weld



Weld tensile capacity

Basic requirement:

Provide continuous full-strength weld (See Note (1)and (2)) or alternatively:

$F_t \le 2D\,a\,(1.25p_w)$

where:

D = RHS depth

p_w = design strength of weld see BS 5950-1 Table 37. (220N/mm^2 for S275 steel 250N/mm^2 for S355 steel.)

For fillet weld:

a = weld throat thickness

= 0.7 s

s = weld leg length $\le$ 12mm

For partial penetration butt with additional fillet:

a = weld throat thickness according to geometry shown in Figure above

Notes:

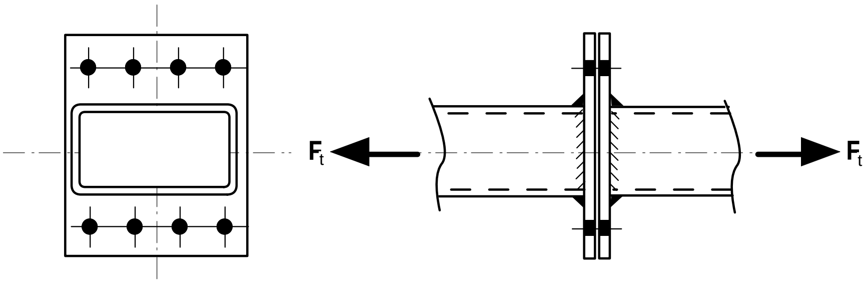
(1) The weld should be capable of developing the full strength of the RHS. A fillet weld should normally be used, but if the required leg length exceeds 12mm then a partial penetration butt weld with additional fillets may be a more economical solution.

(2) For a partial penetration butt weld with additional fillets, as shown in Figure above, note that:

- the depth of preparation should be 3mm deeper than the required penetration.
- the minimum penetration of $2\sqrt{t}$ specified in BS 5950-1 clause 6.9.2 does not apply to the detail shown.

(3) Depending on splice plate stiffness, the welded perimeter of RHS Sections will be loaded non-uniformly. In the absence of more precise design guidance, the effective weld length should be taken as the side lengths adjacent to the loaded bolts in tension.

CHECK 6	Member capacity



Member capacity

Basic requirement:

$$F_t \leq Ap_y$$

where:

A = cross-sectional area of RHS

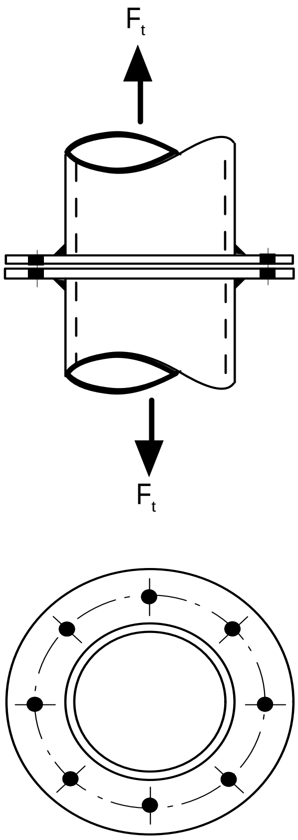
p_y = design strength of RHS

7.8 DESIGN PROCEDURE CHS END-PLATE SPLICE IN TENSION

Recommended design model

The design procedures for tension splices in circular hollow sections consider tension bolts being evenly placed radially around the section. There are no structural integrity checks given, since the principal force is tension.

The semi-empirical rules are based on the recommendations given in the CIDECT Design Guide[28] and take account of prying forces; it is therefore permissible to use the exact method bolt tension capacities.



CHECK 1 – Recommended detailing practice

CHECK 2 – Complete end plate yeilding

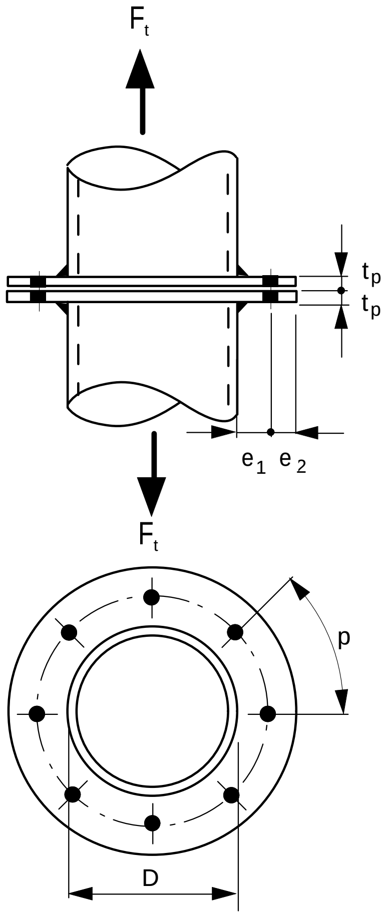
CHECK 3 – Bolt failure with end plate yielding

CHECK 4 – Bolt failure

CHECK 5 – Weld capacity

CHECK 6 – Member capacity

CHECK 1	Recommended detailing practice CHS Splice in tension Bolted end plates



Hole diameter D_h

$D_h = d + 2mm$ for $d \leq 24mm$

$D_h = d + 3mm$ for $d > 24mm$

edge distance $e_2 \geq 1.4\ D_h$

Bolt spacing p

$p \geq 2.5\ d$

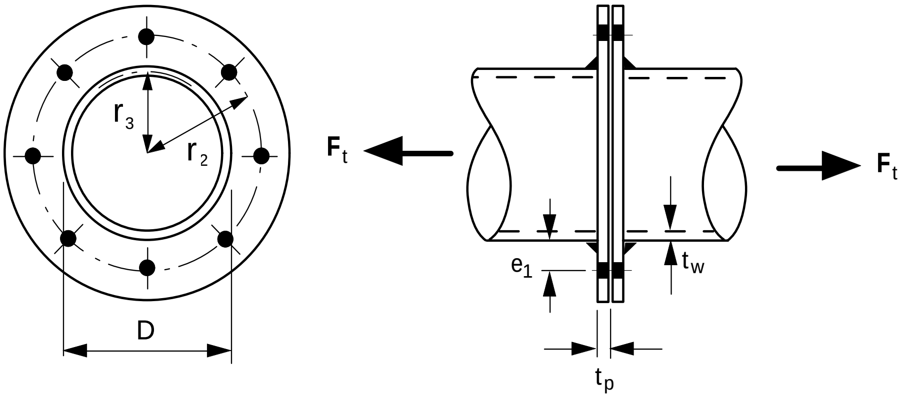
$p \leq 10\ d$

Notes: (1) Bolts to be equally spaced around circumference.

(2) At least 4 bolts to be used.

(2) Dimension e_1 to be kept to a minimum.

CHECK 2	Complete end plate yielding



Basic requirement:

$$F_t \leq \frac{t_p^2 \; p_{yplate} \, \pi \; f_3}{2}$$

where:

$$f_3 = \frac{1}{2k_1}\left(k_3 + (k_3^2 - 4\,k_1)^{0.5} \right)$$

k_1 = $ln\,(r_2\,/r_3)$ [*ln* = natural logarithm]

$$r_2 = \frac{D}{2} + e_1$$

$$r_3 = \frac{D - t_w}{2}$$

k_3 = $k_1 + 2$

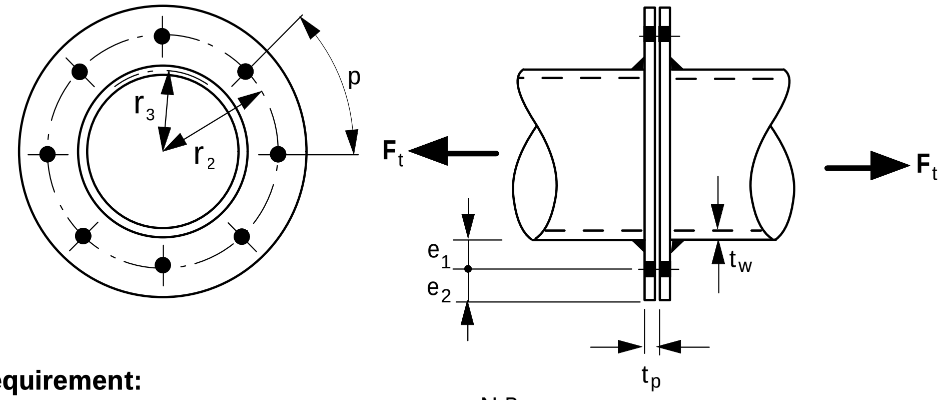
t_p = end plate thickness

p_{yplate} = design strength of plate

D = CHS diameter

t_w = CHS wall thickness

CHECK 3	Bolt failure with end plate yielding



Basic requirement:

$$F_t \leq \frac{N\,P_t}{\left(1 - \dfrac{1}{f_3} + \dfrac{1}{f_3\,ln(r_1/r_2)}\right)}$$ [*ln* = natural logarithm]

where:

$f_3 = \frac{1}{2k_1}\left(k_3 + (k_3^2 - 4\,k_1)^{0.5}\right)$

$k_1 = ln(r_2/r_3)$ [*ln* = natural logarithm]

N = total number of bolts

P_t = Exact tension bolt capacity (kN) (see inset box)

$r_1 = \frac{D}{2} + e_1 + e_{eff}$

$r_2 = \frac{D}{2} + e_1$

$e_{eff} = \mathrm{Min}(e_2,\ 1.25e_1)$

$r_3 = \frac{D - t_w}{2}$

$k_3 = k_1 + 2$

Tension capacity - 8.8 bolts	
Bolt size	Exact tension capacity P_t
M20	137 kN
M24	198 kN
M30	314 kN

CHECK 4	Bolt Failure

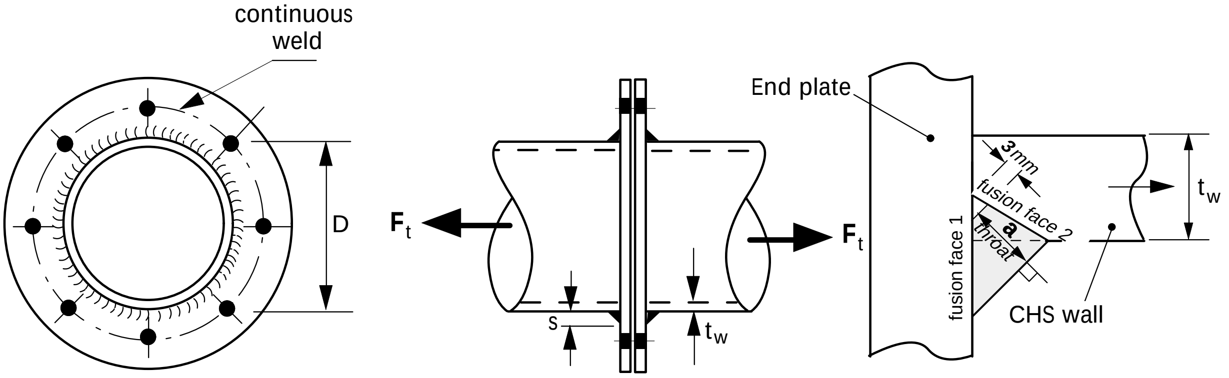
Basic requirement:

$F_t \leq N\,P_t$ (kN)

where: N = total number of bolts

P_t = Exact tension bolt capacity (kN) (see inset box above)

CHECK 5	Weld



Weld tensile capacity

Basic requirement:

provide continuous full-strength weld (See Note (1)and (2)) or alternatively

$F_t \leq \pi D a p_w$

where:

D = CHS diameter

p_w = design strength of weld see BS 5950-1 Table 37. (220N/mm^2 for S275 steel 250N/mm^2 for S355 steel.)

For fillet weld:

a = weld throat thickness

= 0.7 s

s = weld leg length $\leq$ 12mm

For partial penetration butt with additional fillet:

a = weld throat thickness according to geometry shown in Figure above

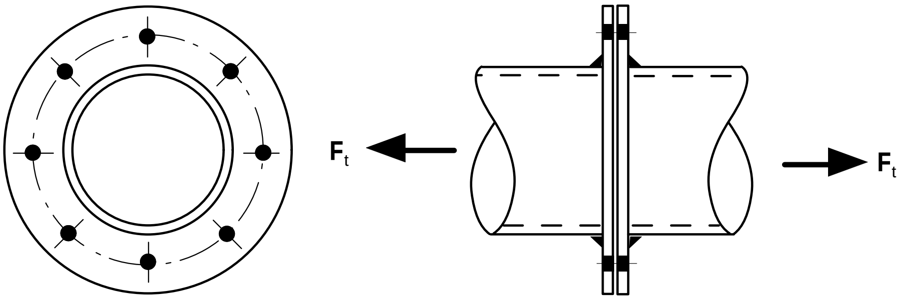
Notes:

(1) The weld should be capable of developing the full strength of the CHS. A fillet weld should normally be used, but if the required leg length exceeds 12mm then a partial penetration butt weld with additional fillets may be a more economical solution.

(2) For a partial penetration butt weld with additional fillets, as shown in Figure above, note that:

- the depth of preparation should be 3mm deeper than the required penetration.
- the minimum penetration of $2\sqrt{t}$ specified in BS 5950-1 clause 6.9.2 does not apply to the detail shown.

CHECK 6	Member capacity



Member capacity

Basic requirement:

$$F_t \leq A\,p_y$$

where:

A = cross-sectional area of CHS

p_y = design strength of CHS

7.9 WORKED EXAMPLES

The five worked examples for splices illustrate the design checks required for the most commonly used details:

Example 1: A bearing splice for connecting two different size Universal Column sections using external cover plates and with a division plate between.

Example 2: A connection as Example 1 but with a bending moment where net tension is present (additional checks which have to be made are shown).

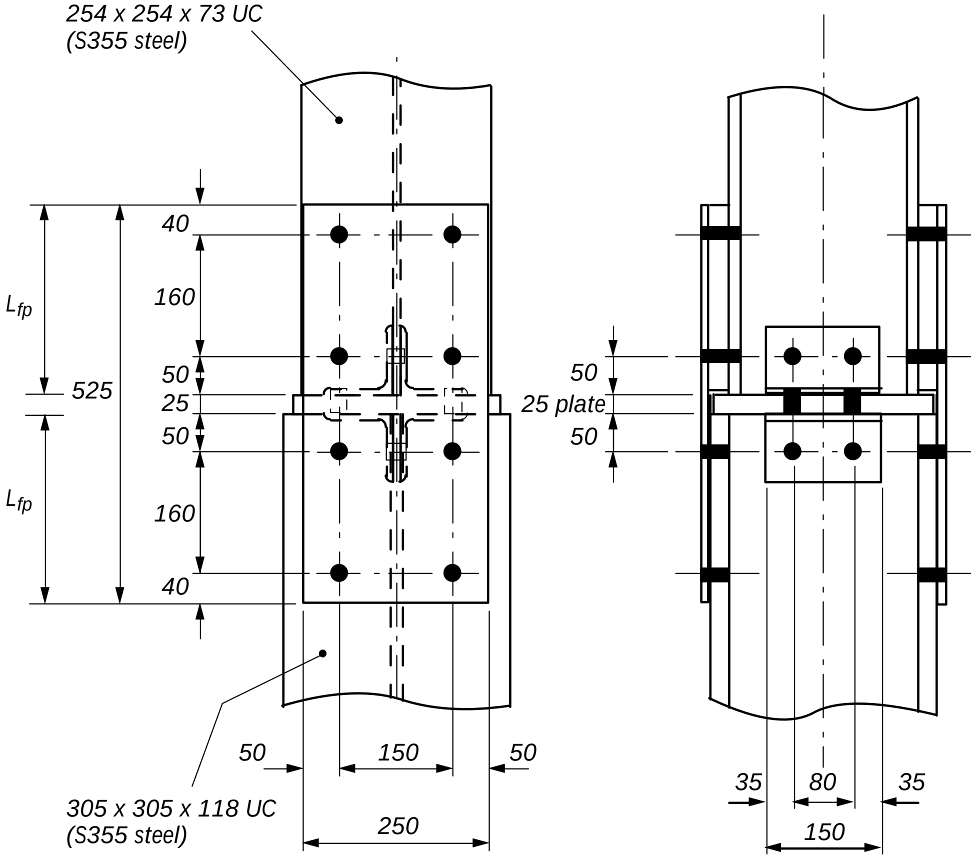
Example 3: A non-bearing splice for UCs with all forces developed in the bolted cover plates, including a structural integrity check.

Example 4: An RHS splice made with bolted end-plates which is subject to axial tension.

Example 5: A CHS splice made with bolted end-plates which is subject to axial tension.

CALCULATION SHEET BCSA	Job No: Joints in Steel Construction - Simple Connections		Sheet 1 of 7
	Title: Example 1 & 2 – Column Splices - UC bearing splice		
	Client: SCI/BCSA Connections Group		
	Calcs by: RS	Checked by: AM	Date: May 2002

BEARING COLUMN SPLICE FOR USE IN EXAMPLES 1 AND 2



See splice capacity Table H.33 in the yellow pages

Design information:

Flange cover plates:	2/250 x 12 x 525
Flange Packs:	2/250 x 30 x 240
Cleats:	4/90 x 90 x 8 Angles x 150 Long
Web Packs:	2/85 x 2 x 150
Division plate:	265 x 25 x 310
Bolts:	M20 Grade 8. 8
Fittings material:	S275 steel

Title	*Example 1 – Column Splices – UC bearing splice (No net tension)*	Sheet *2 of 7*

DESIGN EXAMPLE 1 — SPLICE PREPARED FOR CONTACT IN BEARING

(note:- In this example no net tension is developed in the flange cover plates.)

Check the column splice shown on sheet 1 for the following design forces.

Design Loading	***(Factored loads)***	
Dead	*825kN*	} *Axial compression*
Imposed	*942kN*	
Moment	*15kNm (About xx axis of column)*	
Shear	*8kN*	

SUMMARY OF FULL DESIGN CHECKS FOR EXAMPLE 1 ON SHEET 3

No checks needed on shear, which can be resisted by friction across the bearing surfaces.

Sheet Nos	***CHECK***	***Capacity***	***Applied Load***	***Comments***
3	***CHECK 1*** *Recommended detailing practice*	*Not Applicable*	*Not Applicable*	*All recommendations adopted, within reasonable practical limits*
3	***CHECK 2*** *Flange Cover Plates Net Tension Check*	*104.8kNm*	*15kNm*	*No net tension developed*

Title	Example 1 – Column Splices – UC bearing splice (No net tension)	Sheet 3 of 7

CHECK 1: Recommended detailing practice

External flange cover plates

Projection, L_{fp}	$\geq$	B_1 and 225 = 254.6 and 225 **Say 250mm**	**Acceptable**
Width, W_{fp}	$\geq$	B_1 = 254.6 **Say 250mm**	**Acceptable**

Note: Maximum vertical bolt spacing p = 14t, i.e. minimum thickness is p/14 — *BS 5950-1 Cl 6.2.1.2*

$$\text{Thickness} \geq \frac{T_1}{2} \text{ and } 10 \text{ and } \frac{p}{14}$$

$$= \frac{14.2}{2} \text{ and } 10 \text{ and } \frac{160}{14}$$

$$= 7.1 \text{ and } 10 \text{ and } 11.4$$ **Say 12mm** — **∴ O.K.**

$$\text{Packs, } t_{pa} = \frac{D_2 - D_1}{2} = \frac{314.5 - 254.1}{2}$$

$$= 30.2$$ **Say 30mm** — **Acceptable**

Division Plate

$$\text{Thickness} \geq \left[(D_2 - 2T_2) - (D_1 - 2T_1)\right] \times 0.5$$

$$= [(314.5 - (2 \times 18.7)) - (254.1 - (2 \times 14.2))] \times 0.5$$

$$= 25.7$$ **Say 25mm** — **Acceptable**

Web cleats

Use 90 x 90 x 8 Angles *to accommodate M20 bolts in opposite positions on adjoining legs*

$$\text{Length} \geq 0.5\,D_1 = 0.5 \times 254.1 = 127$$ **Say 150mm** — **∴ O.K.**

$$\text{Packs } t_{pa} = \frac{t_2 - t_1}{2} = \frac{12.0 - 8.6}{2}$$

$$= 1.7$$ **Say 2mm** — **∴ O.K.**

(Alternatively omit packs and provide slotted holes in cleats)

CHECK 2: Flange cover plates

Net Tension Check

Basic requirement for no net tension: $M < \dfrac{F_{cd}\ D}{2}$

$$\frac{F_{cd}\ D}{2} = \frac{825 \times 254.1}{2 \times 10^3} = 104.8\text{kNm}$$

$$M = 15\text{kNm} < 104.8\text{kNm}$$ **∴ O.K.**

Tension does not occur and the details derived in CHECK 1 can be adopted without further investigation.

Title	Example 2 – Column Splices – UC bearing splice (Net tension developed)	Sheet 4 of 7

DESIGN EXAMPLE 2 — SPLICE PREPARED FOR CONTACT IN BEARING

(Note:- In this example net tension is developed in the flange cover plates.)

Check the splice shown on sheet 1 for the following design forces

Design Loading (Factored loads)

Dead	*760kN*	} *Axial compression*
Imposed	*870kN*	
Moment	*110kNm*	*(About xx axis of column)*
Shear	*60kN*	

SUMMARY OF FULL DESIGN CHECKS FOR EXAMPLE 2 ON SHEETS 5 TO 7

Sheet Nos	***CHECK***	***Capacity***	***Applied Load***	***Comments***
5	***CHECK 1*** *Recommended detailing practice*	*Not Applicable*	*Not Applicable*	*All recommendations adopted, within reasonable practical limits*
5	***CHECK 2*** *Flange cover plate Net Tension Check*	***96.6kNm***	***110kNm***	***CRITICAL CHECK*** *Net tension developed*
5	***CHECK 3*** *Tensile Capacity of flange cover plate*	*815.8kN*	*52.9kN*	*Net tension is not significant and ordinary bolts are adequate*
6	***CHECK 4*** *Flange bolt group* *Shear Capacity* *Bearing Capacity*	 *265kN* *441.6kN*	 *52.9kN* *52.9kN*	*No need to check column flange in this example*
7	***CHECK 5*** *Structural integrety*	*Not Applicable*	*Not Applicable*	*If necessary carry out CHECKS 2 and 3 with tie force from BS 5950–1cl.2.4.5.3(c)*
7	***Additional CHECK*** *Shear Capacity of splice interface*	*163kN*	*60kN*	

Title	Example 2 – Column Splices – UC bearing splice (Net tension developed)	Sheet	5 of 7

CHECK 1: Recommended detailing practice

As for design example 1

CHECK 2: Flange Cover Plates

Net Tension Check

Basic requirement for no net tension: $M < \dfrac{F_{cd}\,D}{2}$

$$\frac{F_{cd}\;D}{2} = \frac{760 \times 254.1}{2 \times 10^3} = 96.6\text{kNm}$$

$$\mathbf{M = 110kNm \nless 96.6kNm}$$ **∴ Fails**

∴ Net tension <u>does</u> occur and the flange cover plates and their fastenings must be checked for a tensile force, F_2

$$F_2 = \frac{M}{D} - \frac{F_{cd}}{2}$$

$$= \frac{110 \times 10^3}{254.1} - \frac{760}{2}$$

$$= 52.9\text{ kN}$$

CHECK 3: Tensile Capacity of Flange Cover Plate

Basic requirement: $F_2 \leq \text{Min}\,(p_y\;A_{fp}\,,\,K_e\,p_y\,A_{fp.net})$

Gross area, $A_{fp} = 250 \times 12 = 3000\text{mm}^2$

Net area, $A_{fp.net} = 3000 - (2 \times 22 \times 12) = 2472\text{mm}^2$

$$p_y\,A_{fp} = \frac{275 \times 3000}{10^3} = 825\text{kN}$$

$$K_e\,p_y\,A_{fp.net} = \frac{1.2 \times 275 \times 2472}{10^3} = 815.8\text{kN}$$

Tension capacity = 815.8kN

$$\mathbf{F_2 = 52.9kN < 815.8kN}$$ **∴ O.K.**

Check the suitability of ordinary bolts.

10% of the design strength of the 254 x 254 x 73 UC (S355)

$$= 10\%\text{ of }355\text{N/mm}^2 = 35.5\text{N/mm}^2$$ *See 7.2 Fasteners*

Stress induced in the column flange by F_2
(it is sufficiently accurate to base this calculation on the gross area of the flange.)

$$= \frac{52.9 \times 10^3}{254.6 \times 14.2}$$

$$\mathbf{= 14.6N/mm^2 < 35.5N/mm^2}$$ **∴ O.K.**

There is no significant net tension in the column flange and the use of ordinary bolts in clearance holes is satisfactory.

Title	Example 2 – Column Splices – UC bearing splice (Net tension developed)	Sheet	6 of 7

CHECK 4: Flange bolt group

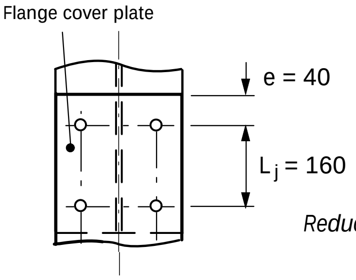
Shear Capacity of Bolt Group

Basic requirement: $F_2 \leq$ Reduction Factor $\times \Sigma P_s$

Single shear capacity, M20 8.8 bolt $P_s = 91.9kN$

Bolt capacities from Yellow pages Table H.49

For top pair of bolts $P_s = \min(91.9\ ,\ 0.5\, e\, t_{fp}\ p_{bs})$



$$0.5\, e\, t_{fp}\ p_{bs} = \frac{0.5 \times 40 \times 12 \times 460}{10^3}$$

$$= 110.4kN$$

p_{bs} from BS 5950-1 Table 32

$$\therefore P_s = 91.9kN$$

$$\text{Reduction Factor} = \min\left(\frac{5500 - L_j}{5000}\ ,\ \frac{9d}{8d + 3t_{pa}}\ ,\ 1\right)$$

$$\frac{9d}{8d + 3t_{pa}} = \frac{9 \times 20}{(8 \times 20) + (3 \times 30)}$$

$$= 0.72$$

Joint length, $L_j = 160mm < 500mm$

Therefore there is no long joint effect.

$$\text{Total shear capacity} = \text{Reduction Factor} \times \Sigma P_s$$

$$= 0.72 \times 4 \times 91.9$$

$$= 265kN$$

$\boldsymbol{F_2 = 52.9 < 265kN}$ **$\therefore$ O.K.**

Bearing Capacity

Basic requirement: $F_t \leq \Sigma P_{bs}$

Bearing capacity for one bolt, $P_{bs} = d\ t_{fp}\ p_{bs}$

$$= \frac{20 \times 12 \times 460}{10^3}$$

$$= 110.4kN$$

p_{bs} from BS 5950-1 Table 32

For top pair of bolts, $P_{bs} = \min(d\, t_{fp}\, p_{bs}\ ,\ 0.5e\, t_{fp}\, p_{bs})$

Total bearing capacity, $\Sigma P_{bs} = 4 \times 110.4$

$$= 441.6kN$$

$\boldsymbol{F_2 = 52.9 < 441.6kN}$ **$\therefore$ O.K.**

The column flange is thicker and of a higher grade of steel than the flange cover plate and is therefore adequate in bearing.

Note: *The capacity of the flange cover plate, 265kN (i.e. minimum of CHECKS 3 and 4) can be obtained from Table H.33 in the yellow pages in lieu of applying CHECKS 3 and 4. When using the tables it is still necessary to investigate the suitability of ordinary bolts (CHECKS 2 and 3) and the adequacy of the column flange in bearing (CHECK 4).*

Title	Example 2 – Column Splices – UC bearing splice (Net tension developed)	Sheet	7 of 7

CHECK 5: Structural Integrity of Splice

If it is necessary to comply with structural integrity requirements, then CHECKS 3 and 4 should be carried out with:

$$F_2 = F_{tie} / 2$$

based on the conservative assumption that the tie force is resisted by the flange cover plates.

F_{tie} is the tensile force obtained from BS 5950-1, clause 2.4.5.3(c).

Additional CHECK : Horizontal shear Capacity of Splice Interface

For a bearing type splice, any horizontal shear F_v is assumed to be resisted by friction across the splice interface.

Basic requirement: $F_v \leq$ **shear capacity of splice interface**

The coefficient of friction μ_f for a steel to steel interface depends upon the surface condition of the steel and on any coatings provided.

Conservatively, for steel with no surface treatment, and with complete mill scale, the coefficient of friction, μ_f may be taken as 0.20. ECCS Pub No. 37[33]

Shear capacity of splice interface = Vertical load x Coefficient of friction

$$\text{Vertical load with coexistent shear} = \frac{M}{D} + \frac{F_{cd}}{2}$$

$$= \frac{110 \times 10^3}{254} + \frac{760}{2}$$

$$= 433 + 380$$

$$= 813\text{kN}$$

$$\text{Shear capacity of splice interface} = 813 \times 0.20$$

$$= 163\text{kN}$$

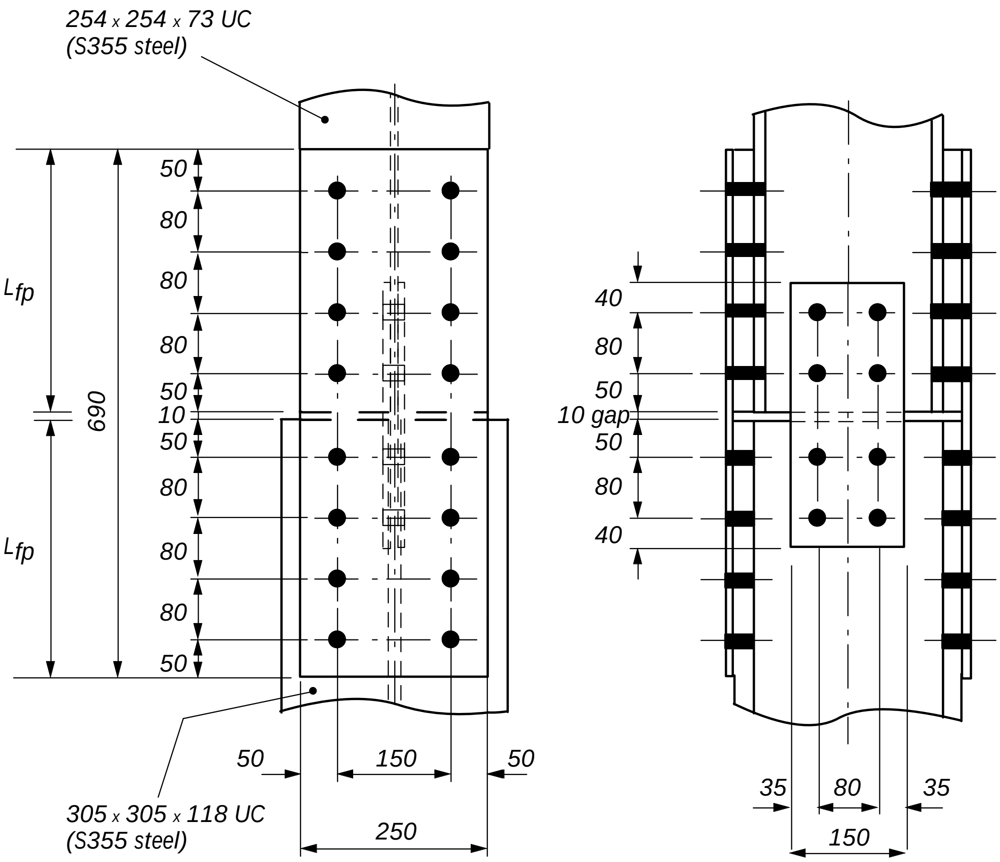
$$F_v = 60\text{kN} < 163\text{kN}$$

∴ **O.K.**

Therefore the splice detail is adequate.

CALCULATION SHEET BCSA	Job No Joints in Steel Construction - Simple Connections		Sheet 1 of 7
	Title Example 3 – Column Splices - UC Non-bearing splice		
	Client SCI/BCSA Connections Group		
	Calcs by RS	Checked by AM	Date May 2002

DESIGN EXAMPLE 3 – NON BEARING COLUMN SPLICE



Design Information:

Flange cover plates:	2/250 x12 x 690
Flange packs:	2/250 x 30 x 340
Web cover plates:	2/150 x 8 x 350
Web packs:	2/150 x 2 x 170
Bolts:	M24 grade 8. 8
Fittings material:	S275 steel

Title	*Example 3 – Column Splices – UC Non-bearing Splice*	Sheet *2 of 7*

Check the column splice shown on sheet 1 for the following design forces.

Design Loading (factored loads)

Dead	*825kN*	} *axial compression*
Imposed	*942kN*	
Moment	*15kNm*	*(About xx axis of column)*
Shear	*8kN*	*(Resisted by web cover plates)*
Tie force (structural Integrity)	*400kN*	*Load from floor below splice*

SUMMARY 0F FULL DESIGN CHECKS FOR EXAMPLE 3

Sheet Nos	*CHECK*	*Capacity kN*	*Applied kN*	*Comments*
3	***CHECK 1*** *Recommended detailing practice*	*Not applicable*	*Not applicable*	*All recommendations adopted within reasonable practical limits.*
3	***CHECK 2*** *Flange cover plates* *Axial Capacity*	 *825*	*No net tension* *744*	
4	***CHECK 3*** *Flange Bolt Group* *Shear capacity* *Bearing Capacity*	 *813* *1060*	 *744* *744*	
5	***CHECK 4*** *Web cover plates* *Axial Capacity*	 *330*	 *199*	
5,6	***CHECK 5*** *Web Bolt Group* *Shear capacity* *Bearing Capacity (web cover plate)* *Bearing capacity (column web)*	 *411* *324* ***454***	 *199* *199* ***398***	 ***CRITICAL CHECK***
7	***CHECK 6*** ***Structural Integrity*** *Axial Capacity of Flange Cover Plate*	 ***784***	 ***200***	 ***CRITICAL CHECK*** *for Structural integrity*

Title *Example 3 – Column Splices – UC Non-bearing Splice*	Sheet *3 of 7*

CHECK 1: Recommended detailing practice

Bolt diameter $\geq$ *3/4 thickness of packing either side*

$24mm > 0.75 \times 30 = 22.5mm$ $\therefore$ **O.K.**

If this is found to be impracticable then the packs may be fillet welded to the column flanges and this check omitted.

All other requirements are satisfied as in design example 1.

CHECK 2: Flange Cover Plate

Axial Capacity of flange cover plates

Basic requirement for compression:

$$\mathbf{F_1 \leq p_y \; A_{fp}}$$

Conservatively,

$$F_1 = \frac{M}{D} + F_c\left[\frac{A_f}{A}\right]$$

A_{fp} = *area of flange cover plate*

A_f = *area of one flange of smaller column*

A = *total area of smaller column*

$$F_1 = \frac{15 \times 10^3}{254} + (825 + 942)\left[\frac{254 \times 14.2}{93.1 \times 10^2}\right]$$

$$= 744kN \text{ compression}$$

Compression capacity $= p_y \; A_{fp}$

$$= \frac{275 \times 250 \times 12}{10^3} = 825kN$$

$$\mathbf{F_1 = 744kN < 825kN}$$ $\therefore$ **O.K.**

Check for tension,

$$F_2 = \frac{M}{D} - F_{cd}\left[\frac{A_f}{A}\right]$$

$$= \frac{15 \times 10^3}{254} - 825\left[\frac{254 \times 14.2}{93.1 \times 10^2}\right]$$

$$= -261kN$$

This indicates a net compression of 261kN

Therefore tension is not present.

Assuming that slip is acceptable, and since there is no net tension, the use of ordinary bolts is satisfactory.

Title	Example 3 – Column Splices – UC Non-bearing Splice	Sheet	4 of 7

CHECK 3: Flange Bolt Group

Shear Capacity of Bolt Group

Basic requirement: $F_1 \leq$ Reduction Factor $\times \Sigma P_s$

bolt capacities from Yellow pages Table H.49

Single shear capacity for an M24 8.8 bolt P_s = 132kN

For top pair of bolts $P_s = \min(132\ ,\ 0.5\ e\ t_{fp}\ p_{bs})$

$$= \frac{0.5 \times 50 \times 12 \times 460}{10^3}$$

p_{bs} from BS 5950-1 Table 32

= 138kN > 132kN

$\therefore P_s$ = 132kN

Reduction Factor $= \min\left(\frac{5500 - L_j}{5000}\ ,\ \frac{9d}{8d + 3t_{pa}}\ ,\ 1\right)$

$$\frac{9d}{8d + 3t_{pa}} = \frac{9 \times 24}{(8 \times 24) + (3 \times 30)}$$

= 0.77

Joint length, L_j = 3 x 80 = 240mm < 500mm

Therefore there is no long joint effect.

Total shear capacity = Reduction Factor $\times \Sigma P_s$

= 0.77 x 8 x 132

= 813kN > 744kN

F_1 = 744kN < 813kN **$\therefore$ O.K.**

Bearing Capacity

Basic requirement: $F_1 \leq \Sigma P_{bs}$

Bearing capacity for one bolt $P_{bs} = d\ t_{fp}\ p_{bs}$

$$= \frac{24 \times 12 \times 460}{10^3}$$

p_{bs} from BS 5950-1 Table 32

= 132.5kN

For top pair of bolts $P_{bs} = \min(d\ t_{fp}\ p_{bs}\ ,\ 0.5e\ t_{fp}\ p_{bs})$

$\therefore P_{bs}$ = 132.5kN

Total bearing capacity, ΣP_{bs} = 8 x 132.5

= 1060kN

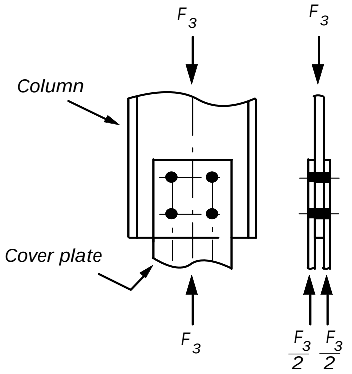
F_1 = 744kN < 1060kN **$\therefore$ O.K.**

The column flange, being 14.2mm thick and of S355 steel is also adequate in bearing.

Title	*Example 3 – Column Splices – UC Non-bearing Splice*	Sheet	*5 of 7*

CHECK 4: Web cover plate

Axial capacity



Basic requirement: $\frac{F_3}{2} \leq p_y\ A_{wp}$

Conservatively, $F_3 = \frac{F_c\ A_w}{A}$

F_c = *Axial load due to factored dead & imposed loads*

A = *Total area of upper column*

A_w = *Area of web of upper column* = $A - 2A_f$

A_f = *Flange area of upper column*

$$F_3 = \frac{(825 + 942)\left[(93.1 \times 10^2) - (2 \times 254 \times 14.2)\right]}{93.1 \times 10^2}$$

= *398kN*

Compression capacity = $p_y A_{wp}$

$= \frac{275 \times 150 \times 8}{10^3}$

= *330kN*

$\frac{F_3}{2}$ = **199kN < 330kN** ∴ **O.K.**

CHECK 5: Web cover plate bolt group

Shear capacity

Basic requirement: $\frac{F3}{2} < \Sigma P_s$

For an M24 8.8 bolt, P_s = *132kN* *as before* — CHECK 3

For top pair of bolts (end distance = 40mm),

$P_s = \min(132,\ 0.5e\ t_{wp}\ p_{bs})$

$0.5e\ t_{wp}\ p_{bs} = \frac{0.5 \times 40 \times 8 \times 460}{10^3}$ — *p_{bs} from BS 5950-1 Table 32*

= *73.6kN*

∴ P_s = *73.6kN*

Total shear capacity = ΣP_s

= $(2 \times 132.4) + (2 \times 73.6)$ = *411kN*

$\frac{F_3}{2}$ = **199KN < 411kN** ∴ **O.K.**

Title	*Example 3 – Column Splices – UC Non-bearing Splice*	Sheet	*6 of 7*

Bearing Capacity

Basic requirement: $\frac{F_3}{2} \leq \Sigma P_{bs}$

Bolts generally:

Bearing capacity, $P_{bs} = d\ t\ P_{bs}$ — *p_{bs} from BS 5950-1 Table 32*

$= \frac{24 \times 8 \times 460}{10^3}$

$= 88.3kN$

For top pair of bolts:

$P_{bs} = 73.6kN$ *as before*

Total bearing capacity $= \Sigma P_{bs}$

$= (2 \times 88.3) + (2 \times 73.6) = 324kN$

$\frac{F_3}{2}$ **= 199kN < 324kN** — **∴ O.K.**

Bearing on Column Web

The thickness of the column web (8.6mm) is less than the combined thickness of the web cover plates (2 x 8 = 16mm) and must therefore be checked for bearing.

Basic requirement: $F_3 \leq \Sigma P_{bs}$

End distance of bolts adjacent to end of column

$= 50mm > 2d$

Therefore bearing capacity per bolt on grade S355 web, — *p_{bs} from BS 5950-1 Table 32*

$P_{bs} = d\ t\ p_{bs}$

$= \frac{24 \times 8.6 \times 550}{10^3}$

$= 113.5kN$

Total bearing capacity $= \Sigma P_{bs}$

$= 4 \times 113.5 = 454kN$

F_3 **= 398kN < 454kN** — **∴ O.K.**

Title *Example 3 – Column Splices – UC Non-bearing Splice*	Sheet *7 of 7*

CHECK 6: Check for Structural Integrity

The maximum factored vertical load applied to the column from any floor down to the next splice

BS 5950-1 Cl 2.4.5.3(c)

$$F_{tie} = 400kN$$

All column splices must be capable of resisting a tensile force of not less than the maximum vertical load applied to the column from any floor down to the next splice.

Design force applied to each flange cover plate,

$$F_2 = \frac{400}{2} = 200kN \quad \text{tension}$$

Axial capacity of flange cover plate.

$$\text{Tension capacity} = \min(p_y A_{fp}, K_e p_y A_{fp.net})$$

$$A_{fp} = 250 \times 12$$

$$= 3000\ mm^2$$

$$A_{fp.net} = 3000 - (2 \times 26 \times 12)] = 2376mm^2$$

$$p_y A_{fp} = \frac{275 \times 3000}{10^3} = 825kN$$

$$K_e p_y A_{fp.net} = \frac{1.2 \times 275 \times 2376}{10^3} = 784kN$$

$$\text{Tension capacity} = 784kN$$

$$\mathbf{F_2 = 200kN < 784kN}$$

∴ O.K.

Shear capacity of bolt group connecting flange cover plate to column flange (CHECK 3).

Total shear capacity = 813kN (as before) — *Sheet 4*

$$\mathbf{F_2 = 200kN < 813kN}$$

∴ O.K.

Bearing capacity of flange cover plate connected to column flange (CHECK 3).

Total bearing capacity = 1060 kN (as before) — *Sheet 4*

$$\mathbf{F_2 = 200kN < 1060kN}$$

∴ O.K.

Therefore, the splice detail shown on Sheet 1 is adequate.

Structural Integrity Checks Using Capacity Tables

BS 5950-1 Cl 2.4.5.3(c)

Total tie force applied to the column splice = 400kN

In this example the flange plates are 250 x 12 but are connected using 8 M24 bolts instead of 4 M20 bolts as in the bearing type splices shown in Example 1 and in the yellow pages Table H.33 standard geometry for these members.

From the Yellow pages standard geometry Table H.33 the tensile capacity of one standard splice plate is 264kN. Therefore the tensile capacity of the whole connection as stated in Table H.31 note (5) is:

$$264 \times 2 = 528kN$$

$$\mathbf{400kN < 528kN}$$

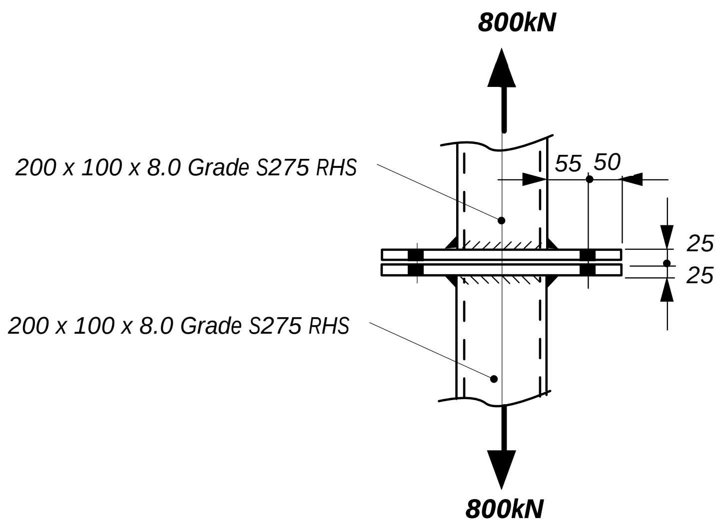
∴ O.K.

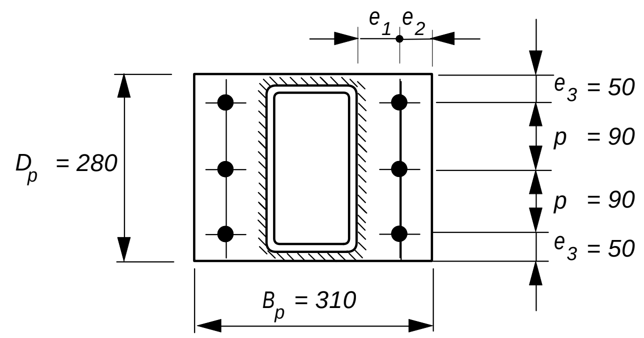
Thus the stronger non-bearing splice in this example must also be adequate.

CALCULATION SHEET BCSA	Job No Joints in Steel Construction - Simple Connections	Sheet 1 of 5
	Title Example 4 - Splices - RHS tension splice	
	Client SCI/BCSA Connections Group	
	Calcs by RS / Checked by AW	Date May 2002

DESIGN EXAMPLE 4 – RHS TENSION SPLICE

Check the following connection for the design forces shown:





Total number of bolts N = 6

Design Information:

End Plates:	280 x 310 x 25
Bolts:	M24 8.8
Material:	S275 steel
Weld:	12mm fillet

Title	Example 4 - Splices - RHS tension splice	Sheet 2 of 5

SUMMARY 0F FULL DESIGN CHECKS FOR EXAMPLE 4

Sheet Nos	CHECK	Capacity kN	Applied kN	Comments
3	**CHECK 1** Recommended detailing practice	Not Applicable	Not Applicable	All recommendations adopted
3	**CHECK 2** Complete end plate yielding	-	-	Not applicable due to end plate thickness limitations
3	**CHECK 3** Bolt failure with end plate yielding	**850**	**800**	**CRITICAL CHECK**
4	**CHECK 4** Bolt Group - Tension capacity	1188	800	
5	**CHECK 5** Weld - Tension capacity	924	800	Full strength welds
5	**CHECK 6** Member - Tension capacity	1232	800	

CONNECTION DESIGN USING CAPACITY TABLES FROM YELLOW PAGES

Yellow pages Table H.38

200 x 100 x 8.0 RHS Grade S275 Standard connection

Total number of bolts = 6

End plate thickness = 25mm

Tying capacity = 817 kN

See Table H.34 note (2)

Applied force = 800 kN

800kN < 817kN

∴ O.K.

Note: The capacity from Table H.38 is conservatively based on the thickest section in the range.

Title	Example 4 - Splices - RHS tension splice	Sheet 3 of 5

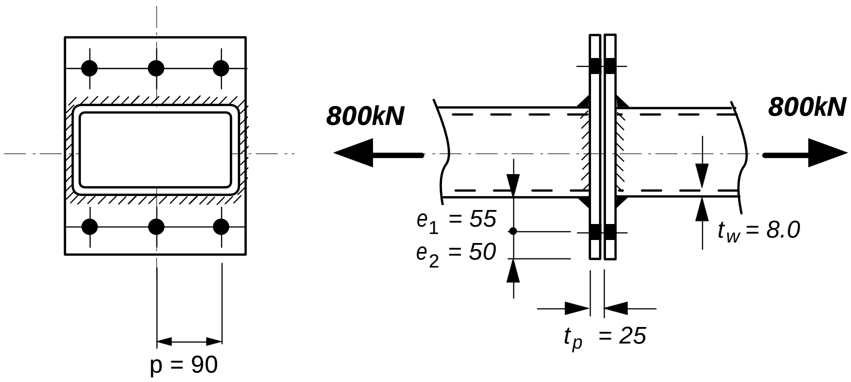
CHECK 1: Recommended detailing practice

End Plates	($\geq 12mm < 26mm$)	t_p	=	25mm
Bolts	($N \leq \frac{2D}{p} + 2 = 6.4$)	N	=	6 $\geq$ 4
		d	=	24mm dia
Holes	($d + 2mm$)	D_h	=	26mm
Spacing	($p \geq 2.5d = 60$)	p	=	90mm
	($e_2 \geq 1.4D_h = 36$)	e_2	=	50mm
	($e_3 \geq 1.4D_h = 36$)	e_3	=	50mm

CHECK 2: Complete end plate yielding - Not applicable

(due to plate thickness limitations)

CHECK 3: Bolt failure with end plate yielding



Basic Requirement:

$$F_t \leq \left(\frac{t_p^2 (1 + \delta\alpha) N}{K} \right)$$

$$\delta = 1 - \frac{D_h}{p} = 1 - \frac{26}{90} = 0.711$$

$$e_{eff} = \text{minimum of } e_2 \text{ and } 1.25e_1 = 50 \text{ mm}$$

$$K = \frac{4\,(e_1 - (d/2) + t_w) \times 10^3}{p_{y\,plate}\;\; p}$$

$$= \frac{4\,(55 - (24/2) + 8.0) \times 10^3}{265 \times 90} = 8.55$$

$$\alpha = \left(\frac{K\,P_t}{t_p^2} - 1 \right) \left(\frac{e_{eff} + (d/2)}{\delta\,(e_{eff} + e_1 + t_w)} \right) \qquad P_t = 198kN \text{ (Exact bolt capacity)}$$

P_t from Yellow pages Table H.49

$$= \left(\frac{8.55 \times 198}{25^2} - 1 \right) \left(\frac{50 + (24/2)}{0.711\,(50 + 55 + 8.0)} \right) = 1.319$$

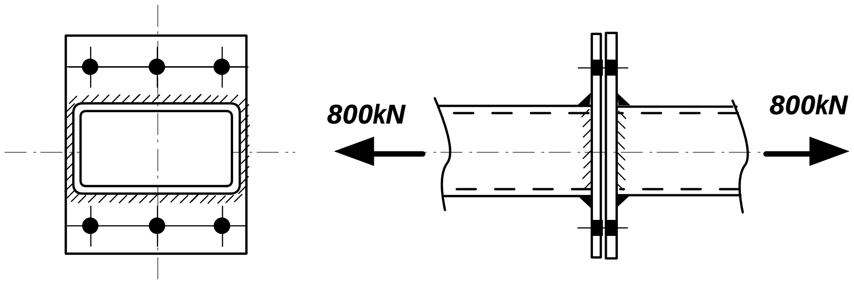
$$\left(\frac{t_p^2 (1 + \delta\alpha) N}{K} \right) = \left(\frac{25^2 (1 + (0.711 \times 1.319))\, 6}{8.55} \right) = 850N$$

$$\mathbf{F_t = 800kN < 850\ kN}$$

∴ O.K.

Title	*Example 4 - Splices - RHS tension splice*	Sheet *4 of 5*

CHECK 4: Bolt failure



Basic Requirement for bolt tensile capacity:

$$F_t \leq N P_t$$

P_t from Yellow pages Table H.49

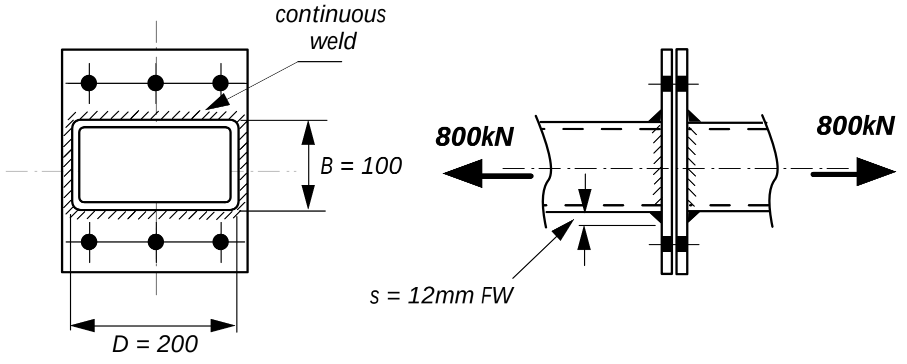
$N P_t = 6 \times 198$

$= 1188$ kN

$F_t = 800\text{kN} \leq 1188\text{kN}$ $\therefore$ **O.K**

Title	Example 4 - Splices - RHS tension splice	Sheet	5 of 5

CHECK 5: Welds



Basic Requirement for weld tensile capacity:

(i) $t_w \leq a$ **(continuous full strenghth weld)**

or

(ii) $F_t \leq 2\,D\,a\,p_w \times 1.25$

p_w = design strength of weld = $220N/mm^2$ — *p_w from BS 5950 -1 Table 37*

weld throat $a = 0.7\,s = 0.7 \times 12mm = 8.4mm$

(i) $t_w = 8.0mm \leq 8.4mm$ — ∴ O.K

$$2\,D\,a\,p_w \times 1.25 = \frac{2 \times 200 \times 8.4 \times 220 \times 1.25}{10^3} = 924kN$$

(ii) $F_t = 800kN < 924kN$ — ∴ O.K

CHECK 6: Member Capacity

Basic Requirement:

$F_t \leq A\,p_y$

$A = 44.8\ cm^2$ — *Yellow pages Table H.72*

$p_y = 275\ N/mm^2$ — *p_y from BS 5950 -1 Table 9*

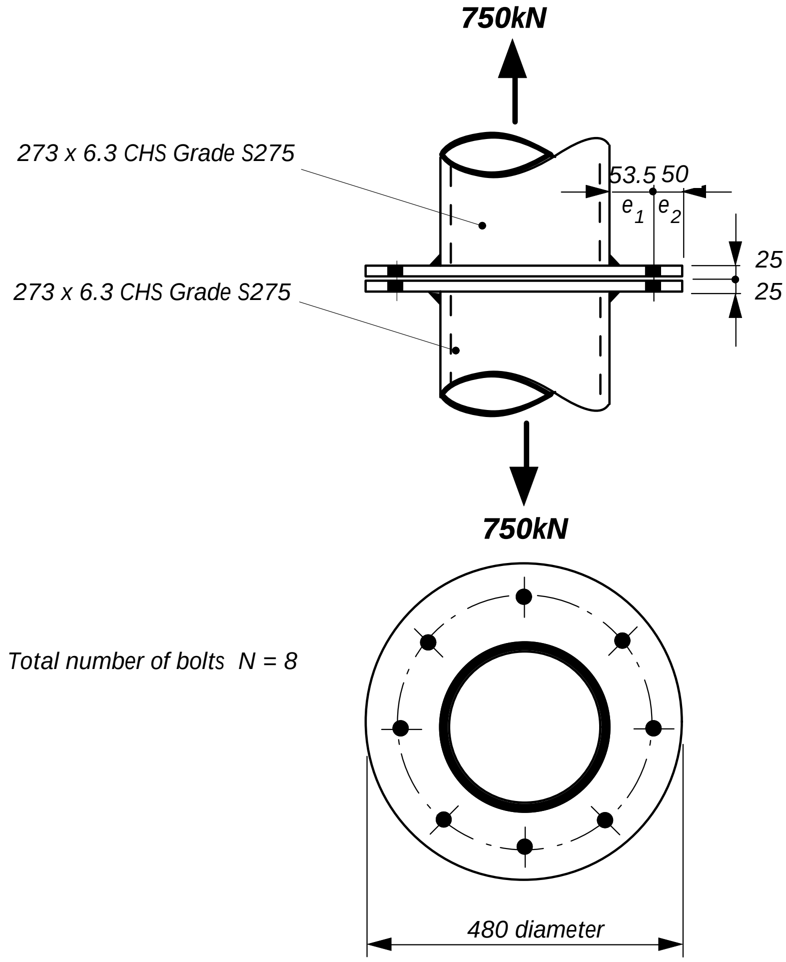
$$A\,p_y = \frac{44.8 \times 10^2 \times 275}{10^3} = 1232kN$$

$F_t = 800kN < 1232kN$ — ∴ O.K

CALCULATION SHEET BCSA	Job No: Joints in Steel Construction - Simple Connections		Sheet 1 of 5
	Title: Example 5 - Splices - CHS tension splice		
	Client: SCI/BCSA Connections Group		
	Calcs by: RS	Checked by: AW	Date: May 2002

DESIGN EXAMPLE 5 – TENSION SPLICE

Check the following connection for the design forces shown:



Design Information:

End Plates:	25mm thick
Bolts:	M24 8.8
Material:	S275 steel
Weld:	12mm fillet

Title	*Example 5 - Splices - CHS tension splice*	Sheet *2 of 5*

SUMMARY 0F FULL DESIGN CHECKS FOR EXAMPLE 5

Sheet Nos	***CHECK***	***Capacity kN***	***Applied kN***	***Comments***
3	***CHECK 1*** *Recommended detailing practice*	*Not Applicable*	*Not Applicable*	*All recommendations adopted*
3	***CHECK 2*** *Complete end plate yielding*	*1610*	*750*	
4	***CHECK 3*** *Bolt failure with end plate yielding*	***1036***	***750***	***CRITICAL CHECK***
4	***CHECK 4*** *Bolt failure*	*1584*	*750*	
5	***CHECK 5*** *Weld* *- Tension Capacity*	*1585*	*750*	*Full strength welds*
5	***CHECK 6*** *Member* *- Tension capacity*	*1452*	*750*	

CONNECTION DESIGN USING CAPACITY TABLES FROM YELLOW PAGES

Yellow pages Table H.36

273 x 6.3 CHS Grade S275 Standard connection

Total number of bolts N	=	*8*
End plate thickness	=	*20mm*
Tying capacity	=	*946 kN*
Applied force	=	*750 kN*
750kN	**<**	***946kN***

See Table H.34 note (2)

∴ O.K.

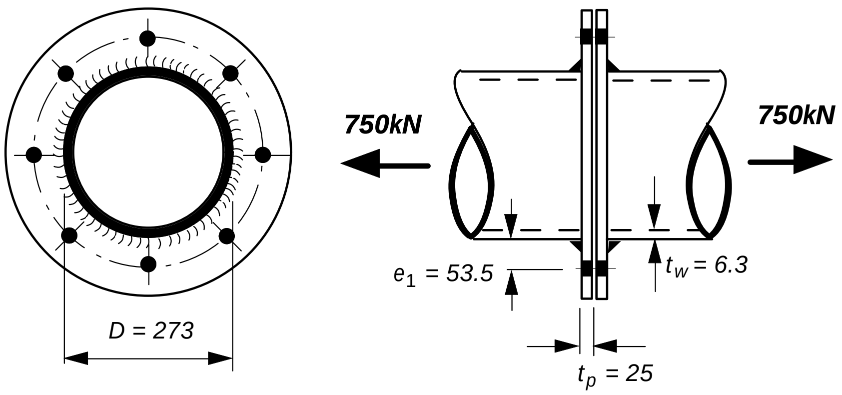
Note: *The capacity from Table H.36 is conservatively based on the thickest section in the range.*

Title	Example 5 - Splices - CHS tension splice	Sheet 3 of 5

CHECK 1: Recommended detailing practice

End Plates	t_p	=	25mm
Bolts	d	=	24mm dia
Holes	D_h	=	26mm
Spacing ($\geq 2.5d = 60$)	p	=	$380 \times \pi / 8$
($< 10d = 240$)		=	149mm
Edge distance ($\geq 1.4D_h = 36$)	e_2	=	50mm

CHECK 2: Complete end plate yielding



Basic requirement:

$$F_t \leq \frac{t_p^2\, p_{y\,plate}\, \pi\, f_3}{2}$$

$$f_3 = \frac{1}{2k_1}\left(k_3 + (k_3^2 - 4\,k_1)^{0.5}\right)$$

$$k_1 = l_n\,(r_2 / r_3) \qquad [l_n = \text{natural } \log_e]$$

$$r_2 = \frac{D}{2} + e_1 = \frac{273}{2} + 53.5 = 190$$

$$r_3 = \frac{D - t_w}{2} = \frac{273 - 6.3}{2} = 133.4$$

$$k_1 = l_n \frac{190}{133.4} = l_n 1.42 = 0.354$$

$$k_3 = k_1 + 2 = 0.354 + 2 = 2.354$$

$$\therefore\ f_3 = \frac{1}{2 \times 0.354}\left(2.354 + (2.354^2 - 4 \times 0.354)^{0.5}\right)$$

$$= 6.19$$

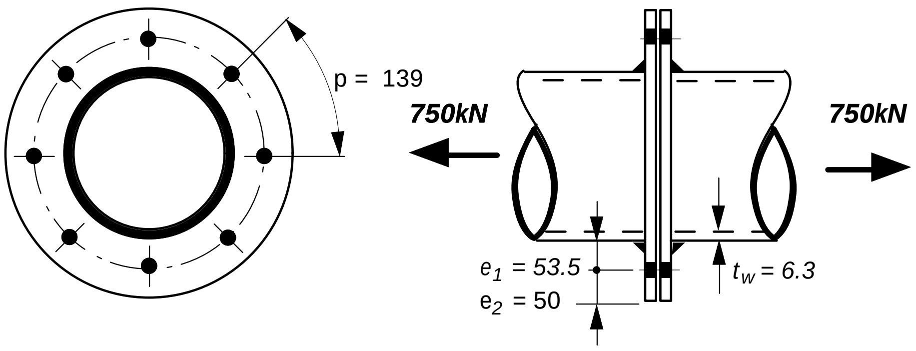
$$\therefore\ \text{Plate capacity} = \frac{25^2 \times 265 \times \pi \times 6.19}{2 \times 10^3} = 1610\text{kN}$$

$$F_t = 750\text{kN} \leq 1610\text{ kN}$$

∴ O.K.

Title	*Example 5 - Splices - CHS tension splice*	Sheet *4 of 5*

CHECK 3: Bolt failure with end plate yielding



Basic requirement:

$$F_t \le \frac{NP_t}{\left(1 - \frac{1}{f_3} + \frac{1}{f_3\, l_n(r_1/r_2)}\right)} \qquad [l_n = \text{natural } \log_e]$$

P_t = Exact bolt capacity = 198kN — *P_t from Yellow pages Table H.49*

f_3 = 6.19 — *CHECK 2*

e_{eff} = minimum of e_2 and $1.25e_1$ = 50 mm

$$r_1 = \frac{D}{2} + e_1 + e_{eff} = \frac{273}{2} + 53.5 + 50 = 240\text{mm}$$

$$r_2 = \frac{D}{2} + e_1 = \frac{273}{2} + 53.5 = 190\text{mm}$$

$$l_n(r_1/r_2) = l_n \frac{240}{190} = l_n\ 1.263 = 0.234$$

$$\therefore \text{ Bolt group capacity} = \frac{8 \times 198}{\left(1 - \frac{1}{6.19} + \frac{1}{6.19 \times 0.234}\right)} = 1036\text{kN}$$

$$\mathbf{F_t = 750kN < 1036\ kN}$$

∴ O.K.

CHECK 4: Bolt failure

Basic requirement for bolt tensile capacity:

$$F_t \le N\,P_t$$

P_t from Yellow pages Table H.49

$N P_t$ = 8 x 198

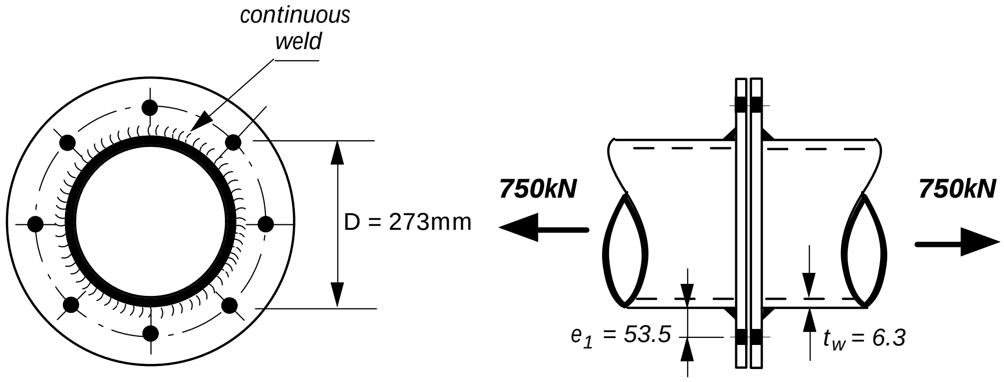
= 1584 kN

$$\mathbf{F_t = 750kN \le 1584kN}$$

∴ O.K

Title	*Example 5 - Splices - CHS tension splice*	Sheet	*5 of 5*

CHECK 5: Welds



Basic requirement for weld tensile capacity:

(i) $t_w \leq a$ **(continuous full strenghth weld)**

or

(ii) $F_t \leq \pi D a p_w$

p_w = design strength of weld = 220N/mm^2 — *p_w from BS 5950 -1 Table 37*

weld throat $a = 0.7\,s = 0.7 \times 12\text{mm} = 8.4\text{mm}$

(i) $t_w = 6.3\text{mm} \leq 8.4\text{mm}$ — **∴ O.K**

$$\pi D a p_w = \frac{\pi \times 273 \times 8.4 \times 220}{10^3} = 1585\text{kN}$$

(ii) $F_t = 750\text{kN} < 1585\text{kN}$ — **∴ O.K**

CHECK 6: Member Capacity

Basic requirement:

$F_t \leq A\,p_y$

$A = 52.8\ \text{cm}^2$ — *Yellow pages Table H.70*

$p_y = 275\ \text{N/mm}^2$ — *p_y from BS 5950 -1 Table 9*

$$A\,p_y = \frac{52.8 \times 10^2 \times 275}{10^3} = 1452\text{kN}$$

$F_t = 750\text{kN} < 1452\text{kN}$ — **∴ O.K**

8. COLUMN BASES

8. 1 INTRODUCTION

Typical column bases as shown in Figure 8.1 consist of a single plate fillet welded to the end of the column and attached to the foundation with four holding down bolts. The bolts are cast into the concrete base in location tubes or cones and are fitted with anchor plates to prevent pull-out. Bedding material is inserted in the space below the plate (see Figure 8.2).

Such column bases are usually assumed to be subject to axial compression and shear only. However, uplift should be considered for column bases in braced bays.

The base plate should be of sufficient size, stiffness and strength to transmit the axial compressive force from the column to the foundation through the bedding material, without exceeding the local bearing capacity of the foundation.

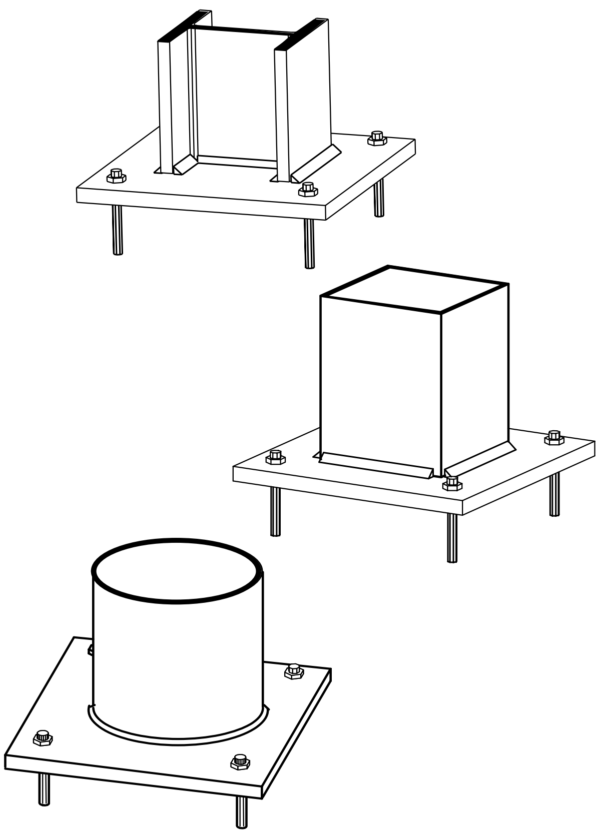


Figure 8.1 Typical column bases

Holding down systems are intended to meet the following requirements:

- In service they must transmit shear from the column to the foundation;
- during erection they must be capable of stabilising the column until other structural elements have been erected;
- during construction they must resist uplift;
- in service they must resist uplift in braced bays.

For most columns in simple multistorey construction this type of base is simple to fabricate and relatively inexpensive. Rarely will it be more economical to use thinner bases augmented by stiffeners, an arrangement that has been more popular in the past.

As with column splices, column bases fall Into two main categories:

- **Bearing type:** the loads are transferred in direct bearing from the column to the baseplate.
- **Non-bearing type:** the loads are transmitted via the welds which have to be designed accordingly.

The bearing type is the normal connection used today. The non-bearing type often requires a large amount of welding.

There is no difference in the design procedure for these two cases except that the weld between the column and the base plate must be checked for the vertical loads in the non-bearing case.

However, the two systems do have different implications for the Steelwork Contractor. Ends prepared for contact in bearing can be achieved by providing a sawn cut end for the column, and a base plate made from plates or wide flats up to 55mm thick. Such plates are normally assumed to have a flat surface and require no preparation for tight bearing. Material over this thickness may require machining The National Structural Steelwork Specificatlon[8] gives workmanship details and all tolerances for both the bearing and non-bearing cases.

The way in which horizontal shear forces are transferred to the foundation is uncertain. For further guidance see SCI-P207/95 [24] which suggests that generally, shear loads less than 30% of the axial load may be resisted by friction.

Column bases to braced bays may be required to deal with relatively high shear loads. An attempt[34] to establish a design approach for holding down systems produced no general consensus, except strong statements that no reliance should be placed on the bedding material to transmit shear force. However, SCI-P207/95[24] gives some design guidance on the topic.

This publication does not present a design method for transferring shear from the base plate to the foundation. It is noted however that it is common and successful industrial practice to use the holding down bolts of portal frames to resist the substantial shear forces that exist in that structural form.

For bearing surfaces, shear between the column end and the base plate can be transmitted by friction or the nominal weld between the column and the base plate. For non-bearing surfaces, the weld should be designed to transmit the shear.

8.2 PRACTICAL CONSIDERATIONS

Column shaft

The normal preparation for a bearing type connection is for the column section to be sawn square to its axis. A good quality saw in proper working order is adequate for this purpose. It must be emphasised that direct bearing does not necessitate the machining or end milling of the column ends. Guidance on the allowable tolerance between bearing surfaces can be found in BS 5950-2[1] or the National Structural Steelwork Specification[8].

Base plates

Base plates will usually be flame cut or sawn from S275 rather than S355 plate.

The portion of base plate in contact with the column shaft should be flat within a deviation of 0.75mm. It will be found that most plates have a sufficiently flat bearing surface without machining or cold pressing.

Welds

For bearing type bases the main function of the weld is to hold the column shaft securely in position on the base plate.

This being the case, only 6mm or 8mm fillet welds are generally required, usually run along the outside of the flanges and for a short distance either side of the web. Full profile welds will usually only be used for non-bearing bases or if additional strength is needed during erection or as an anti-corrosion measure.

Because of the thickness of the parts to be joined, consideration must be given to hydrogen cracking, which can be a problem during welding thick material. Table 8.1 gives values for the maximum combined plate thicknesses using a range of fillet welds and different types of electrode. As can be seen, as long as 8mm welds or low hydrogen electrodes are used, then cracking can be avoided without having to resort to the expense of pre-heating.

It should be noted that welding guidance rules given here refer to single pass manual metal arc welding only. More guidance on other welding procedures is given in BS EN 1011[13].

Holding down bolts

Holding down bolts are normally manufacturerd in accordance with BS 7419[35] which covers:

- bolts with square head and neck, and
- bolts with hexagon head and round neck.

For the square head type, bolt rotation during tightening is prevented by placing the square neck in a square hole in the anchor plate. For the hexagon head type, a small 'keep' flat is usually welded to the underside of the plate to bear against one of the hexagonal head flats.

The embedded length of the bolt in the concrete will usually be in the region of 16 to 18 bolt diameters. The thread length must allow for tolerances and should be 100mm plus the bolt diameter.

Holding down bolts are usually provided in grade 4.6 material. For lighter construction, M20 H.D bolts are often used, although M24 bolts are recommended for bases up to 50mm thick, increasing to M36 for plates over 50mm thick.

Although bolts are normally used in the non-coated condition, they can be supplied:

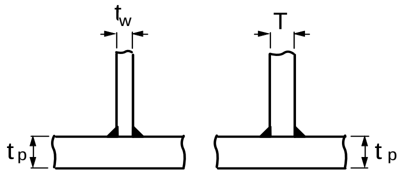
- electroplated to BS 7371-3[36];
- galvanized to BS 7371-6[37];
- sherardized to BS 7371-8[38].

Dimensions of holding down bolts to BS 7419[35] are Included with the detailing information in the yellow pages, table H.63

Table 8.1 Maximum combined thicknesses for manual metal arc welding to avoid preheating

Steel grade*	Max C E V	Fillet weld size	Max combined thickness (mm) to avoid preheat relative hydrogen scale (see BS EN 1011)[13] A	B	C	D
S275	0.40	6	70	∞	∞	∞
		8	∞	∞	∞	∞
S355	0.45	6	50	60	70	∞
		8	70	∞	∞	∞

* *the higher grade of the base plate and the column*



t_p = Base plate thickness

t_w = Column web thickness

T = Column flange thickness

Combined thickness = $2t_p + t_w$ or

= $2t_p + T$

Notes: *1* *The table covers fillet welds made using single run welds.*

2 *Electrodes are to BS EN 499: 1995[39] coating types R and B only.*

3 *For other Carbon Equivalent Values (CEV) and other processes refer to BS EN 1011[13].*

4 *If combined thickness is greater than given in the table, preheat will be required in accordance with BS EN 1011.*

5 *∞ Signifies that there is no limit on the combined thickness.*

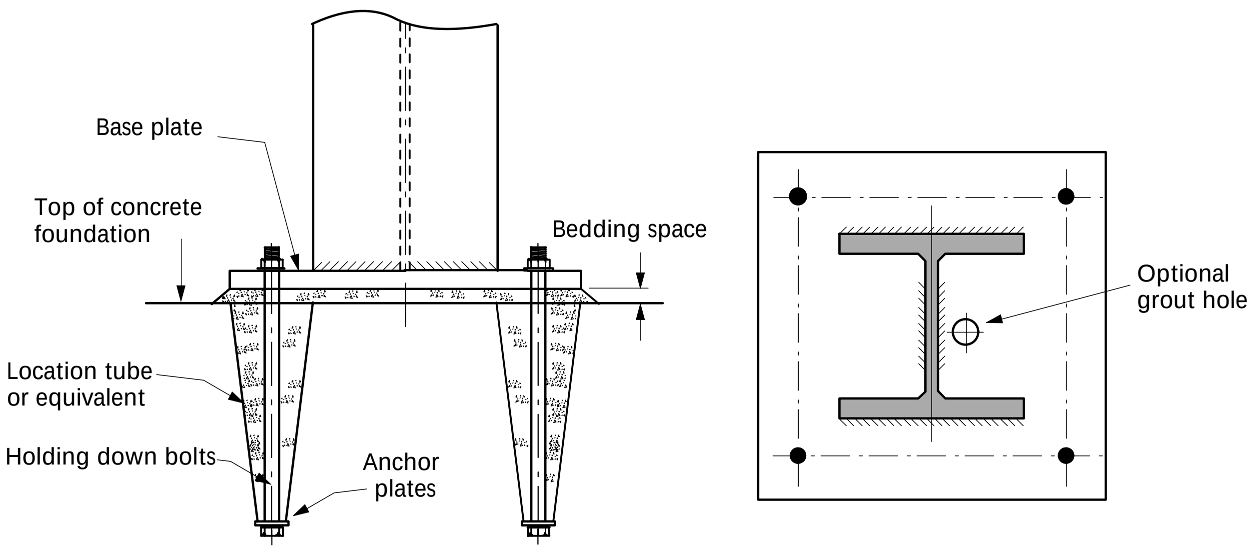


Figure 8.2 Column base holding down bolts

Clearance holes, washers

Clearance holes in the base plate should be 6mm larger than the bolt diameter to allow for adjustment, and for bases thicker than 60mm this figure may need to be increased.

Form G extra large washers to BS 4320[40] are needed under the nuts, or alternatively washers can be made from plate.

Location tubes

Normally the bolts are set into location tubes or cones (usually polystyrene or cardboard). The diameter of the tube or the tops of the cones should be at least 100mm or 3 times the diameter of the bolt, whichever is greater.

Bolts are occasionally cast solidly into the concrete. However this is not recommended as a general practice as great accuracy is needed and it gives no allowance for site adjustment.

Base packs

Columns are normally erected on central steel levelling packs. Steel wedges placed around the edges of the base plate are also necessary to ensure stability during erection. See BS 5531[41], and GS28[30].

Table 8.2 Strength of bedding material

Bedding Material	Characteristic cube strength at 28 days f_{cu} (N/mm^2)
Mortar	20.0 - 25.0
Fine Concrete	30.0 - 50.0

Note: *For design purposes, the lower values should be used, unless larger values can be justified by test.*

Table 8.3 Concrete strengths

Concrete grade	Characteristic cube strength at 28 days. f_{cu} (N/mm^2)	Bearing strengths (N/mm^2) Eff. Area method (BS 5950-1: 2000) $0.6f_{cu}$	Bearing strengths (N/mm^2) BS 5950: part 1:1990 method $0.4f_{cu}$
C25	25.0	15	10
C30	30.0	18	12
C35	35.0	21	14
C40	40.0	24	16

Bedding space, grout

A bedding space of at least 50mm is the normal allowance when using high strength bedding material. This gives reasonable access for grouting the bolt pockets, which is necessary to prevent corrosion, and for thoroughly filling the space under the base plate. It also makes a reasonable allowance for tolerances. For smaller, more lightly loaded bases a gap of 25 to 50mm will be found to be adequate.

In slab bases of size 700mm x 700mm or larger, 50mm diameter grout holes should be provided to allow trapped air to escape and also for inspection. A hole should be provided for each 1/2 m^2 (5000 cm^2) of base area. If it is expected to place grout through these holes then the diameter should be increased to 100mm.

Normal practice is to choose a bedding material at least equal in strength to that of the concrete foundation. It will consist either of mortar, fine concrete or perhaps one of the proprietary non-shrink grouts which are readily available.

Table 8.2 gives typical cube strengths for mortar and fine concrete and is taken from 'Holding Down Systems for Steel Stanchions' [34].

The strength of fine concrete depends on many factors such as the quality of the mix and degree of compaction. By using hammered or dry packed fine concrete, cube strengths up to 50N/mm^2 can be achieved.

For the concrete foundations, Table 8.3 gives values of characteristic cube strengths and bearing strengths and is based on the grades of concrete given in BS 5328-1[42].

8.3 RECOMMENDED GEOMETRY

No specific detailing requirements for column bases can be provided, although in practice there are two main considerations:

- the plate dimensions must be sufficient to spread the design loads to the foundation and to accommodate the holding down bolts;
- the base and holding down system must be sufficiently robust to withstand loads experienced during erection resulting from wind loads, lack of verticality and asymmetric loading.

In view of this, normal practice is for the base to be at least 100mm larger all round than the column, with a thickness greater than or equal to that of the column flange and with four holding down bolts positioned outside the section.

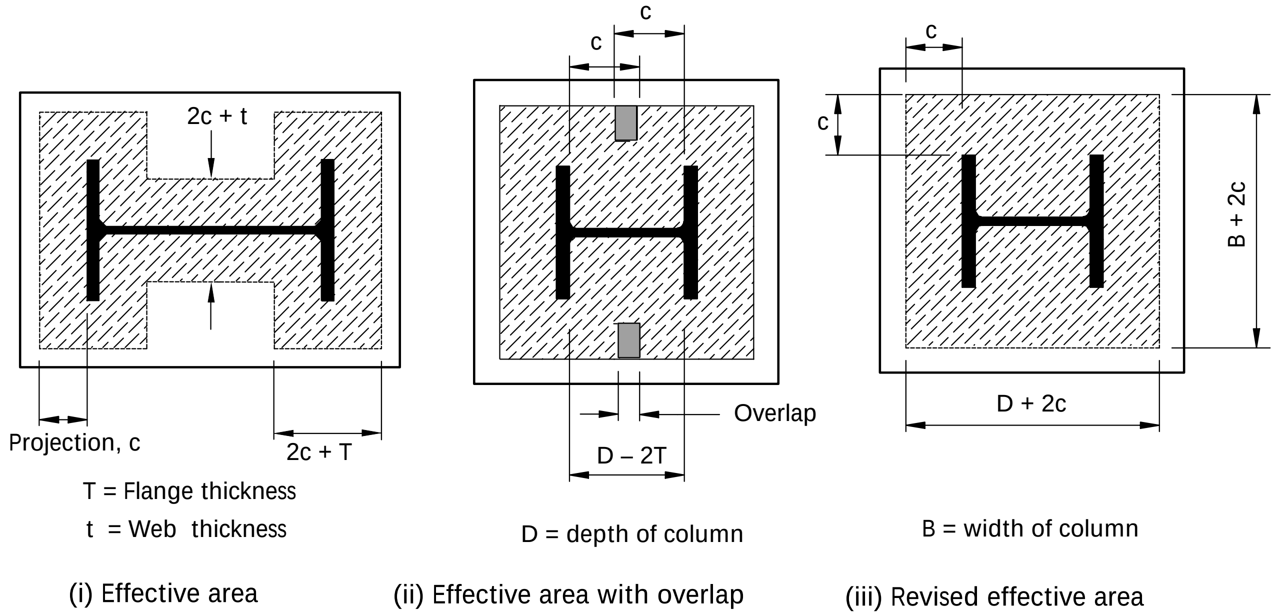


Figure 8.3 Calculated effective area for a rolled section

8.4 DESIGN

Two design procedures are given for slab column bases:

- the effective area method. (BS5950-1:2000)
- the BS 5950: Part 1:1990 method

The effective area method is marginally more efficient over the full range of rolled sections used as columns and the design procedure is fully described in Section 8.5.

Although not in the current standards, the BS 5950: Part 1:1990 method is dealt with in Appendix E. This method should only be used to obtain the initial plate dimensions.

Column bases subject to both axial load and an overturning moment have to take into account the effect of tension on one side of the base. This treatment can be found in *Moment Connections*[24] which is another design guide in this series.

Effective area method

In this method it is assumed that the bearing pressure on the effective area is uniform and that the plate acts as a simple cantilever around the perimeter of the section. The effective area is illustrated in Figure 8.3 with respect to a rolled **I** section.

The projection width c, shown in (i) is the minimum that is needed to keep the base pressure below the limiting bearing strength which in this case is taken as 0.6 f_{cu}, where f_{cu} is the characteristic cube strength of the concrete base or the bedding material, whichever is less (See Tables 8.2 and 8.3).

In some circumstances, it can be found that the projection, c becomes so large that the strips overlap between the column flanges as shown in (ii).

i.e. $c > (D-2T)/2$

This clearly can not be allowed to happen and c must be therefore recalculated on the basis of the effective area shown in (iii).

The design is similar for the slab bases for tubular columns and is illustrated for RHS and CHS columns in Figure 8.4. Of course, if the internal projection overlaps the centre of the plate, a readjusted effective area must be recalculated in a similar manner to that for the rolled section.

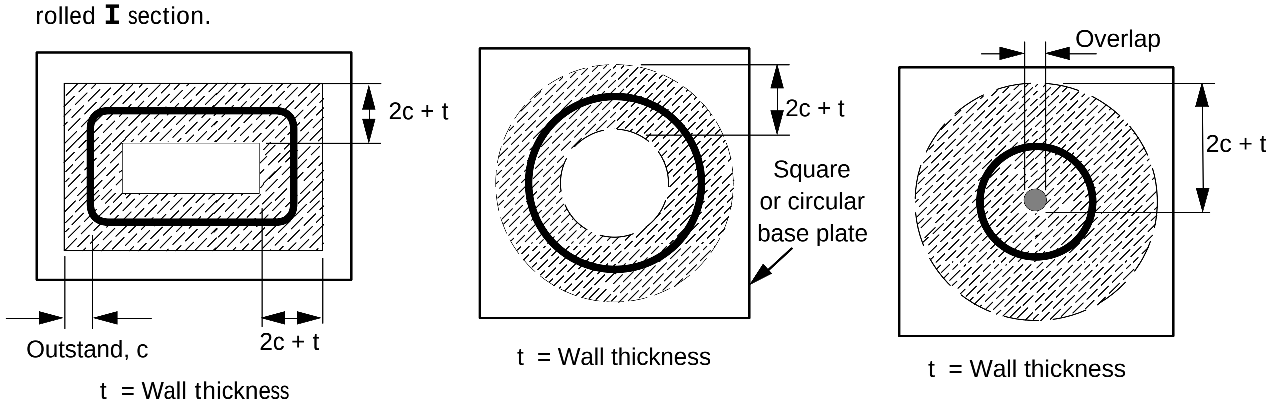


Figure 8.4 Calculated effective area for RHS & CHS sections

The effective area method, does not have the limitations of BS 5950: Part 1:1990 on plate thickness ≥ column flange thickness and plate design strength $p_{yp} \leq 270$ N/mm^2. The bearing strengths for the two methods are 0.6 f_{cu} and 0.4 f_{cu}. Typical values for grout and concrete strengths are given in Tables 8.2 and 8.3.

Although the shaded area represents the size of the base plate theoretically required, the overall size of the plate can be made larger, e.g. to utilize rounded dimensions, to accommodate holding down bolts etc. or square plates.

Worked examples

Four design examples are provided in section 8.6 to illustrate the design checks for the effective area method.

Baseplate capacity tables

Four sets of capacity tables for S275 base plates which are for UC columns, CHS columns, Square and rectangular hollow section columns are provided in the yellow pages (Tables H.41, H.42, H.43 and H.44). They are based on the effective area method, as described in Section 8.5.

8.5 DESIGN PROCEDURES

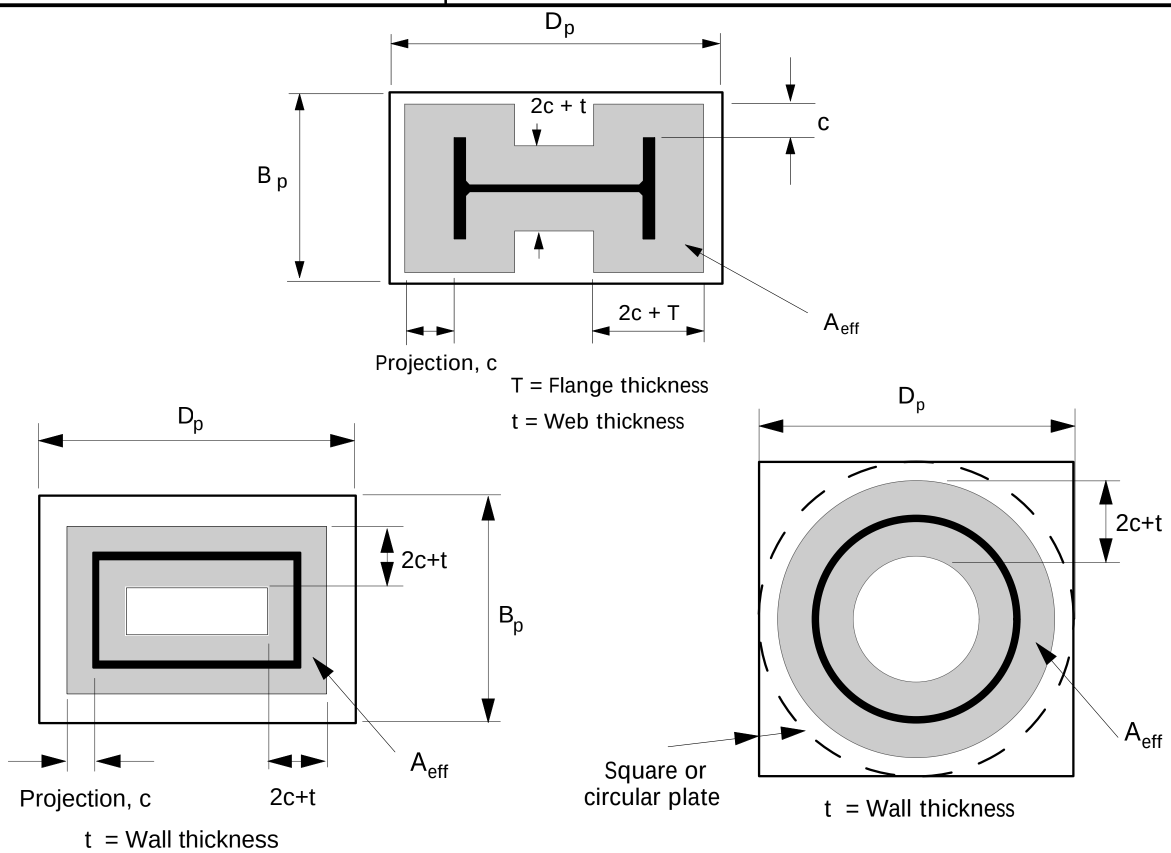
The design models shown in Figure 8.3 are appropriate for **I** sections, and Figure 8.4 are appropriate for RHS and CHS sections. Detail design checks are as follows:

CHECK 1 - Effective area method - base plate size

CHECK 2 - Effective area method - base plate thickness

CHECK 3 - Base plate weld

CHECK 1	Effective area method - Base plate size



Note: A_{eff} is the effective area of base plate developing the load

Basic requirement:

$A_p \geq A_{req}$

A_p = area of base plate $= B_p\, D_p$ for rectangular plate

$= \dfrac{\pi\, D_p^{\,2}}{4}$ for circular plate

A_{req} = required area of base plate $= \dfrac{F_c}{0.6\, f_{cu}}$

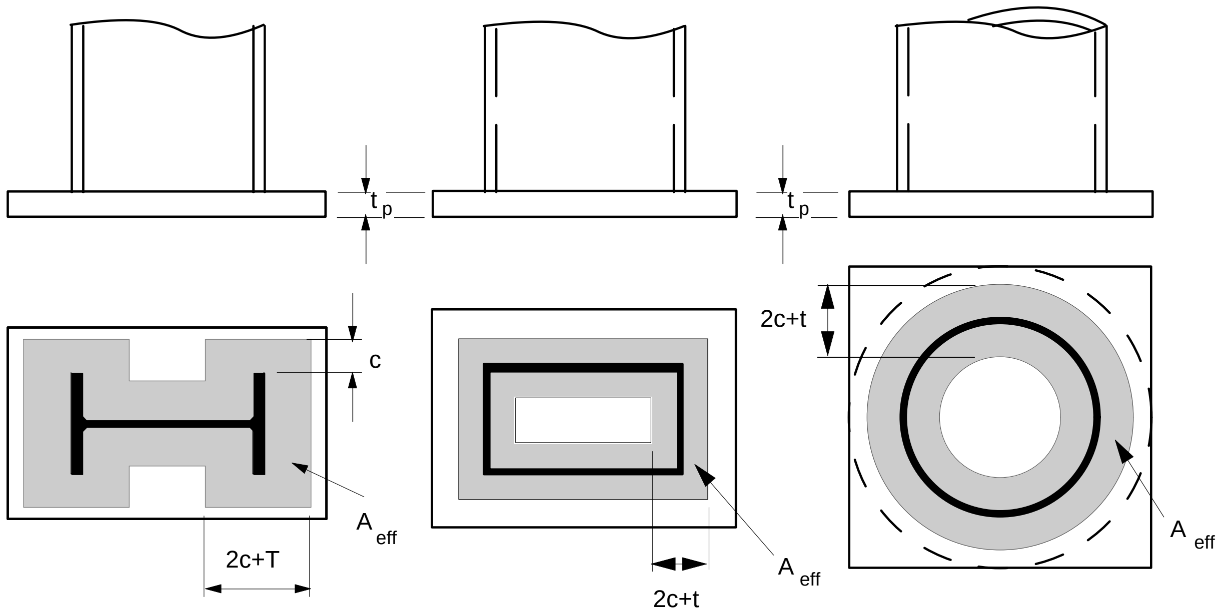
Where

F_c = compressive force due to factored loads

$0.6\, f_{cu}$ = bearing strength

f_{cu} = the smaller of the characteristic cube strength at 28 days of the bedding material or the concrete base

CHECK 2	Effective area method - Base plate thickness



Basic requirement:

$$t_p \geq c \left(\frac{3\,(0.6 f_{cu})}{p_{yp}} \right)^{1/2}$$

t_p = thickness of the base plate

Where:

p_{yp} = design strength of base plate

$0.6\,f_{cu}$ = bearing strength

f_{cu} = the smaller of the characteristic cube strength at 28 days of the bedding material or the concrete base

The projection c is determined by equating A_{req} to A_{eff} and solving a quadratic equation. Provided there is no "overlap", c may be calculated from the following equations.

For UC, UB $A_{eff} = 4c^2 + c\,(Per_{col}) + A_{col}$

For RHS column A_{eff} = (mean wall perimeter length) $\times$ $(t + 2c)$

For CHS column $A_{eff} = \pi\,(D - t) \times (t + 2c)$

Where:

A_{req} = required area of base plate (from Check 1)

A_{col} = cross sectional area of the column

Per_{col} = column perimeter

F_c = compressive force due to factored loads

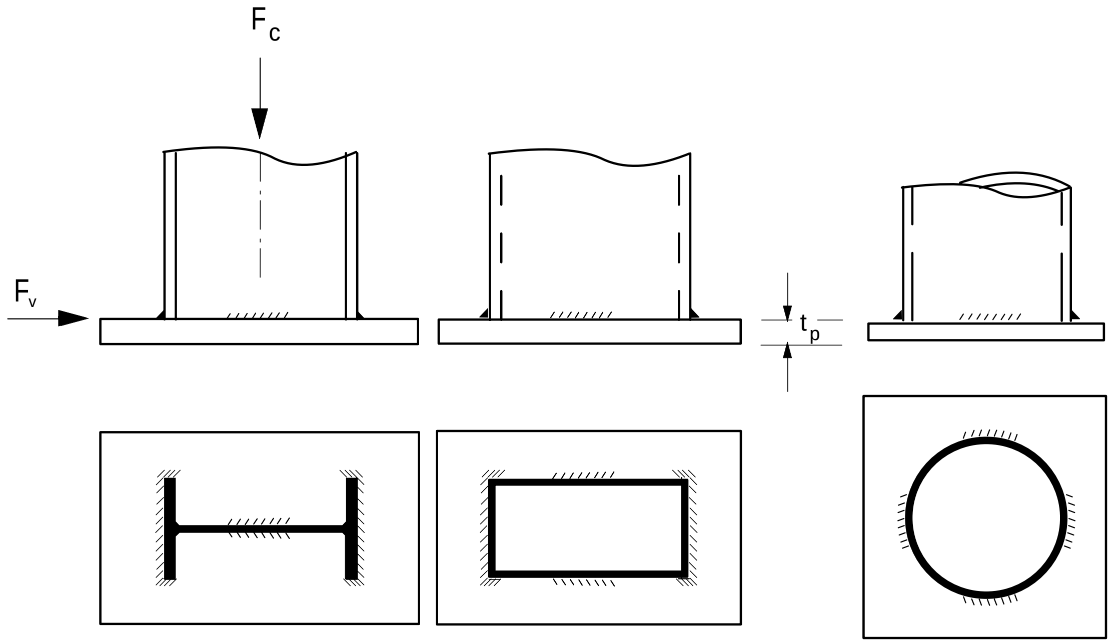
$0.6\,f_{cu}$ = bearing strength

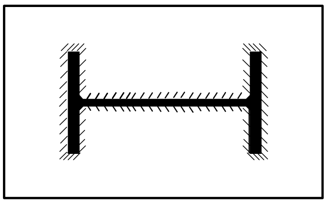
f_{cu} = the smaller of the characteristic cube strength at 28 days of the bedding material or the concrete base

D = outside diameter of CHS

Note: The effective area must be checked for overlap and any necessary adjustment made.

CHECK 3	Base plate welds





Provide full perimeter welds when F_c must be developed in the weld

Basic requirement:

For shear

$F_v \leq P_{weld}$

P_{weld} = capacity of fillet weld to resist shear = $p_w\, l_{wew}\, a$

p_w = design strength of the weld

l_{wew} = total effective length of the welds in direction of shear

a = effective throat size of the weld = 0.7s (normally)

s = weld leg length

For axial load

(This check is only necessary when the contact faces of the column and base plate are not in tight bearing - see section 8.1).

$F_c \leq P_{weld}$

F_c = compressive force due to factored loads

P_{weld} = capacity of fillet weld = $p_w\, l_{wef}\, a$

l_{wef} = total effective length of the welds to the column flange for rolled sections

= total effective length of the weld for RHS and CHS sections

8.6 WORKED EXAMPLES

The four worked examples for column bases illustrate the design checks required for the most commonly used details:

Example 1: The design of a UC column base, using the effective area method, where the column is in direct bearing but welds must develop the shear force at the base. In this design the effective area does not cause overlap.

Example 2: A connection similar to Example 1 but with a higher axial load where the effective area calculation produces overlap and a recalculation of outstand 'c' has to be made.

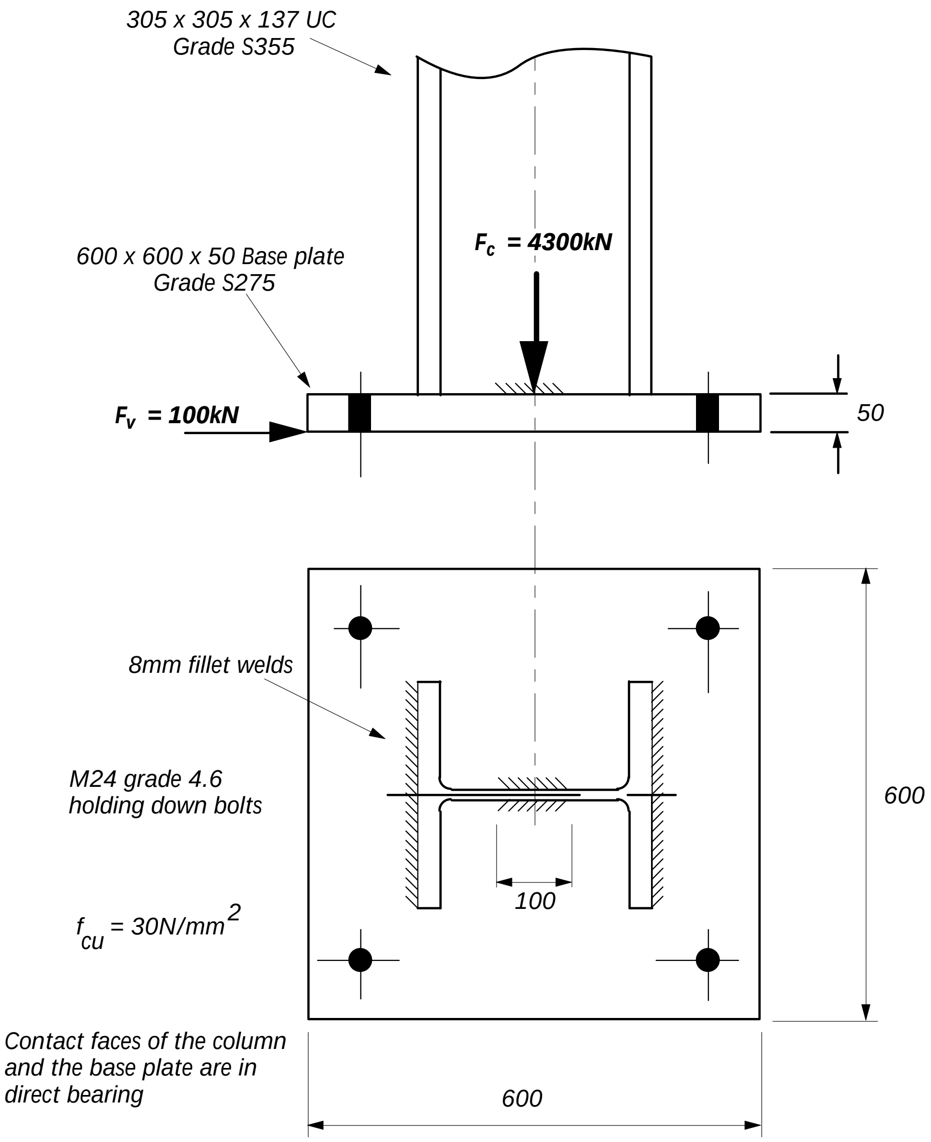
Example 3: The design of a RHS column base, using the effective area method, where the column is in direct bearing but welds must develop the shear force at the base.

Example 4: The design of a CHS column base, using the effective area method, where the column is in direct bearing but welds must develop the shear force at the base.

CALCULATION SHEET BCSA	Job No Joints in Steel Construction - Simple Connections		Sheet 1 of 4
	Title Example 1 - Column Base - UC, no overlap		
	Client SCI/BCSA Connections Group		
	Calcs by RS	Checked by AM	Date May 2002

DESIGN EXAMPLE 1

Check the column base for the design forces shown.



Title *Example 1 - Column Base - UC, no overlap* | Sheet *2 of 4*

CAPACITY USING TABLES FROM THE YELLOW PAGES

For a 600 x 600 x 50 baseplate with C30 concrete, — *Yellow pages Table H.41*

Capacity given in tables = 4620kN > 4300kN — **∴ O.K.**

Therefore the base is adequate.

CHECK 1: Effective area method - base plate size.

Basic requirement: $\boldsymbol{A_p \geq A_{req}}$

Area of baseplate $A_p = B_p D_p = 600 \times 600 = 360000mm^2$

Area required, $A_{req} = \dfrac{F_c}{0.6 f_{cu}}$

$= \dfrac{4300 \times 10^3}{0.6 \times 30}$

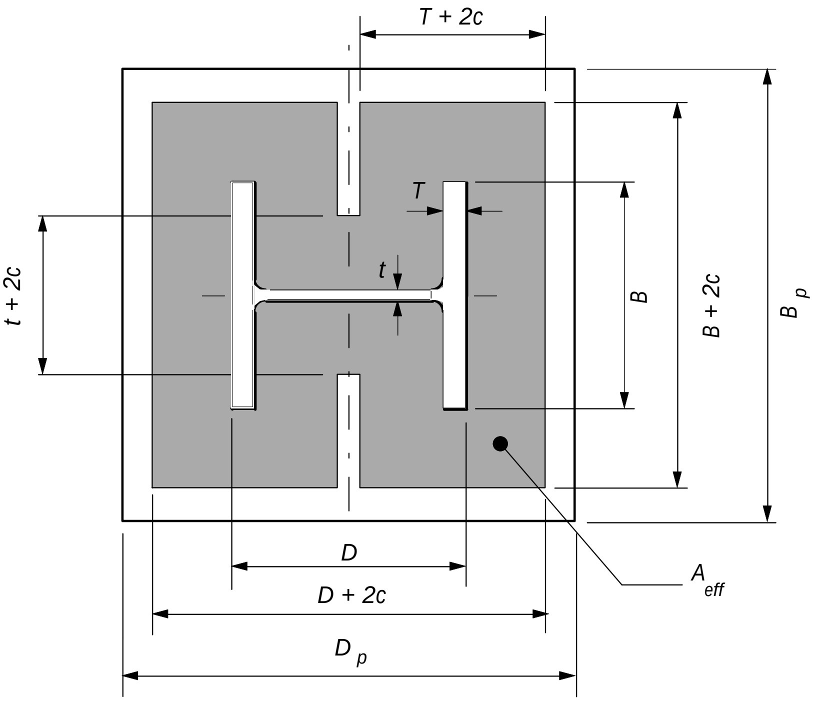
$= 238900mm^2$

$\boldsymbol{A_p = 360000mm^2 > 238900mm^2}$ — **∴ O.K.**

Title Example 1 - Column Base - UC, no overlap	Sheet 3 of 4

CHECK 2: Effective area method - base plate thickness

Basic requirement: $t_p \geq c\left(\frac{3 \times 0.6 f_{cu}}{p_{yp}}\right)^{1/2}$



Calculation of c

(i) Assuming no overlap

Effective Area A_{eff} (shaded)

$A_{eff} = 4c^2 + c(Per_{col}) + A_{col}$

Column perimeter $Per_{col} \approx 1820mm$

Area of column $A_{col} = 17400mm^2$

Per_{col} from Surface area per metre, Yellow pages Table H.65

Equating the effective area, A_{eff} to the required area, A_{req} (from Check 1) gives

$4c^2 + 1820c + 17400 = 238900$

$\therefore c = 99.8mm$

$$\frac{D - 2T}{2} = \frac{320.5 - (2 \times 21.7)}{2} = 138.6mm$$

Since $\frac{D - 2T}{2} = 138.6mm > 99.8mm$

Therefore **assumption of no overlap is correct.**

Also, check that effective area fits on the baseplace.

$D + 2c = 320.5 + (2 \times 99.8) = 520mm < D_p = 600mm$

$B + 2c = 309.2 + (2 \times 99.8) = 509mm < B_p = 600mm$

Hence the calculated value of c above is valid. (Otherwise recalculate c)

$\therefore$ **c = 99.8mm** **$\therefore$ O.K.**

Design strength of the 50mm plate, $p_{yp} = 255N/mm^2$ *BS 5950-1 Table 9*

$$c\left(\frac{3 \times 0.6 f_{cu}}{p_{yp}}\right)^{1/2} = 99.8\left(\frac{3 \times 0.6 \times 30}{255}\right)^{1/2} = 45.9mm$$

$\therefore$ **t_p = 50mm > 45.9mm** **$\therefore$ O.K.**

Title *Example 1 - Column Base - UC, no overlap* | Sheet *4 of 4*

CHECK 3 : Baseplate welds (Shear capacity of column-to-base weld)

Basic requirement: $F_v \leq P_{weld}$

Shear capacity of fillet welds, $P_{weld} = p_w \, l_{wew} \, a$

Effective length of welds to column web, l_{wew} *(i.e. in direction of shear)*

$$l_{wew} = 2\,(l - 2s)$$

$$= 2\,\big(100 - (2 \times 8)\big)$$

$$= 168\text{mm}$$

Design strength of the weld p_w

$$p_w = 220 \text{ N/mm}^2$$

BS 5950-1 Table 37

Throat size, a

$$a = 0.7s$$

$$= 0.7 \times 8$$

$$= 5.6\text{mm}$$

$$P_{weld} = \frac{220 \times 168 \times 5.6}{10^3} = 207\text{kN}$$

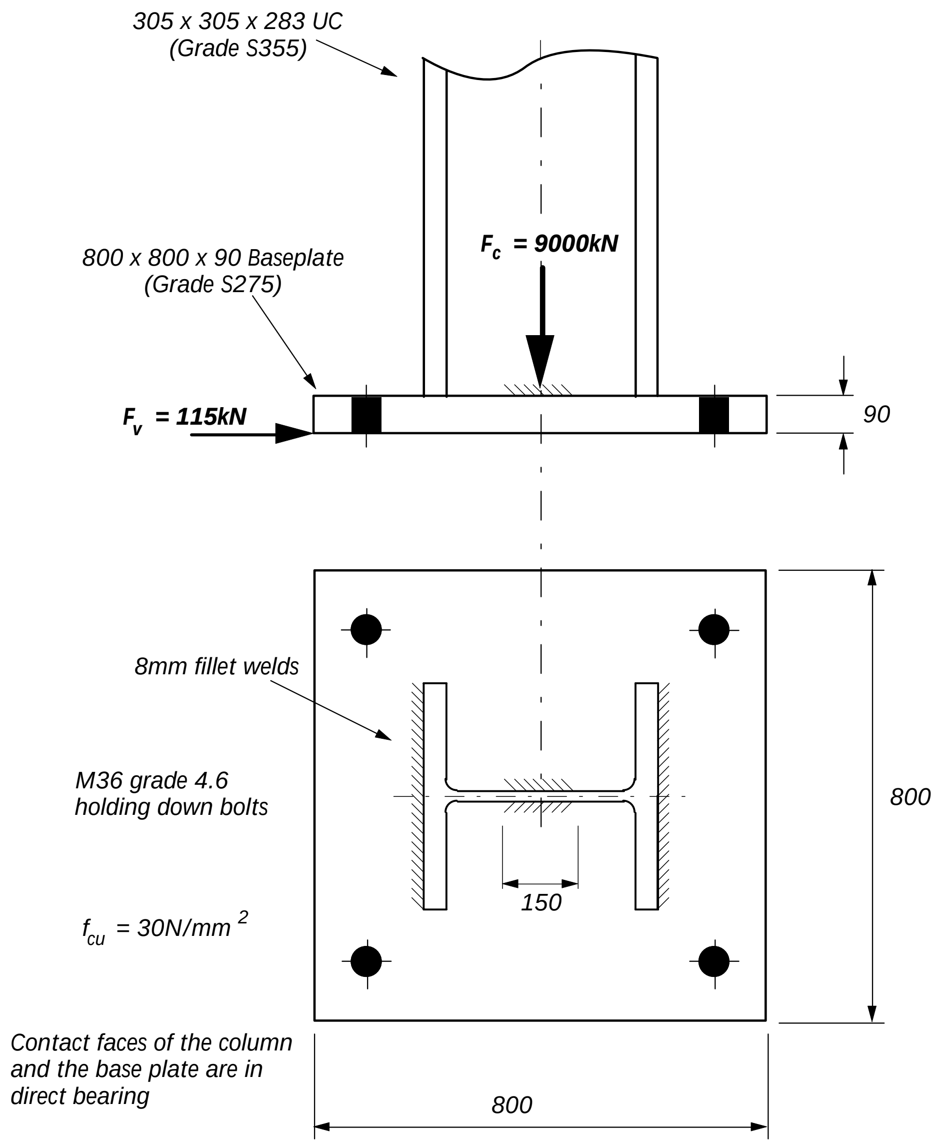
$$\mathbf{F_v = 100kN < 207kN}$$

∴ O.K.

CALCULATION SHEET BCSA	Job No Joints in Steel Construction - Simple Connections		Sheet 1 of 4
	Title Example 2 - Base Plates - UC, overlap		
	Client SCI/BCSA Connections Group		
	Calcs by RS	Checked by AM	Date May 2002

DESIGN EXAMPLE 2

Check the column base for the design forces shown.



Title *Example 2 - Base Plates - UC, overlap*	Sheet *2 of 4*

CAPACITY USING TABLES FROM THE YELLOW PAGES

For a 800 x 800 x 90 baseplate with C30 concrete, — *Yellow pages Table H.41*

Capacity given in tables = 8370kN $\ngtr$ 9000kN — **∴ Fails**

The capacity tables indicate that the base is inadequate for the design load given. However, the table values are calculated using the smallest section within a serial size and are therefore conservative for larger weight columns. Thus, a higher capacity may be obtained if the design is carried out manually for the particular column size in question. — *Table H.39 note (2)*

CHECK 1: Effective area method – base plate size.

Basic requirement, $A_p \geq A_{req}$

Area of baseplate $A_p = B_p D_p = 800 \times 800 = 640000\text{mm}^2$

Area required, $A_{req} = \dfrac{F_c}{0.6 f_{cu}}$

$$= \frac{9000 \times 10^3}{0.6 \times 30}$$

$$= 500000\text{mm}^2$$

$A_p = 640000\text{mm}^2 > 500000\text{mm}^2$ — **∴ O.K.**

Title Example 2 - Base Plates - UC, overlap	Sheet 3 of 4

Check 2: Effective area method – baseplate thickness

Basic requirement: $t_p \geq c\left(\frac{3 \times 0.6 f_{cu}}{p_{yp}}\right)^{1/2}$

Calculation of c

See Example 1 (Figure in check 2)

(i) Assuming no overlap

$A_{eff} = 4c^2 + c(Per_{col}) + A_{col}$

Column perimeter $Per_{col} \approx 1940mm$

Area of column $A_{col} = 36000mm^2$

Yellow pages Table H.65

Equating the effective area, A_{eff} to the required area, A_{req} (from Check 1) gives

$4c^2 + 1940c + 36000 = 500000$

$\therefore c = 175.6mm$

$$\frac{D - 2T}{2} = \frac{365.3 - (2 \times 44.1)}{2} = 138.6mm$$

Since $\frac{D - 2T}{2} = 138.6mm \ngtr 175.6mm$

Therefore **assumption of no overlap is INCORRECT**

(ii) Assuming overlap

Recalculate c on the basis of a revised effective area as shown in Figure 8.3(iii)

$A_{eff} = (D + 2c)(B + 2c)$

$= 4c^2 + 2(D + B)c + DB$

Equating the effective area, A_{eff} to the required area, A_{req} (from Check 1) gives

$4c^2 + 2(365.3 + 322.2)c + 365.3 \times 322.2 = 500000$

$\therefore c = 182mm$

Also check that effective area fits on the baseplate

$D + 2c = 365.3 + (2 \times 182) = 729.3mm < D_p = 800mm$

$B + 2c = 322.2 + (2 \times 182) = 686.2mm < B_p = 800mm$

hence the calculated value of c in (ii) above is valid.

$\boldsymbol{\therefore c = 182mm}$ **∴ O.K.**

Design strength of the 90mm plate

BS 5950-1 Table 9

$p_{yp} = 235N/mm^2$

$$c\left(\frac{3 \times 0.6 f_{cu}}{p_{yp}}\right)^{1/2} = 182\left(\frac{3 \times 0.6 \times 30}{235}\right)^{1/2} = 87.2mm$$

$\boldsymbol{t_p = 90mm > 87.2mm}$ **∴ O.K.**

Title *Example 2 - Base Plates - UC, overlap*	Sheet *4 of 4*

CHECK 3: Baseplate welds (Shear capacity of column-to-baseplate weld)

Basic requirement: $F_v \leq P_{weld}$

Shear capacity of fillet welds, $P_{weld} = p_w \, l_{wew} \, a$

Effective length of welds to column web, l_{wew} *(i.e. in direction of shear)*

$$l_{wew} = 2\,(l - 2s)$$

$$= 2\,(150 - (2 \times 8))$$

$$= 268\text{mm}$$

Design strength of the weld, p_w

$$p_w = 220 \text{ N/mm}^2$$

BS 5950-1 Table 37

Throat size, a

$$a = 0.7s$$

$$= 0.7 \times 8$$

$$= 5.6\text{mm}$$

$$P_{weld} = \frac{220 \times 268 \times 5.6}{10^3} = 330\text{kN}$$

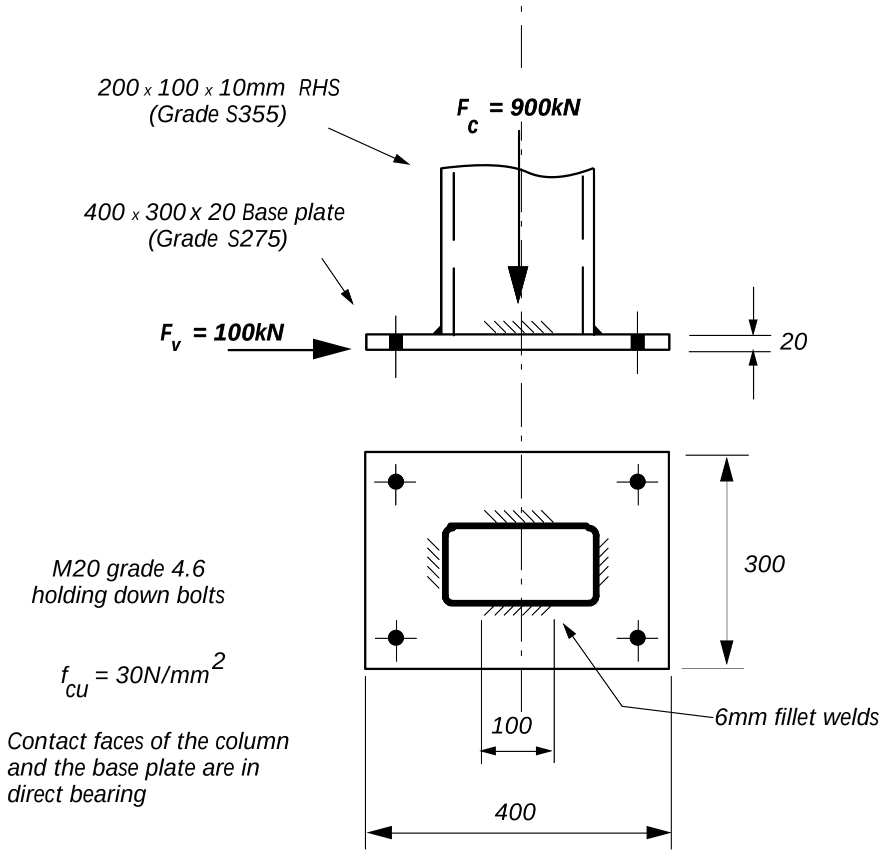
$$\mathbf{F_v = 115kN < 330kN}$$

∴ O.K.

CALCULATION SHEET BCSA	Job No Joints in Steel Construction - Simple Connections	Sheet 1 of 4
	Title Example 3 - Column Base - RHS	
	Client SCI/BCSA Connections Group	
	Calcs by RS | Checked by AM	Date May 2002

DESIGN EXAMPLE 3

Check the RHS column base for the design forces shown.



Title	Sheet
Example 3 - Column Base - RHS	*2 of 4*

CAPACITY USING TABLES FROM THE YELLOW PAGES

Yellow pages Table H.44

For a 400 x 300 x 20 baseplate with C30 concrete,

Capacity = 977kN > 900kN **∴ O.K.**

Therefore the base is adequate.

CHECK 1: Effective area method - base plate size.

Basic requirement: $A_p \geq A_{req}$

Area of baseplate $A_p = B_p D_p = 300 \times 400 = 120000mm^2$

Area required, $A_{req} = \dfrac{F_c}{0.6\, f_{cu}}$

$= \dfrac{900 \times 10^3}{0.6 \times 30}$

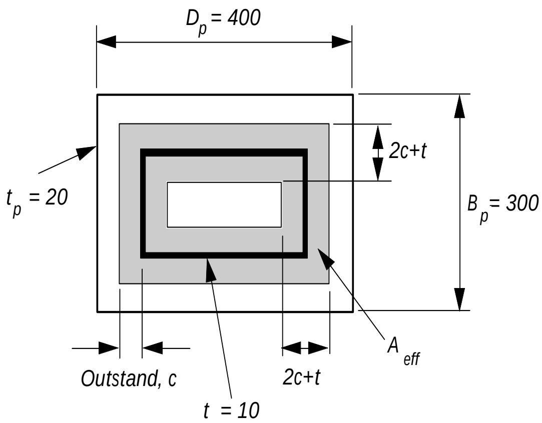
$= 50000mm^2$

$\mathbf{A_p = 120000mm^2 > 50000mm^2}$ **∴ O.K.**

Title Example 3 - Column Base - RHS	Sheet 3 of 4

CHECK 2: Effective area method - base plate thickness.

Basic requirement: $t_p \geq c\left(\frac{3 \times 0.6f_{cu}}{p_{yp}}\right)^{1/2}$



Calculation of c

(i) Assuming no overlap

Effective Area A_{eff} (shaded)

A_{eff} = (mean perimeter of RHS) x (t + 2c)

= 2(D + B - 2t) x (t + 2c)

Equating the effective area, A_{eff} to the required area, A_{req} (from Check 1) gives

$2(200 + 100 - 20) \times (10 + 2c) = 50000$

$5600 + 1120c = 50000$

$\therefore c = 39.6mm$

$$\frac{B - 2t}{2} = \frac{100 - (2 \times 107)}{2} = 40mm$$

Since $\frac{B - 2T}{2} = 40mm > 39.6mm$

Therefore **assumption of no overlap is correct.**

Also, check that effective area fits on the baseplate.

$D + 2c = 200 + (2 \times 39.6) = 279.2mm < D_p = 400mm$

$B + 2c = 100 + (2 \times 39.6) = 179.2mm < B_p = 300mm$

Hence the calculated value of c above is valid. (Otherwise recalculate c)

$\therefore$ **c = 39.6mm** **$\therefore$ O.K.**

Design strength of the 20mm plate, p_{yp} = 265N/mm^2 — BS 5950-1 Table 9

$$c\left(\frac{3 \times 0.6f_{cu}}{p_{yp}}\right)^{1/2} = 39.6\left(\frac{3 \times 0.6 \times 30}{265}\right)^{1/2} = 17.9mm$$

$\therefore$ **t_p = 20mm > 17.9mm** **$\therefore$ O.K.**

Title *Example 3 - Column Base - RHS*	Sheet *4 of 4*

CHECK 3: Baseplate welds (Shear capacity of column-to-base weld)

Basic requirement: $F_v \leq P_{weld}$

Shear capacity of fillet welds, $P_{weld} = p_w \; l_{wew} \; a$

Effective length of welds to column, l_{wew} *(i.e. in direction of shear)*

$$l_{wew} = 2\,(l - 2s)$$

$$= 2\,\big(100 - (2 \times 6)\big)$$

$$= 176\text{mm}$$

Design strength of the weld p_w

$$p_w = 220\ \text{N/mm}^2$$

BS 5950-1 Table 37

Throat size, a

$$a = 0.7s$$

$$= 0.7 \times 6$$

$$= 4.2\text{mm}$$

$$\therefore P_{weld} = \frac{220 \times 176 \times 4.2}{10^3} = 163\text{kN}$$

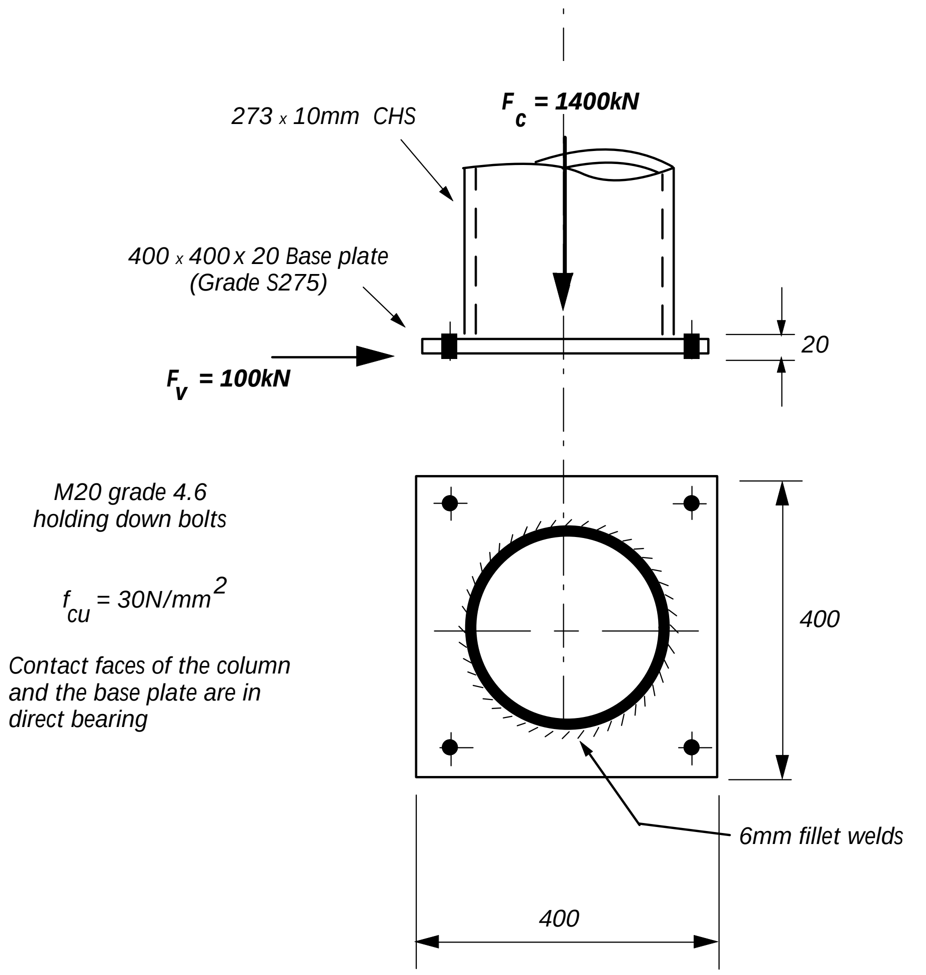
$$\boldsymbol{F_v = 100\text{kN} < 163\text{kN}}$$

∴ O.K.

CALCULATION SHEET BCSA	Job No Joints in Steel Construction - Simple Connections		Sheet 1 of 4
	Title Example 4 - Column Base - CHS		
	Client SCI/BCSA Connections Group		
	Calcs by RS	Checked by AM	Date May 2002

DESIGN EXAMPLE 4

Check the CHS column base for the design forces shown.



Title *Example 4 - Column Base - CHS*	Sheet *2 of 4*

CAPACITY USING TABLES FROM THE YELLOW PAGES

For a 400 x 400 x 20 baseplate with C30 concrete, — *Yellow pages Table H.42*

Capacity = 1420kN > 1400kN

Therefore the base is adequate. — **∴ O.K.**

CHECK 1: Effective area method - base plate size.

Basic requirement: $A_p \geq A_{req}$

Area of baseplate $A_p = B_p D_p = 400 \times 400 = 160000\text{mm}^2$

Area required, $A_{req} = \dfrac{F_c}{0.6 f_{cu}}$

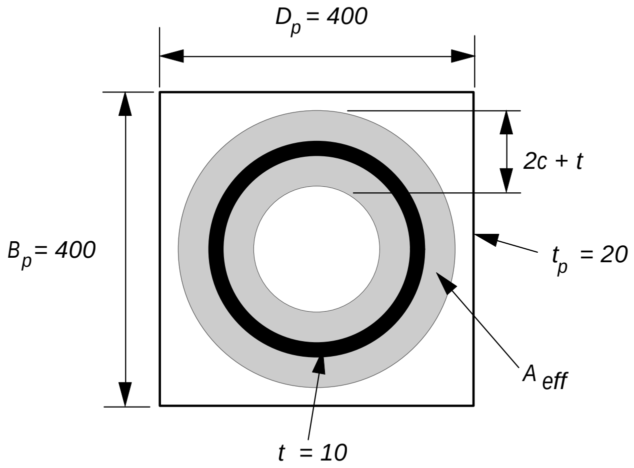
$= \dfrac{1400 \times 10^3}{0.6 \times 30}$

$= 77778\text{mm}^2$

$A_p = 160000\text{mm}^2 > 77778\text{mm}^2$ — **∴ O.K.**

CHECK 2: Effective area method - base plate thickness.

Basic requirement: $t_p \geq c\left(\frac{3\ 0.6f_{cu}}{p_{yp}}\right)^{1/2}$



Calculation of c

(i) Assuming no overlap

Effective Area A_{eff} (shaded)

$$A_{eff} = \pi(D - t) \times (t + 2c)$$

Equating the effective area, A_{eff} to the required area, A_{req} (from Check 1) gives

$$\pi(273 - 10) \times (10 + 2c) = 77778$$

$$8262 + 1652c = 77778$$

$$\therefore c = 42.1\text{mm}$$

$$\frac{D - 2t}{2} = \frac{273 - (2 \times 10)}{2} = 126.5\text{mm}$$

$$\text{Since } \frac{D - 2t}{2} = 126.5\text{mm} > 42.1\text{mm}$$

Therefore ***assumption of no overlap is correct.***

Also, check that effective area fits on the baseplate.

$$D + 2c = 273 + (2 \times 42.1) = 357.2\text{mm} < D_p = 400\text{mm}$$

Hence the calculated value of c above is valid. (Otherwise recalculate c)

$$\therefore c = \mathbf{42.1mm}$$ **∴ O.K.**

BS 5950-1 Table 9

Design strength of the 20mm plate, p_{yp} $= 265\text{N/mm}^2$

$$c\left(\frac{3 \times 0.6f_{cu}}{p_{yp}}\right)^{1/2} = 42.1\left(\frac{3 \times 0.6 \times 30}{265}\right)^{1/2} = 19.0\text{mm}$$

$$t_p = \mathbf{20mm} > \mathbf{19.0mm}$$ **∴ O.K.**

Title *Example 4 - Column Base - CHS*	Sheet *4 of 4*

CHECK 3: Baseplate welds (Shear capacity of column-to-base weld)

Basic requirement, $F_v \leq P_{weld}$

Shear capacity of fillet welds, P_{weld}

$$= p_w \, l_{wew} \, a$$

Effective length of welds in direction of shear, l_{wew}

$$l_{wew} = 2 \times \frac{\pi D}{4}$$

$$= 2 \times \frac{\pi \times 273}{4} = 429\text{mm}$$

Design strength of the weld p_w

$$p_w = 220 \text{ N/mm}^2$$ *BS 5950-1 Table 37*

Throat size, a

$$= 0.7s$$

$$= 0.7 \times 6 = 4.2\text{mm}$$

$$\therefore P_{weld} = \frac{220 \times 429 \times 4.2}{10^3} = 396\text{kN}$$

$$\mathbf{F_v = 100 \text{ kN} < 396\text{kN}}$$ **∴ O.K.**

Note: *Four short lengths of non-continuous welds would suffice, but a continuous weld is often used.*

9. BRACING CONNECTIONS

9. 1 INTRODUCTION

This Section gives general guidance on bracing connections and, where appropriate, refers to other publications for comprehensive detailed design.

Connections for bracing members comprising flats, angles, channels, **I**-sections, RHS and CHS are included. Gusset plates incorporating kidney shaped slots are sometimes used in bracing connections for single and multi-storey frames; they are described in Section 9.3.

Bracing is usually designed assuming that all forces intersect on member centroids but if this assumption is carried out in the connection design then this may produce a large connection. It is often more convenient to arrange the member intersections to make a more compact joint and check locally for the effects of eccentricities.

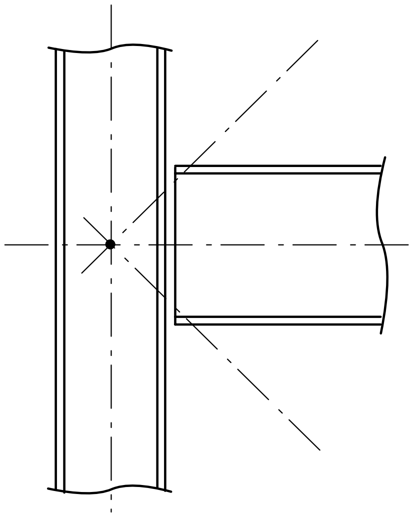
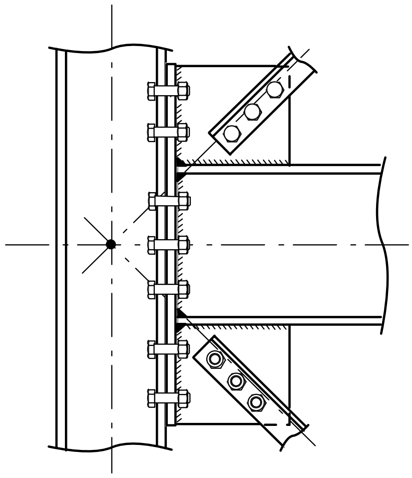
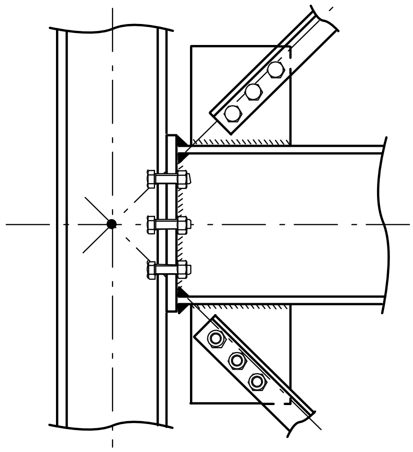
Single angle bracing with welded gusset plates to beams or columns and bolted site connections are simple to fabricate. CHS bracing is also economical, being effective in both tension and compression, the connection usually being made with a 'T' shaped element welded at each end of the member for site bolting to gusset plates. Fabrication costs are generally higher for the other types of connections shown which have either, more elements to fabricate, or a greater weld content.

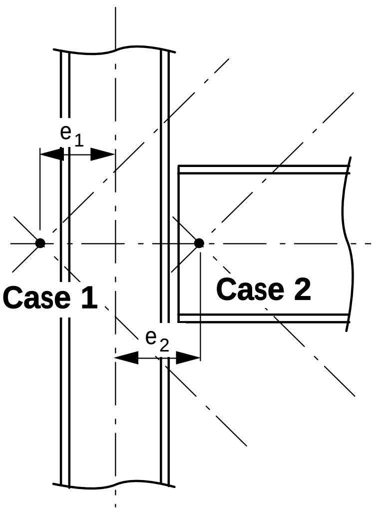
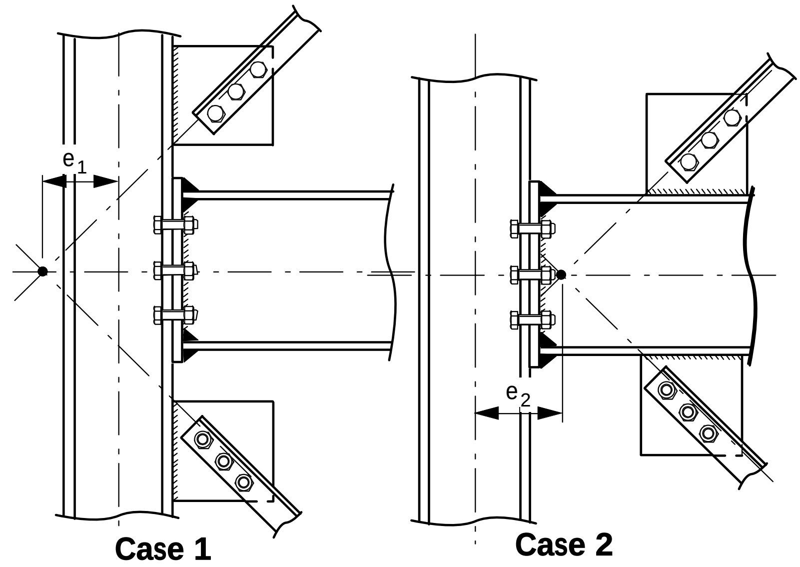
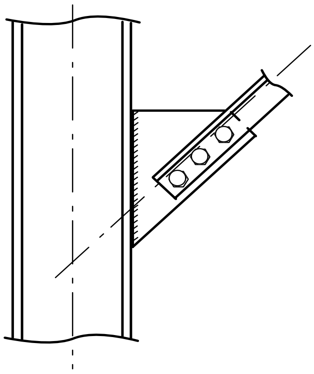
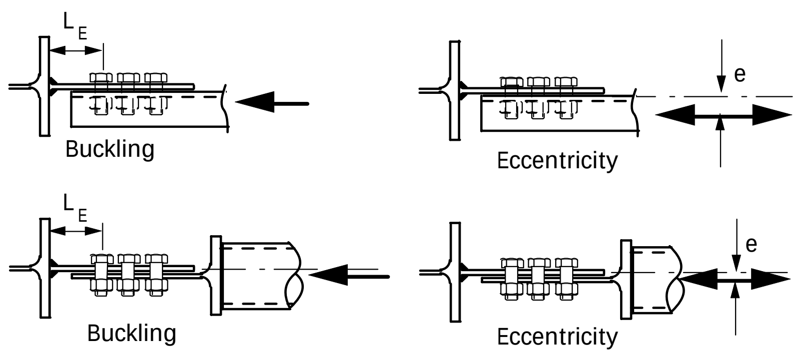
The components in the structure that attach to bracing are often the first parts to be erected since stability is then available for erection of further components. Connections should therefore be made such that erection can be made swiftly and, as far as possible, without the need of temporary supports. Care should also be taken to ensure that there are no encumbrances built into the connection which make erection difficult.

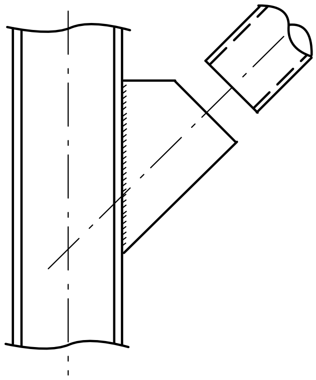
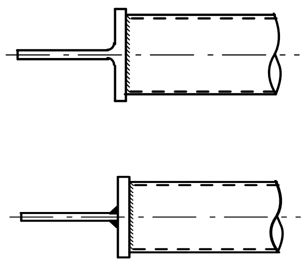
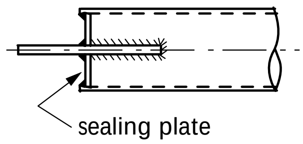
Bracing systems may include beams acting as horizontal members in bracing systems; the beams may be provided with double angle cleats, flexible end plates or fin plate connections as decribed in Sections 4, 5 or 6. It is necessary to allow for in the connection design any axial force present in the beams and induced shear forces, in addition to the normal end reactions.

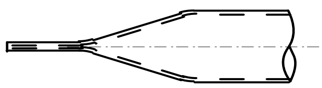
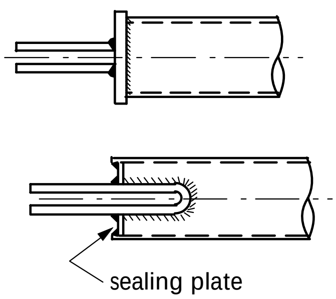
9. 2 DESIGN CONSIDERATIONS

Table 9.1 shows possible solutions for bracing connections and indicates matters relevant to each type of connection. The design method is to decide on a force path to be adopted that provides equilibrium at the joint and check each element for shear, tension, bearing, or buckling as appropriate.

<table>
<tr><th colspan="3">Table 9.1 Bracing connections</th></tr>
<tr><th>REQUIREMENT</th><th colspan="2">POSSIBLE SOLUTION</th></tr>
<tr><td rowspan="6">(1) Multiple Connections
Noding joints
</td><td colspan="2">(i) Extended end plates with fully welded gusset
</td></tr>
<tr><th>Effect</th><th>Design</th></tr>
<tr><td>• Gusset plates stiffen the extended end plate.
• Overall connection may become a rigid connection.</td><td>Connection may be designed using Joints in Steel construction: Moment Connections [24], using the combined effects of coexistent moments, shears and horizontal tension.</td></tr>
<tr><td colspan="2">(ii) Full or partial depth end plates
</td></tr>
<tr><th>Effect</th><th>Design</th></tr>
<tr><td>Connection treated as a nominally pinned connection
• Gusset plates may be vulnerable to damage during transit.
• Gusset plates can be stiffened if necessary.</td><td>Connection to be checked for:
• total shear due to bracing loads and beam loads
• transfer of horizontal forces
• eccentricity of the vertical shear from the bracing system.</td></tr>
</table>

<table>
<tr><th colspan="3">Table 9.1 Bracing connections (continued)</th></tr>
<tr><th>REQUIREMENT</th><th colspan="2">POSSIBLE SOLUTION</th></tr>
<tr><td rowspan="3">(2) Multiple Connections Non-noding joints

</td><td colspan="2">Individual bracing plates

</td></tr>
<tr><th>Effect</th><th>Design</th></tr>
<tr><td>• Additional moments are induced in members.
• Gusset plates may be vulnerable to damage during transit.</td><td>Check for:
• Additional moment due to eccentricity e_1 or e_2.
• total shear due to bracing components and beam loads
• horizontal forces due to bracing</td></tr>
<tr><td rowspan="3">(3) Gusset Plates in compression and axial alignment
</td><td colspan="2">
</td></tr>
<tr><th>Effect</th><th>Design</th></tr>
<tr><td>• Some gusset plates that are in compression may be prone to buckling between the first bolt and the connecting member.
• Single sided gusset connections are by their nature out of alignment</td><td>• Plate buckling may be checked using strut curve C Table 24 of BS 5950-1[1]. For further design guidance see reference: CIMsteel Engineering Basis [43], Cidect Guides[28] [29]
• Moments due to force eccentricity 'e' can be ignored. Angle, channel and T-section struts to be designed in accordance with BS 5950-1 cl. 4.7.10</td></tr>
</table>

<table>
<tr><th colspan="2">Table 9.1 Bracing connections (continued)</th></tr>
<tr><th>REQUIREMENT</th><th>POSSIBLE SOLUTION</th></tr>
<tr><td rowspan="6">(4) Bracing Members
Hollow section (CHS, RHS)
</td><td>Design guidance for all types of CHS and RHS connections is given in Cidect Guides[28] [29]</td></tr>
<tr><td>(i) Rolled or fabricated 'T'

Stalks of standard rolled T sections are relatively thin and therefore bolt bearing may control capacity.</td></tr>
<tr><td>(ii) Slotted tube


Sealing plates may be required. This type of connection involves a relatively high amount of fabrication.</td></tr>
<tr><td>(iii) Flattened end CHS

During cold flattening longitudinal cracks/splitting may apear on the edges of the flattened CHS. Usually this has no effect on the performance of the connection.</td></tr>
<tr><td>(iv) Fork plates


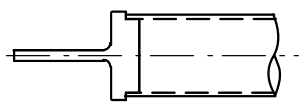
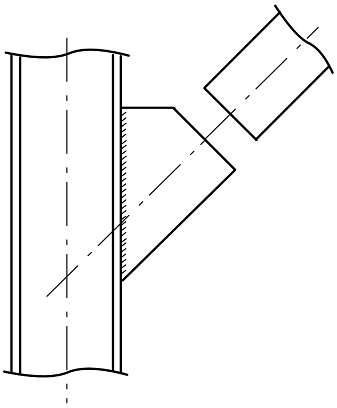
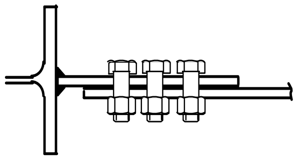
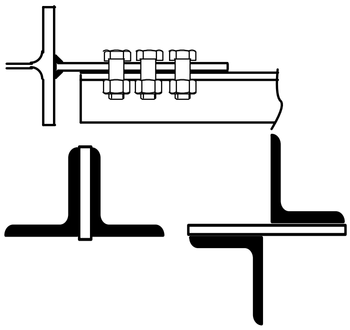
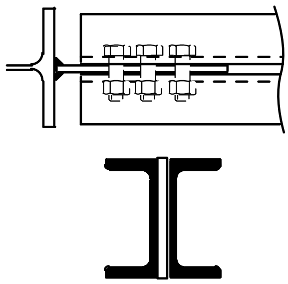
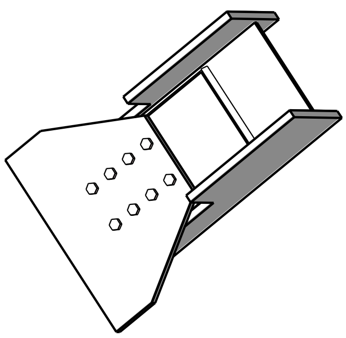
Both fork plate types make more effective use of bolts (double shear) but are more expensive to fabricate. May be required for architectural reasons for single pin connections .
Sealing plates may be required.</td></tr>
<tr><td>(v) Castings

Castings can be economical if sufficient numbers are required. Refer to manufacturers.
Not used in orthodox buildings. For further guidance see SCI-P172[44].</td></tr>
</table>

Table 9.1 Bracing connections (continued)

REQUIREMENT	POSSIBLE SOLUTION
(5) Bracing Members **Flats, angles, channel, UB/UC** 	**(i) Flat plate**  Simple tension bracing but lacks compression capability. Susceptible to distortion during transport and erection.
	(ii) Double Angle  Double angle bracing can be ‘back-to-back’, separated by division plates spaced at intervals along the length (of same thickness as the gusset) or 'starred' formation with battens in both directions spaced at intervals along the length.
	(iii) Double Channel  Double channel bracing can be ‘back-to-back', separated by division plates spaced at intervals along the length of same thickness as the gusset.
	(iv) UB/UC  UB/UC The flanges on one side can be removed to allow the gusset plate to be bolted to the web. A web reinforcing plate can be used if it is necessary to increase connection capacity.

9.3 KIDNEY SHAPED SLOT CONNECTIONS

Kidney shaped slots used in bracing connections provide a practical solution for connecting members of varying lengths and angles with standardised components. The kidney shaped slot is generally formed in the gusset plate rather than at the end of the bracing member, as shown in Figure 9.1.

Non-preloaded bolts (ordinary "bearing" bolts) are used rather than preloaded (friction grip) bolts.

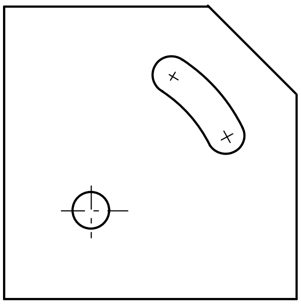


Figure 9.1 Gusset plate with kidney shaped slot

Practical Considerations

The three common methods of forming the kidney shaped slot are:

- Punching full size, in one operation, using a die that matches the slot dimensions.
- Drilling two holes and completed by cutting.
- Machine operated plasma or flame cutting.

The advantages of a connection incorporating a kidney shaped slot are that:

- A standard end connection can be used for bracing members.
- Standard gusset plates which accommodate a range of bracing member may be used.
- A two bolt connection allows one bolt to be inserted in the connection whilst locating and maintaining alignment with a podger spanner through the other hole.

Standard details encourage a batch production approach to the fabrication of the connection components, and save time in design, detailing, checking and fabrication.

Recommended Geometry

The recommended geometry for the single circular hole and the kidney shaped slot is as shown in Figure 9.2 and given in tables 9.2 and 9.3.

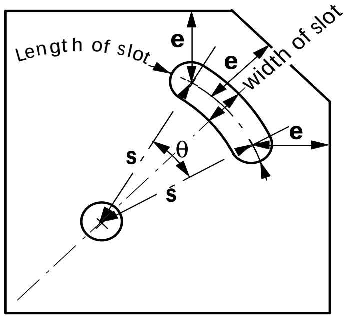


Figure 9.2 Recommended geometry for gusset plate with kidney shaped slot

Table 9.2 Hole and slot dimensions (see Figure 9.2)

Bolt Diameter mm	Circular Hole Diameter mm	Kidney Shaped Slot Maximum dimensions mm	
		Width	Length
≤ 24	d + 2	d +2	3d see Note 1
>24	d + 3	d +3	3d see Note 1

Note 1: but angle θ should not exceed 30°

Table 9.3 Spacing dimensions (see Figure 9.2)

	Minimum	Maximum
Spacing "s" between circular hole and kidney slot:	2.5 d	Lesser of 14t and 200 mm
End and edge distances "e"	2 d	Lesser of 11tε and 40 mm + 4t

where:

$$\varepsilon = \left[\frac{275}{p_y}\right]^{0.5}$$

d = bolt diameter

t = thickness of gusset plate

p_y = design strength of gusset plate

Design Rules

For a two-bolt connection where one of the holes is kidney shaped, the following rules are recommended, in accordance with BS 5950-1:2000:[1] clause 6.3.2.4 and 6.3.3.3.

Connection Capacity

Total shear capacity = $1.6P_s$

P_s = shear capacity of a single bolt in standard clearance hole.

Total bearing capacity = $1.5P_{bs}$

P_{bs} = bearing capacity of a gusset plate for a single bolt in standard clearance hole.

Effective Length of Bracing Member

Since the kidney shaped slot allows rotation of the connection, the connection detail cannot be assumed to provide directional restraint equivalent to that of two bolts in standard clearence holes.

When effective lengths for hollow section bracing members are determined from Table 22 of BS 5950-1[1], a connection incorporating a kidney shaped slot should not be assumed to provide any directional restraint in the plane of the gusset.

When angles are used as bracing members and the connections incorporate kidney shaped slots, the provisions for single bolt connections in Table 25 of BS 5950-1[1] should be adopted, though the reduction to 80% in compression resistance stated in Note 3 to Table 25 need not be applied.

Provision of Washers

Clause 6.1.5(ii) of the National Structural Steelwork Specification (NSSS)[8] specifies that a plate washer or heavy duty washer be used under the bolt head and nut when bolts are used to assemble components with oversize or slotted holes. The origin of this clause concerned slots which were provided to permit movement in service. Shouldered bolts would normally be used in such joints.

The use of kidney shaped slots in bracing connections is a different situation to the one intended to be addressed by Clause 6.1.5(ii) of the NSSS[8]. Ordinary washers are considered to be satisfactory for use in bracing connections with kidney shaped slots.

Further information

Further information can be found in the, SCI Publication 249 *Design Capacity of Kidney Shaped Slotted Connections* (1998)[45].

10. SPECIAL CONNECTIONS

10.1 INTRODUCTION

Steelwork connections for simple construction, illustrated in Sections 1 to 9 of this Guide, will generally produce the most economic steel frame. A departure from these connections will inevitably result in an increase in overall cost. The increase in detail drawing, fabrication and erection costs can be more than 100% if non-standard connections form the majority of the connections used.

It is therefore good economic practice to ensure that steelwork can be placed with centrelines on established grids. The top flanges of beams should where possible be at a constant level, but this is less critical to cost than eccentric connections.

The need for special connections can often be avoided by the judicious selection of member sizes. A minimum weight structure is unlikely to be the most cost effective. Some elegant structures employing several thousand tonnes of structural steelwork have been economically built using only six or eight sections for the whole of the design.

The frame design is best conducted in two stages:

1st Stage:

> *Obtain a satisfactory component design in accordance with the Code of Practice.*

2nd Stage:

> Rationalize these components into groups to achieve a practical arrangement and reasonable range of member sizes.

Steelwork is a versatile material and a design solution can be found for any connection, but this may be at a high cost.

Quite often the solution is to use a slightly modified version of one of the standardised connections, subject to a few additional design checks. Such connections should incorporate, as far as possible, the component sizes given in Section 2 and the design principles adopted in this Guide.

Some typical examples of situations where special connections are required are presented on the following pages, together with possible configurations and any special considerations affecting the design or detailing. It should be kept in mind that such connections should be avoided if an alternative standard detail is possible.

Table 10.1 deals with cases where connecting beams are at different levels. Table 10.2 considers beams which connect members not intersecting at 90^{o}.

Tables 10.3 and 10.4 show connections to RHS columns where welding and conventional bolting are adopted and which provide (a more expensive) alternative to the use of *Flowdrill* and *Hollo-Bolt* connectors.

Table 10.5 takes account of eccentric connections necessary for off-grid beams. Suitable connections for these conditions must be introduced. These are often the single greatest cause of the increased costs in detailing, fabrication and erection of beam/column structures.

A case where different size columns are spliced such that the outer faces are aligned is shown in Table 10.6. The special requirements for a column base in a braced bay is covered in Table 10.7.

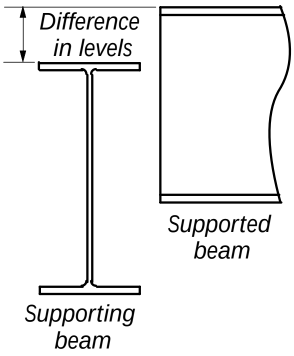
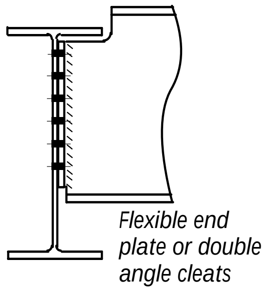
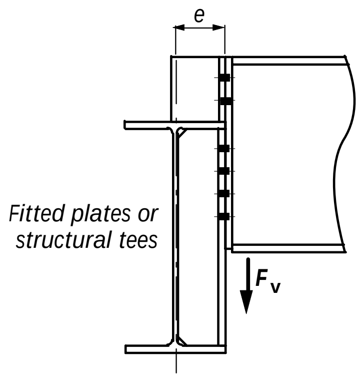
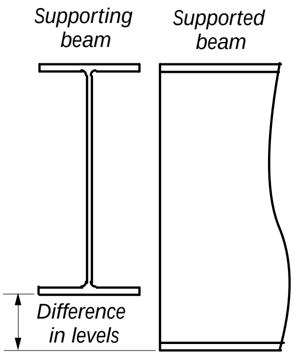
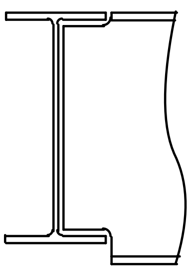
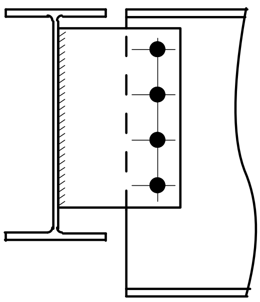
Table 10.1 Beams at different levels		
REQUIREMENT	**POSSIBLE SOLUTION**	**SPECIAL CONSIDERATIONS**
(1) Beam to Beam 	**(i) If level difference is small** 	• *Reduced local shear and bending capacity of supported beam.* • *Unrestrained top flange.* • *Reduced number of bolts.*
	(ii) If level difference is large 	• *To avoid the effects of torsion on the supporting beam, design bolt group, tees and end plates to resist moment of $F_v \times e$.* • *Expensive to fabricate.* • *Pre-set to ensure alignment/level of supported beam (especially for cantilevers).*
(2) Beam to Beam 	**(i) Increase depth of supporting member**	• *Envisage ease of connection at conceptual design stage.*
	(ii) Notched supported beam 	• *Reduced shear and bending capacity of supported beam.* • *Lateral torsional buckling of supported beam if not laterally restrained.*
	(iii) Extended fin plate 	• *Stability of fin plate.* • *Torsional and positional restraint to supported beam.*

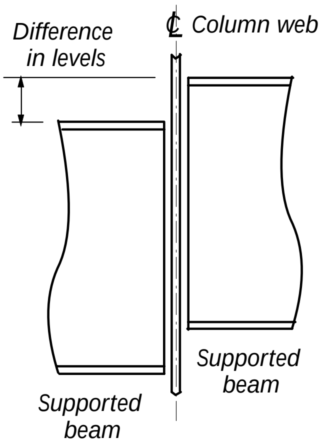
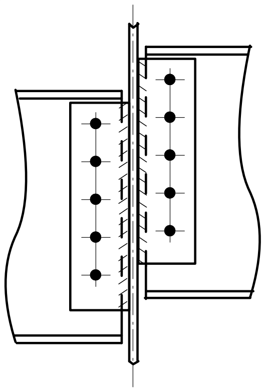
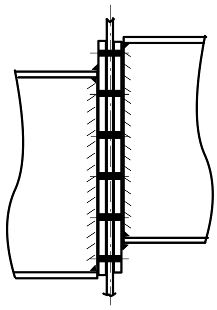
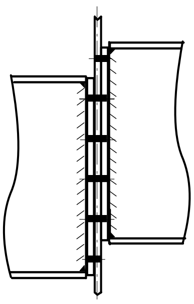
Table 10.1 Beams at different levels (continued)		
REQUIREMENT	**POSSIBLE SOLUTION**	**SPECIAL CONSIDERATIONS**
(3) Beam to Column 	**(i) Fin plates** 	
	(ii) Extended end plates 	• *Connection flexibility.* • *Ease of erection.* • *May need to vary pitch to ensure adequate clearance.*
	(iii) Non-standard pitches 	• *Bearing on column web from central, heavily loaded bolts.* • *Different bolt lengths.* • *May need to vary pitch to ensure adequate clearance.*

Table 10.2 Skewed connections

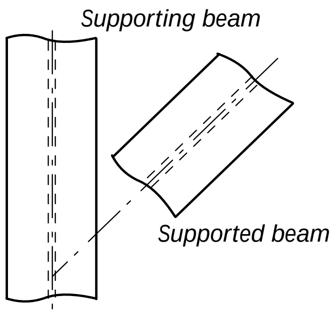
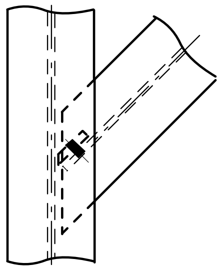
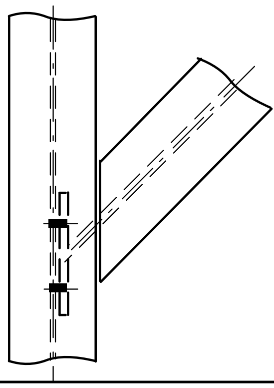
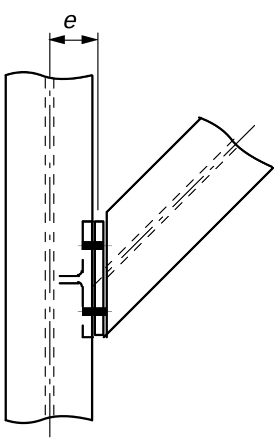
REQUIREMENT	POSSIBLE SOLUTION	SPECIAL CONSIDERATIONS
(1) Beam to Beam 	**(i) Supported beam depth less than supporting beam depth** *Fin Plate* 	• *Reduced local shear and bending capacity of supported beam due to long top notch.* • *Non-standard fin plate may be required due to bolt clearances.* • *Weld to fin plate. See BS 5950-1: 2000[1] clause 6.8.1.*
	(ii) Supported beam depth greater than supporting beam depth *End Plate* 	• *Reduced local shear and bending capacity of supported beam due to long top and bottom notch.* • *Non-standard end plate may be required.* • *Weld to end plate. See BS 5950-1: 2000[1] clause 6.8.1.*
	(iii) Beams of same depth *Extend face of connection to toes of supporting beam.* 	• *To avoid the effects of torsion on the supporting beam, design bolt group, tees and end plates to resist moment of $F_v \times e$.* • *Expensive to fabricate.* • *Clearance for bolts.* • *Weld to end plate. See BS 5950-1: 2000[1] clause 6.8.1.*

Table 10.2 Skewed connections (continued)

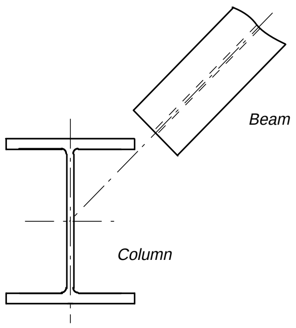
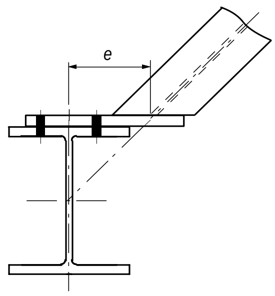
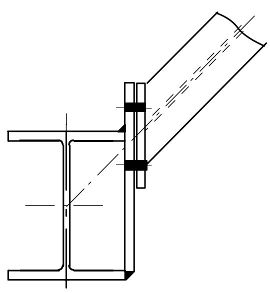
REQUIREMENT	POSSIBLE SOLUTION	SPECIAL CONSIDERATIONS
(2) Beam to Column 	**(i) Extended end plate** 	• *Bolt group and end plate designed to resist moment of F_v x e.* • *Weld to end plate. See BS 5950-1: 2000[1] clause 6.8.1.* • *Structural integrity - large bolt tensions developed.* • *Structural integrity - thick end plate required.* • *Structural integrity - check column flange for bending.*
	(ii) Plate across UC toes 	• *Large shear force on one-sided fillet weld to UC toe.* • *Weld to end plate. See BS 5950-1: 2000[1] clause 6.8.1.* • *Structural integrity - Large tensions developed in weld.* • *Structural integrity - thick plates may be required.*

Table 10.3 I-Beam to RHS column

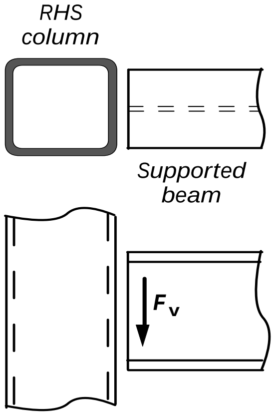
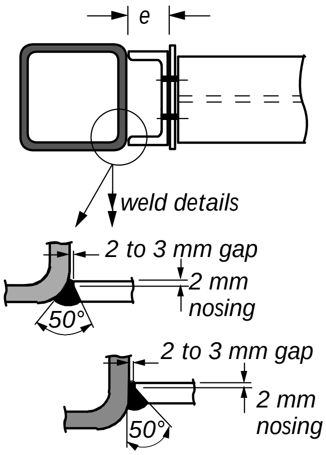
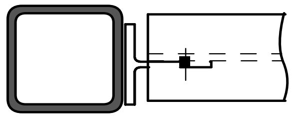
REQUIREMENT	POSSIBLE SOLUTION	SPECIAL CONSIDERATIONS
	**(i) Channel Bracket** 	• *Provides alternative to direct bolting by Flowdrill or Hollo-bolt.* • *Bracket transmits load direct to column side wall welds.* • *Bracket and welds to resist moment due to eccentricity.*
	(ii) Tee connection 	• *Tee stiffens column wall and can be used where a fin plate is insufficient.*

Table 10.4 Parallel Beams to RHS column

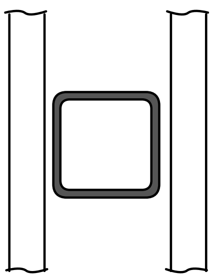
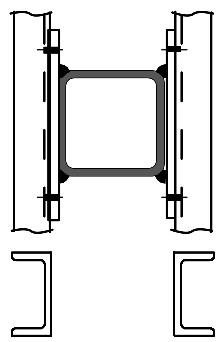
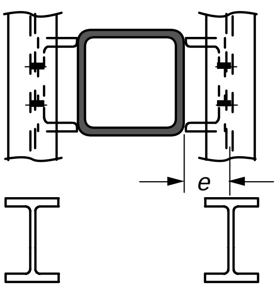
REQUIREMENT	POSSIBLE SOLUTION	SPECIAL CONSIDERATIONS
	**(i) Side plates** 	• *Bolts and welds to resist F_v and any moment due to out of balance loading.*
	(ii) Bracket 	• *Bracket transmits loads direct to column side wall welds.* • *Bracket and welds to resist moment due to eccentricity.*

Table 10.5 Off-grid connections

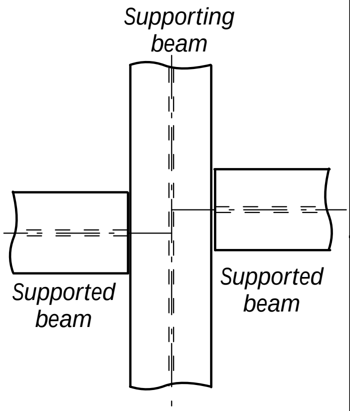
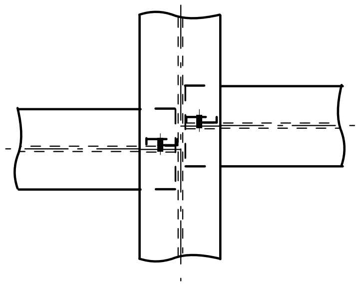
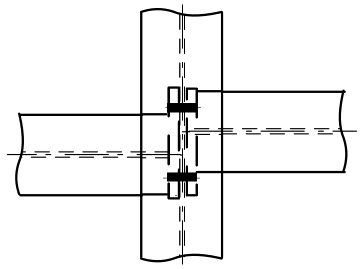
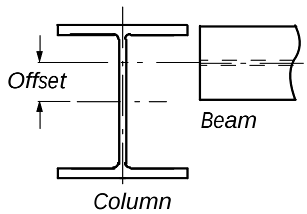
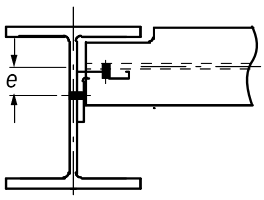
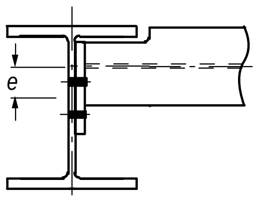
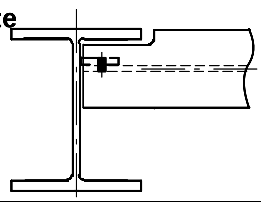
REQUIREMENT	POSSIBLE SOLUTION	SPECIAL CONSIDERATIONS
(1) Beam to Beam 	**(i) Fin Plates** 	• *Ensure beam connects to correct side of fin plate.*
	(ii) End Plates (or non-standard cleats) 	• *Non-standard bolt spacings and fittings required.*
(2) Beam to Column Small offsets 	**(i) Single angle web cleat** 	• *Design bolts to column web to resist moment of $F_v \times e$.* • *Effective length of unrestrained beam for torsional strength.* • *Structural integrity - Tension in cleat.* • *Structural Integrity - Column web bending.*
	(ii)End plate 	• *Bolt group and end plate designed to resist moment of $F_v \times e$.* • *Non-standard bolt centres and end plate.* • *Structural integrity - Large bolt and weld tension developed.* • *Structural Integrity - Thick end plate required.* • *Structural Integrity - Check column web for shear and bending.*
	(iii) Fin plate 	• *Bolt clearances and ease of installation on site.*

Table 10.5 Off-grid connections (continued)

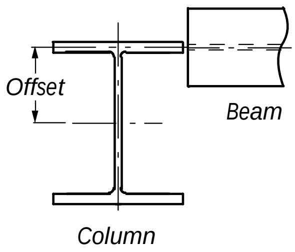
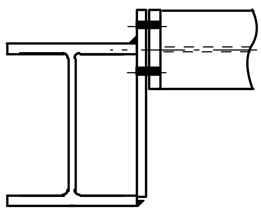
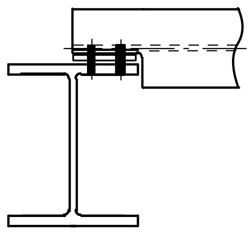
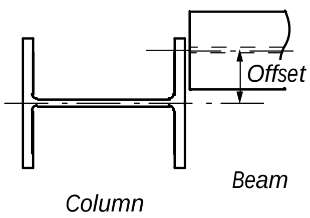
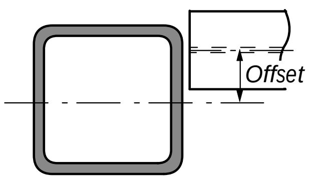
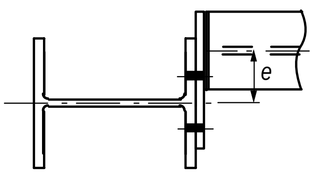
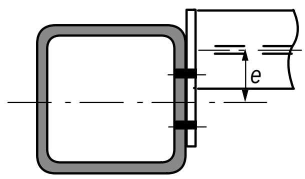
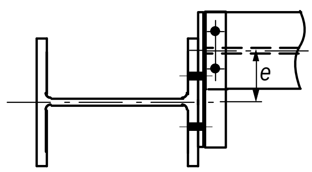
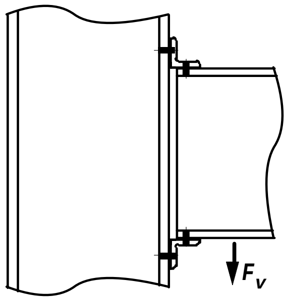
REQUIREMENT	POSSIBLE SOLUTION	SPECIAL CONSIDERATIONS
(3) Beam to Column Large offsets  	**(i) Plate across UC toes** 	• *Large shear force on one-sided fillet weld to UC toe.* • *Non standard fittings.* • *Structural integrity - Bending of plate across toes.* • *Structural Integrity - Large weld tension.*
	(ii) Packs to column flange 	• *Large bolt grip lengths. See BS 5950-1:2000*[1] *clause 6.3.2.3.* • *Reduced bolt shear capacity due to thick packs. See BS 5950-1*[1] *clause 6.3.2.2.*
  	**(iii) Extended end plate**   	• *Design bolt group and end plate for moment of $F_v \times e$.* • *Structural Integrity - Large bolt tensions.* • *Structural integrity - Thick end plate required.* • *Structural Integrity - Column flange bending.*
	(iv) Top and bottom cleats   	• *Design bolts to column flange and bottom cleat for moment of $F_v \times e$.* • *Design bottom cleat to carry the full vertical load, F_v.* • *Check bearing and buckling strength of supported beam web.*

Table 10.6 Column splices

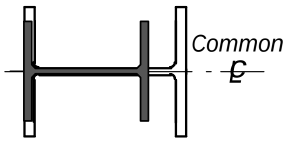
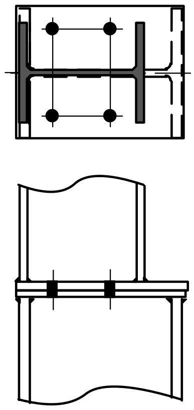
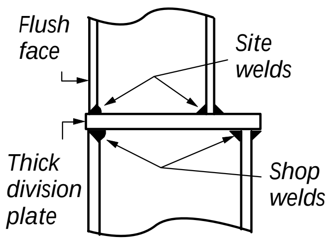
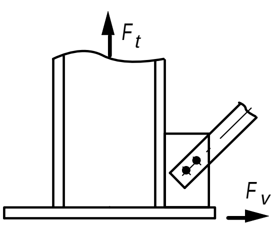
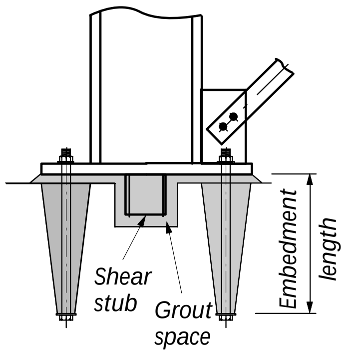
REQUIREMENT	POSSIBLE SOLUTION	SPECIAL CONSIDERATIONS
There may be occasions when architectural or other requirements dictate that one of the column faces align, making the upper and lower columns eccentric.  	**(i) Cap and base splice** 	• *Cap and base splices are especially suitable if there is a large change in section size making cover plates impracticable.* • *Thick plates may be required to transmit bearing.* • *The splice should provide continuity of stiffness about both axes and resist any tension where bending is present (See BS 5950-1[1] clause 6.1.8.2), or where there are any structural integrity requirements.*
	(ii) Site welded splice  	• *Expensive site welding and N.D.T.* • *Thick plate required to transmit bending moment.*

Table 10.7 Column bases in braced bays

REQUIREMENT	POSSIBLE SOLUTION	SPECIAL CONSIDERATIONS
Base in a braced bay  	 	• *Welds to column designed to resist uplift, F_t and horizontal shear, F_v.* • *Design base plate for bending.* • *May need to provide shear stub to resist large horizontal shear.* • *Design H.D embedment length to resist uplift F_t. See Moment Connections publication. [24]*

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2. BRITISH STANDARDS INSTITUTION
   BS4 Structural steel sections
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   (Including amendment 2001)

3. BRITISH STANDARDS INSTITUTION
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   BS EN 10210-1: 1994 Technical delivery requirements
   BS EN 10210-2: 1997 Tolerances and sectional properties. (Replaces BS 4848: 1991)

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    BS 4395-2: 1969 Higher grade bolts and nuts and general grade washers (including amendment 1, amendment 2: 1976)

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APPENDICES

APPENDIX A STRUCTURAL INTEGRITY

A.1 GENERAL

BS 5950-1:2000[1] clause 2.4.5 deals with structural integrity and the avoidance of disproportionate collapse. Compliance with this code is deemed to satisfy the Building Regulations Approved Document A[46].

The structural integrity requirements need to be satisfied for beam to column connections and for column splices. Design methods are given in Sections 4 to 7 for each of the connection types within these headings. It has been assumed that beam to beam connections are very rarely required to meet the structural integrity clauses, and thus no design methods are offered. (Designers may adapt the beam to column design methodology in situations where beam to beam connections are required to meet structural integrity requirements.)

There has also been a lack of data on the ultimate strength of connections making it difficult to relate BS 5950-1[1] to ultimate capacity with a high level of confidence. A test programme[5] completed in June 1990 determined the ultimate capacities for a range of connections covered in this design guide.

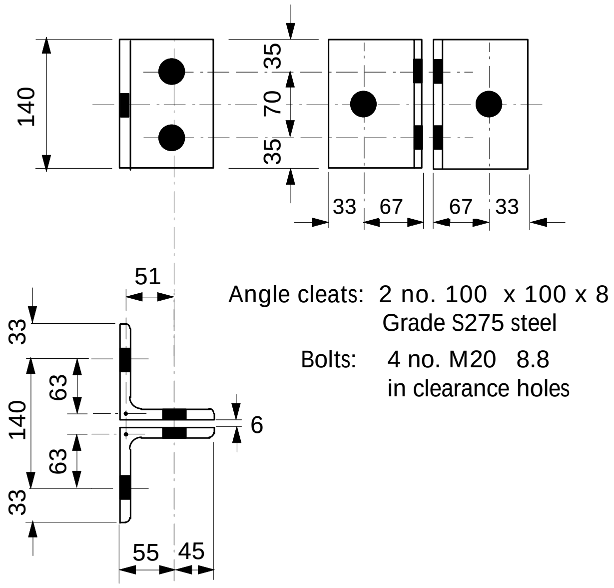
Appendices B, C, and D give a summary of the results of the test programme, and set out complete analytical models. In some cases, simplified versions of these models are presented in the design methods for each type of connection. The simplified models give conservative values.

Although BS 5950-1[1] only relates to design of the steelwork, the structural integrity requirements are intended to ensure robustness of the completed structure. Other structural elements may also be used to satisfy the tying force requirements. For example, with composite floor systems, it is possible to design and detail the reinforcement, the concrete slab, and the shear connectors to comply with BS 5950-1[1]. **This will usually give simpler design procedures and greater economy than carrying the tying force through the connections.**

The following subsections examine the resistance, under large displacements, of double angle web cleats, end plates and fin plates. Critical cases are identified which are likely to be the weakest connections to be used in practice.

A.2 DOUBLE ANGLE WEB CLEATS

Eleven cleats were tested to destruction to verify a postulated design procedure for the ultimate capacity of such connections. Two specimens comprised the smallest cleats likely to be used in practice and which would satisfy BS 5950-1[1] edge distance and other detailing requirements, see Figure A.1; these failed at 178 and 210 kN.



Critical case is:

Maximum gauge,	g	=	140mm
Minimum beam web thickness,	t_w	=	6mm
Minimum cleat thickness,	t_C	=	6mm
Minimum cleat length,	l	=	140mm

Figure A.1 Minimum practical double angle web cleat connection

Appendix B gives a complete design method for determining the ultimate capacity. The design checks in Section 4 give a simplified design method. These two design methods are compared to the test results in Figure A.2.

BS 5950-1[1] clause 2.4.5.2 requires a minimum tie force of 75 kN. The coefficients of regression listed in Figure A.2 show that the ultimate capacities of the smallest cleats are not isolated results, but are part of a family of results.

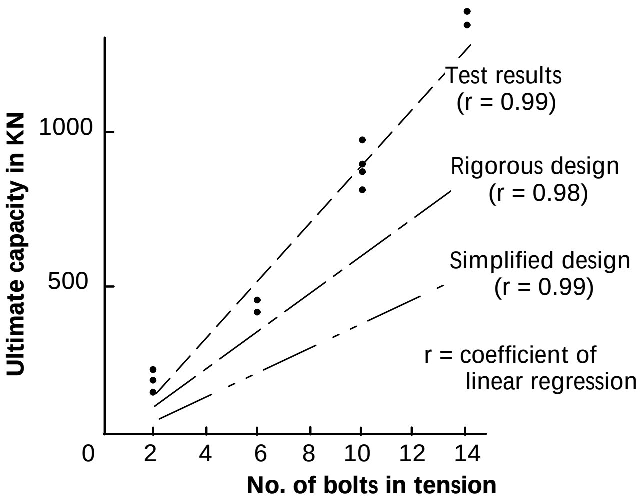
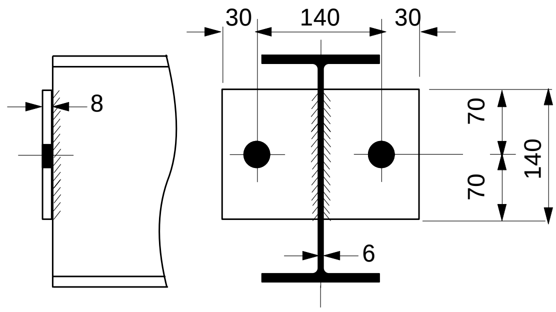


Figure A.2 Comparison of test results against design method for double angle web cleats



End plate: 200 x 8 x 140 long
Grade S275 steel

Bolts: 2 no. M20 8.8
in clearance holes

Critical case is:

Maximum gauge,	g	=	140mm
Minimum beam web thickness,	t_w	=	6mm
Minimum weld leg size,	s	=	6mm
Minimum plate thickness,	t_p	=	8mm
Minimum plate length,	l	=	140mm

Figure A.3 Minimum practical end plate connection

The factor of safety for the lower result of the two small cleats is 178/75 = 2.37.

The design checks therefore include the dimensions of the smallest cleats that were tested as the minimum structural integrity requirement.

Single angle web cleats

The failure mechanism of such connections will differ significantly and deformations will be higher. The strength per bolt may be similar but was not subjected to testing and is not amenable to calculation. A particular weakness is likely to be the possibility of the plate pulling over the bolt head or nut. Such connections should not be used to carry tying forces without experimental evidence of their capacity.

A.3 FLEXIBLE END PLATES

Nine end plates were tested to failure to verify a postulated design procedure for the ultimate capacity of such connections. Three specimens comprised the smallest end plates likely to be used in practice and which satisfy BS 5950-1[1] edge distance and other detailing requirements, see Figure A.3. These specimens were connected to different support conditions, and failed at 203, 211 and 212 kN.

Appendix C gives a general design method for determining the ultimate capacity, and this is included in the design checks for flexible end plates (Section 5). Figure A.4 shows the comparison between the test results and the values derived from the design method.

The coefficients of regression listed in Figure A.4 show that the ultimate capacities of the smallest end plates are not isolated results, but are part of a family of results. BS 5950-1[1], clause 2.4.5.2 requires a minimum tie force of 75kN. The factor of safety for the lowest result of the small end plates is 203/75 = 2.71.

The design checks therefore include the dimensions of the smallest end plates that were tested as the minimum structural integrity requirement.

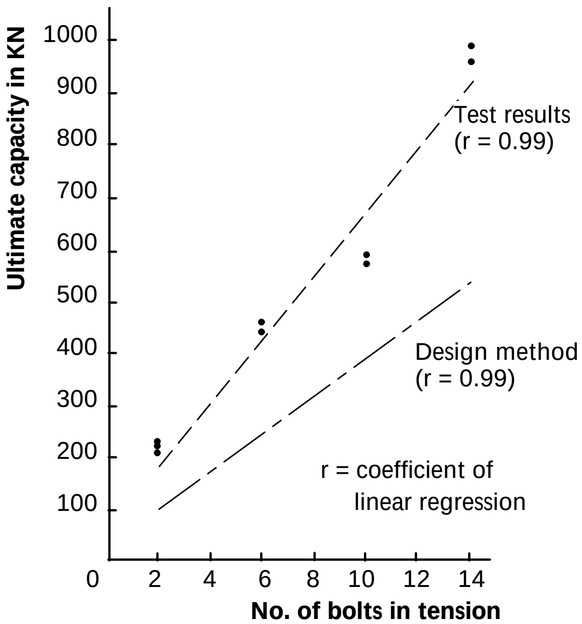


Figure A.4 Comparison of test results against design method for flexible end plates

A.4 FIN PLATES

The ultimate capacity of fin plates can be determined by simple structural analysis; the structural model is not complex. The design checks include the details of the minimum fin plate required to resist 75 kN tie force (BS 5950-1[1], clause 2.4.5.2). Checks are also given for resisting higher tie forces.

A.5 PRYING AND TIE FORCES

BS 5950-1[1], clause 2.4.5.1 accepts substantial permanent deformations of members and their connections. Several unusual features follow from taking account of the benefits of large displacements and strains:

i) The gross deformations can reduce the local eccentricities within the connection.

ii) The local strains are so high that account can be taken of strain hardening, so that the design strength is based on U_s the minimum ultimate tensile strength, with a suitable value γ_m.

iii) Prying forces and bolt strains are so high that it is not appropriate to rely on nominal tension strength ($0.8p_t$) of the bolts (see BS 5950-1[1] clause 6.3.4.2).

iv) Plate bearing strengths for ordinary bolts are set relatively low in BS 5950-1[1] table 32 in order to ensure that bearing deformations at working loads will not lead to any unserviceability. For high strength friction grip bolts the bearing strengths (clause 6.4.4) are significantly higher since joint serviceability is provided by the frictional resistance, so it is appropriate to use these higher values when designing to resist tying forces.

A reduced bolt design strength is derived in Appendix D which takes prying forces into account when resisting tying forces.

APPENDIX B LARGE DISPLACEMENT ANALYSIS DOUBLE ANGLE WEB CLEATS

LARGE DISPLACEMENT ANALYSIS BASED ON ULTIMATE LOAD TESTS

B.1 Behaviour

Figure B.1 shows the deformed shape of a double angle cleat connection that has been subjected to high tension. The noteworthy features are:

i) The potential magnitude of the displacement Δ; ignoring second order effects, Δ defines the displaced geometry of the web cleats. (In the tests, the values of Δ always exceeded 30mm prior to failure.)

ii) These displacements reduce the eccentricities on the web cleats. Part of the tying force is carried by tension in the legs of the cleats. (There are also shears associated with the moments.)

iii) There are four critical sections in each cleat, which are subject to high plastic strains under the combined action of shear, tension and moment. Two are located at, or near, the bolt centrelines; two are located at the tips of the radiused portion of the heel of the cleat. (The heel itself is subject to higher moments but is reinforced by the radius.)

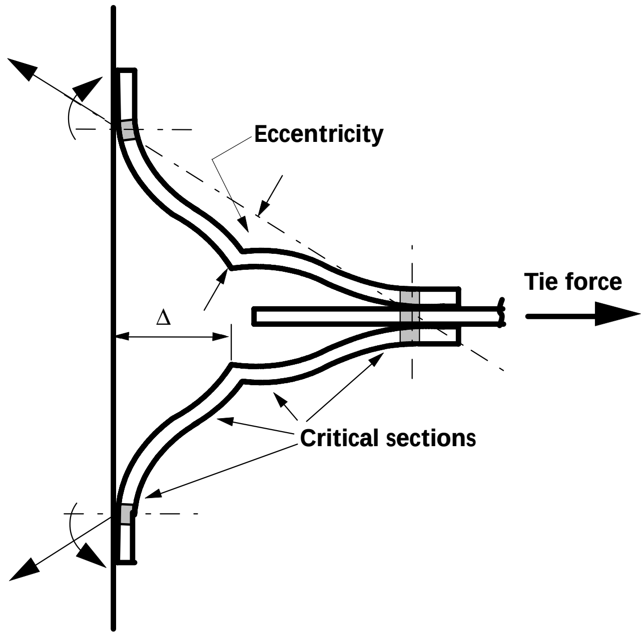


Figure B.1 Double angle web cleats under tension

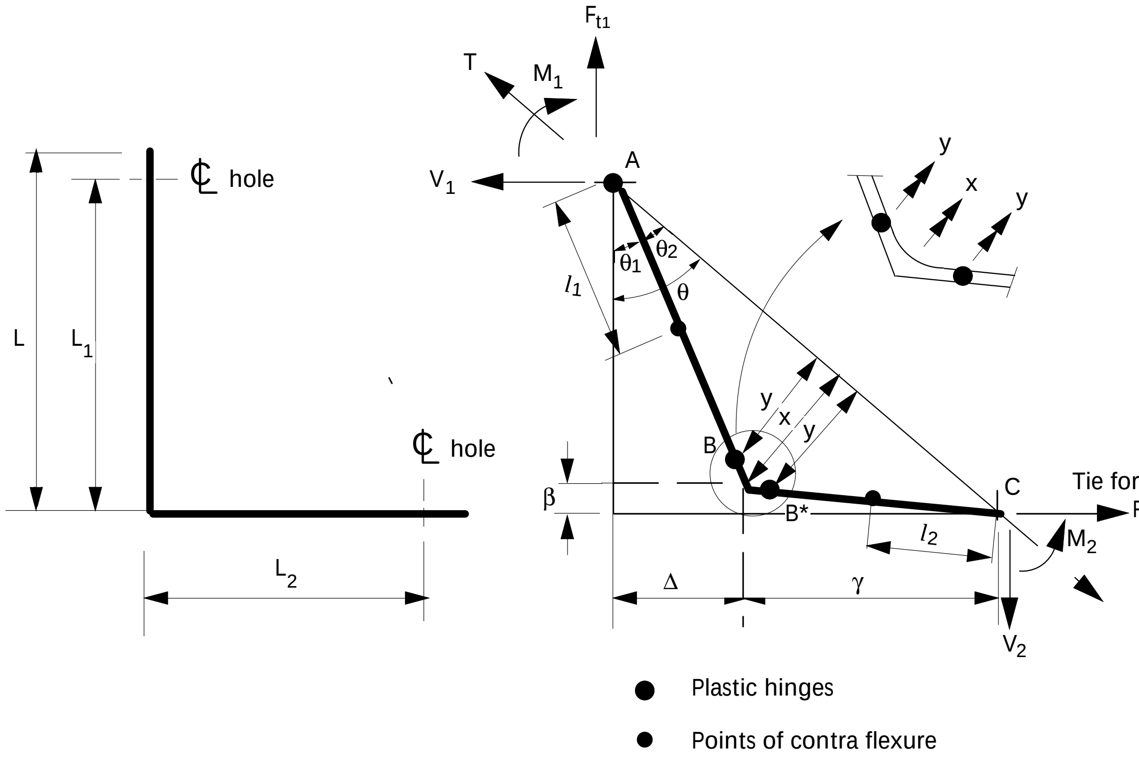


Figure B.2 Stick model of a web angle cleat under tension

B.2 Method of Analysis

Figure B.2 presents a simplified 'stick' model of a web cleat. It is possible to define all the geometry for a given L_1, L_2 and D as follows:

$$\theta_1 = \sin^{-1} \Delta/L_1$$

$$\beta = L_1 (1 - \cos \theta_1)$$

$$\gamma = (L_2^2 - \beta^2)^{1/2}$$

$$\theta = \tan^{-1} (\Delta + \gamma)/L_1$$

$$\theta_2 = \theta - \theta_1$$

$$x = L_1 \sin \theta_2$$

$$y = x - (r + t/2)1/\sqrt{2}$$

where t = cleat thickness

r = root radius

The positions of the points of contraflexure are then defined by the relative magnitudes of the moment capacities at A, B, B* and C respectively. These are approximately linear functions of the effective lengths of the critical sections.

These effective lengths are the full lengths for B and B* (L_{eB}) and are the full lengths less bolt holes and any allowance for loss of effectiveness of end regions for A and C, ie. L_{eA} and L_{eC} (This loss of effectiveness is discussed later.)

Thus:

$$l_1 = [L_1 - t/2 - r] \left[\frac{L_{eA}}{L_{eA} + L_{eB}}\right]$$

$$l_2 = [L_2 - t/2 - r] \left[\frac{L_{eC}}{L_{eC} + L_{eB}}\right]$$

Because the strains at the critical regions are so high, it is more appropriate for design to use the ultimate tensile strength U_s divided by an appropriate γ_m rather than the yield strength. Thus,

$$p_u = \text{design tensile strength} = U_s/\gamma_m$$

At any critical section there is an interaction between the shear V, the tension F_t and the moment M to cause full plasticity. As explained above, these three stress resultants all relate directly to the force T.

In the absence of any more authoritative guidance, the Von Mises yield criterion is used to define the interaction at full strength between the average shear stress, v and the longitudinal strength, p_{uv} in the presence of that shear.

Thus:

$$v = \frac{V}{L_e t} \quad \text{and} \quad p_{uv} = [p_u^2 - 3v^2]^{1/2}$$

Hence, the tensile capacity in the presence of shear, v is:

$$P_{ov} = L_e\, t\, p_{uv}$$

The moment capacity in the presence of shear, v is:

$$M_{ov} = L_e \frac{t^2}{4} p_{uv}$$

Finally, the interaction between the tensile and moment capacities in the presence of shear, v is given by:

$$\left[\frac{F_t}{P_{ov}} \right]^2 + \frac{M}{M_{ov}} = 1$$

Because of the quadratic form of this interactive relationship it cannot be solved directly. In practice, a numerical solution is required, whereby for a given value of Δ, the solution is required to converge to a value of T. This value of T simultaneously satisfies the interactive relationship above and the overall requirement for equilibrium, which is:

$$T.y = M_C + M_{B*} = M_A + M_B$$

(In practice, due to some of the approximations, these two equations cannot quite be satisfied simultaneously; the one associated with the lower value of T is made to govern.)

The foregoing procedure requires a specific value of Δ. In general, the greater its value the more the web cleats will have straightened and the higher will be the calculated capacity. In practice, rupture will intervene to limit strength.

Analysis of the test results showed that the mean deformation capacity is best defined by:

$$\Delta = \frac{2.6\,L}{t} \times \frac{L_1}{60} \leq 30\text{mm}$$

L = length of the cleat leg

As discussed below, this gave calculated strengths (with $\gamma_m = 1.0$) that were always less than the observed strength.

B.3 Experimental verification

A programme of tests was carried out to verity this method of analysis; this is fully reported elsewhere[5]. It comprised eleven tests on typical connections. Table B.1 presents a simple summary of the results.

B.4 Comparison with analysis

These specimens were initially analysed using the method outlined above with the defined values of Δ_{max} and measured mechanical properties. The results are presented in Table B.2.

As can be seen, the method of analysis gives an adequate margin of safety. The mean minus two standard deviations value of P_E/P_C is 1.1 and this seems a suitable value for a radical new procedure.

B.5 Simplified Design Methods

The interactive method of analysis described above has been used to develop the capacity tables in the yellow pages. As an alternative, a simplified procedure is presented in Section 4.

Table B.1 Summary of test results - Double angle cleats

TEST NO.	NO. OF BOLTS IN TENSION	MAXIMUM LOAD (kN)	MAXIMUM DISPLACEMENT (mm)	FAILURE MODE
A1/1	2	178	31.0	Angle cleat pulled over bolt head
A1/2	2	210	42.3	Angle cleat pulled over bolt head
A2	2	135	30.3	Bearing failure of angle cleat
A3/1	6	405	50.3	Bearing failure of beam web
A3/2	6	436	49.2	Bearing failure of beam web
A4/1	10	801	36.2	Fracture close to heel of angle cleat
A4/2	10	882	37.0	Fracture close to heel of angle cleat
A5/1	10	964	30.5	Fracture close to heel of angle cleat
A5/2	10	889	35.0	Fracture close to heel of angle cleat
A6/1	14	1340	31.9	Fracture close to heel of angle cleat
A6/2	14	1372	35.2	Fracture close to heel of angle cleat

Table B.2 Comparison between experimental results and design model

TEST NO.	MAXIMUM LOAD P_E (kN)	CALC. CAPACITY P_C (kN)	P_E/P_C
A1/1	178	123	1.45
A1/2	210	125	1.68
A2/1	135	93	1.45
A3/1	405	331	1.22
A3/2	436	325	1.34
A4/1	801	553	1.45
A4/2	882	550	1.60
A5/1	964	744	1.30
A5/2	889	728	1.22
A6/1	1340	1047	1.28
A6/2	1372	1059	1.30

Mean P_E/P_C = 1.39

Standard deviation P_E/P_C = 0.151

In the limit the cleats would pull out straight and all the tying forces would be carried by membrane action. It therefore seems appropriate to base this simple approach on the net section of the cleats in tension. A parametric study showed that a capacity of 0.6 x the yield capacity of the net section was always less than the design capacity (with γ_m = 1.25) determined by the large displacement analysis above. This study covered all the specimens tested, together with representative standard connections. The results are summarised in Table B.3 in which the calculations for the test specimens are based on measured geometry and properties; those for the standard connections marked with an * are based on nominal geometry and design strengths.

Because the simple tension capacity is based on yield strength while the design capacity is based on ultimate tensile strength, the empirical coefficient 0.6 may only be used for S275 steel. For S355 Steel it should be:

$$0.6 \times \frac{275}{355} \times \frac{490}{430} = 0.53$$

For simplicity 0.5 is therefore adopted for S355 steel

Table B.3 Comparison between design capacity and simple tension model

TEST NO.	DESIGN CAPACITY P_D (= P_C / 1.25) (kN)	SIMPLE TENSION MODEL P_T (kN)	P_D / P_T
A1/1	98	71	1.38
A1/2	100	71	1.41
A2	75	71	1.06
A3/1	265	215	1.23
A3/2	260	211	1.23
A4/1	443	348	1.27
A4/2	440	352	1.25
A5/1	595	546	1 .09
A5/2	583	535	1 .09
A6/1	838	752	1 .11
A6/2	848	763	1 .11
1 Bolt Standard*	106	79	1 .34
4 Bolt Standard*	429	317	1.35
7 Bolt Standard*	750	554	1.35

* standard connections based on nominal geometry and design strengths

B.6 Loss of effectiveness of outer hinges

As shown in Figure B.3 for the outer hinges the full length of the hinge is not developed where the bolt is a long way from the end(s) of the cleat. In principle the same loss of effectiveness could occur between bolts with a large pitch, p. The shape of the yield lines actually formed suggests that the limit for the hinge to be fully developed is that $e_c \leq 2e$ for the ends of the cleat and $p \leq 2e$ for the region between bolts. Since these limits are readily met for most practical connections they were adopted for the design model. It is recognised that these are more conservative than conventional rules but are adopted nonetheless because the latter are not intended to accommodate these gross deformations.

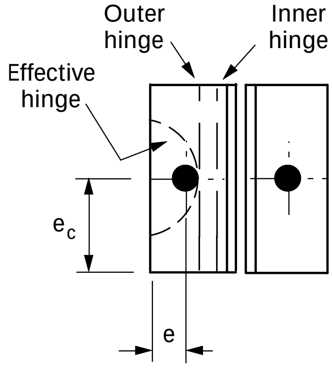


Figure B.3 Plastic hinge positions in double angle web cleat under tension

APPENDIX C LARGE DISPLACEMENT ANALYSIS FLEXIBLE END PLATES

LARGE DISPLACEMENT ANALYSIS BASED ON ULTIMATE LOAD TESTS

C.1 Behaviour

Figure C.1 shows the deformed shape of an end plate which has been subjected to large deformation. The noteworthy features are:

i) Considerable deformation can arise, but only if there is rotation of the hinges at the toes of the welds. These regions may well have suffered some embrittlement from fast cooling after welding. The minimum deformation observed in the test series was 8mm.

ii) These displacements do not reduce the eccentricities within the connection but they do offer an alternative membrane path for some of the tying force. However, this membrane restraint is only available if the end plate is bolted to a more substantial plate or flange. For a general solution this membrane action therefore has to be ignored.

iii) There are four critical sections in the plate. Since membrane action is being discounted these need only be considered under moment. (Moment/shear interaction will exist but need not be considered because the applied shear is only a small proportion of shear capacity.)

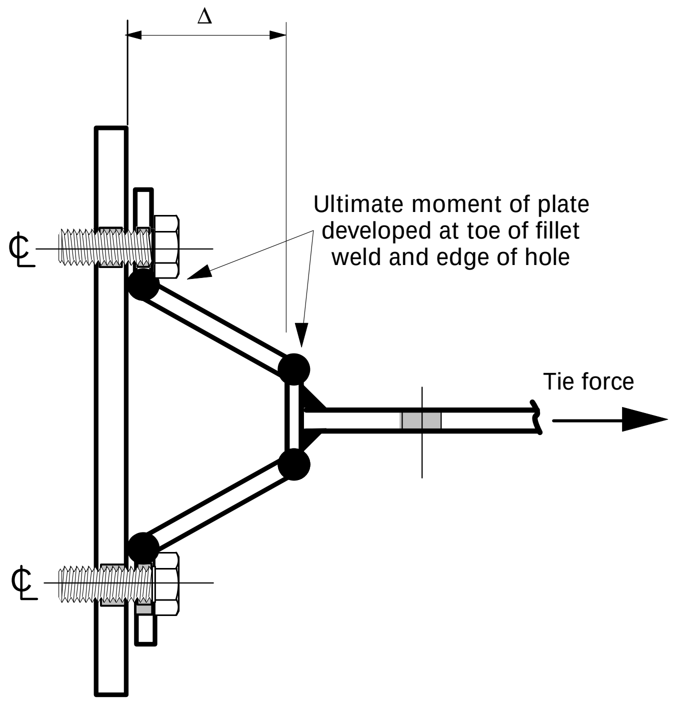


Figure C.1 End plate under tension

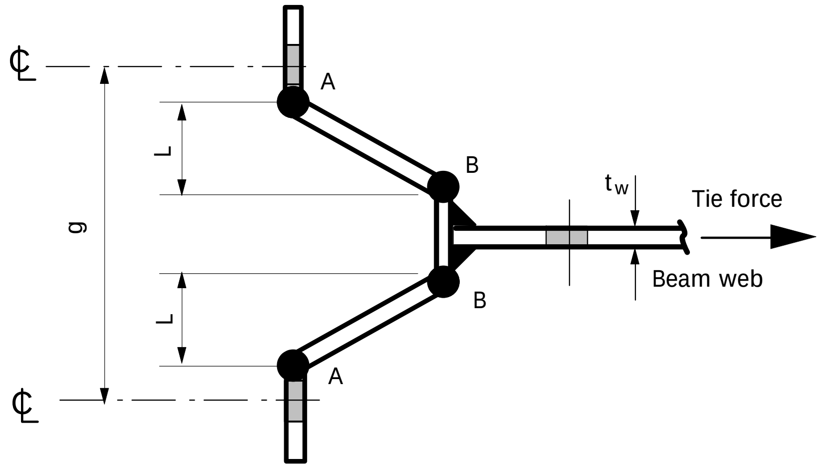


Figure C.2 Model of an end plate under tension

C.2 Method of Analysis

Figure C.2 presents a simple model that discounts any membrane action. It was originally put forward as giving the closest correlation with observed behaviour[47]. It is confirmed by subsequent experimental studies[5].

L is the distance between the plastic hinges, given by

$$L = 1/2\,(g - t_w - 2 \times \text{weld size} - D_h)$$

In recognition of the extremely large strains developed at the plastic hinge locations it is appropriate that the plastic moment capacity of the end plate (M_{ult}) should be based on the ultimate tensile strength of the plate material (U_s) divided by an appropriate γ_m rather than on its yield strength. In this instance, the outer hinges form away from the bolt centre line at the edges of the bolt hole.

It is therefore not necessary to make any deduction for bolt holes. 'The effective lengths of the hinges L_{eA} and L_{eB} are therefore the full lengths less any allowance for loss of effectiveness due to excessive end distances or bolt pitch. [This is discussed In more detail in C.6].

Thus the moment capacity is given by

$$M_u = \frac{U_s}{\gamma_m} \times \frac{L_e t_p^{\,2}}{4}$$

Hence:

$$\text{Tensile capacity} = \frac{2(M_{uA} + M_{uB})}{L}$$

C.3 Experimental Verification

A programme of tests was carried out to verify this method of analysis. It is fully reported elsewhere[5]. It comprised nine tests on typical connections. Table C.1 presents a simple summary of the results.

C.4 Comparison with Analysis

The specimens were initially analysed using the method outlined above with measured geometry and mechanical properties. The results are presented in Table C.2.

Although the coefficients P_E/P_C vary considerably it is clear that the design procedure is conservative. The reason for the variability is probably the unquantified and variable membrane action discussed above.

C.5 Design Method

This design method is considerably simpler than the iterative approach that is necessary for web cleats. It may therefore be adopted directly for design, with γ_m set to 1.25, as is considered appropriate for a rupture condition.

C.6 Loss of effectiveness of hinges

As shown in Figure C.3, where end plates are resisting tying forces, both inner and outer hinges may lose effectiveness. For the outer hinges the top and bottom extremities of the end plate may not be mobilised, with the yield lines curling around and finishing along the long edges. With the inner hinges there is a concentration of bending rotation opposite the bolts. In the presence of partial embrittlement from the welding this may limit the spread of bending action prior to rupture of the plate.

The shape of the outer yield lines suggests that the limit for the hinge to be fully developed should be $e_p \leq b$ for the ends of the plate and $p \leq 2b$ for the region between bolts where p is the bolt pitch, b is the distance from the outer hinge to the edge of the plate and e_p is the end distance. Corresponding expressions for the inner hinges are $e_p \leq L$ and $p \leq 2L$ respectively.

Table C.1 Summary of test results - Flexible end plate

TEST NO.	NO. OF BOLTS IN TENSION	MAXIMUM LOAD (kN)	DEFORMATION PRIOR TO FAILURE (mm)	FAILURE MODE
B1/1	2	211	41	Bearing failure in end plate
B1/2	2	203	39	Bearing failure in end plate
B2	2	212	Inst. failed	Fracture of end plate at weld toe
B3/1	6	404	20	Fracture of end plate at weld toe
B3/2	6	420	19	Fracture of end plate at weld toe
B4/1	10	600	16	Fracture of end plate at weld toe
B4/2	10	575	17	Fracture of end plate at weld toe
B5/1	14	980	30	Fracture of end plate at weld toe
B5/2	14	937	21	Fracture of end plate at weld toe

Table C.2 Comparison between experimental results and design model

TEST NO.	MAXIMUM LOAD P_E (kN)	CALC. CAPACITY P_C (kN)	P_E/P_C
B1/1	211	73	2.89
B1/2	203	74	2.74
B2	212	130	1.63
B3/1	404	289	1.40
B3/2	420	312	1.35
B4/1	600	432	1.39
B4/2	575	407	1.41
B5/1	980	570	1.72
B5/2	937	555	1.69

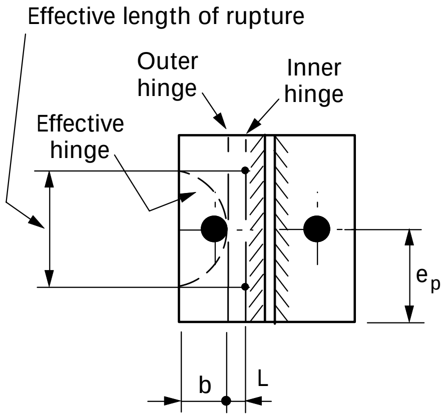


Figure C.3 Plastic hinge positions in end plate under tension

APPENDIX D DESIGN FOR PRYING WHEN RESISTING TYING FORCES

BS 5950-1:2000[1] (clause 6.3.4) gives two methods for calculating the tension capacity of a connection. The simple method (clause 6.3.4.2) ignores prying action and uses nominal bolt strength of $0.8p_t$. The more exact method (clause 6.3.4.3) takes account of prying action and uses the full tension strength p_t of the bolts. It is not appropriate to use the simple method for tying force calculations. This is because the bending checks are based on the ultimate bending strength of the critical cross-sections and the analysis relies on gross deformations which will not permit any alleviation of prying action by bolt yielding.

Bolt tensions are not generally very high. What is required is an effective upper bound on prying forces, which can be used in conjunction with P_{bult}/γ_m. As with other aspects of this design approach, a γ_m of 1.25 is used for a rupture condition.

Web cleats

For standard cross centres of 140mm, a minimum beam web of 6mm, a minimum web cleat thickness of 8mm, a root radius of 8mm and in accordance with Figure D.1,

$$l_1 + l_3 = \frac{140}{2} - \frac{6}{2} - 8 - 8 = 51\text{mm}$$

For maximum prying ,l_1/l_3 should be a maximum. This ratio is proportional to the relative bending strength at A and B respectively. Maximum prying occurs when bolts are at their greatest practical spacing. This is taken as 4d, where d is the bolt diameter. Thus:

$$l_1 : l_3 = 80 - 22 : 80 = 58 : 80$$

$$\text{Hence } l_1 = 51 \times \frac{58}{58 + 80} = 21\text{mm}$$

The above ignores displaced geometry effects; these are taken into account by assuming the displacement, $\Delta = 30$mm. (In the tests, Δ always exceeded 30mm prior to failure.) The lever arm becomes, $l_1 \cos\varphi$, where:

$$\varphi = \tan^{-1} 30/51$$

$$\text{hence, } \varphi = 30.5°$$

$$\text{lever arm} = 21 \cos 30.5° = 18\text{mm}$$

The prying force will act at a lever arm of $2t_c$ from the bolt centre line, ie. 16mm with $t_c = 8$mm.

$$\text{Hence, prying ratio} = \frac{16 + 18}{16} = 2.13$$

The ultimate tensile strength of 8.8 bolts is 784N/mm^2, hence nominal bolt stress should not exceed,

$$\frac{784}{2.13 \times 1.25} = 295\text{N/mm}^2 \text{ for 8.8 bolts}$$

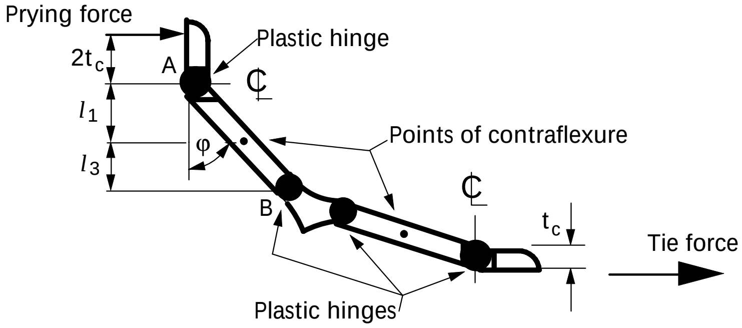


Figure D.1 Web angle cleat under tension

End plates

For standard cross-centres of 140mm, a minimum beam web of 6mm, a minimum end plate thickness of 8mm, an 8mm weld (also noting that these hinges form at the edge of the hole), and in accordance with Figure D.2,

$$l_1 + l_3 \quad = \quad \frac{140}{2} \; - \; \frac{6}{2} - 8 - 11 \quad = \quad 48\text{mm}$$

Since both capacities are based on gross sections, $l_1 = l_3$

$$\text{Hence, } l_1 \quad = \quad \frac{48}{2} \quad = \quad 24\text{mm}$$

The prying force may be considered to act at a lever arm of $(D_h/2 + 2t_p)$ from the outer plastic hinge, ie. 27mm with t_p = 8mm.

$$\text{Hence, prying ratio} \quad = \quad \frac{27 + 24}{27} \quad = \quad 1.89$$

The ultimate tensile strength of 8.8 bolts is 784N/mm^2, hence the nominal bolt stress should not exceed

$$\frac{784}{1.89 \times 1.25} \quad = \quad 332\text{N/mm}^2 \text{ for Grade 8.8 bolts}$$

Thus an appropriate simple design check for both connections is to ensure that the nominal tensile stress in bolts does not exceed 300 N/mm^2.

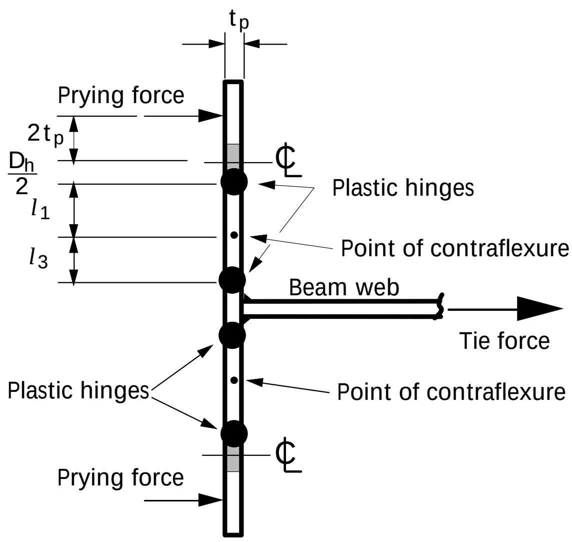


Figure D.2 End plate under tension

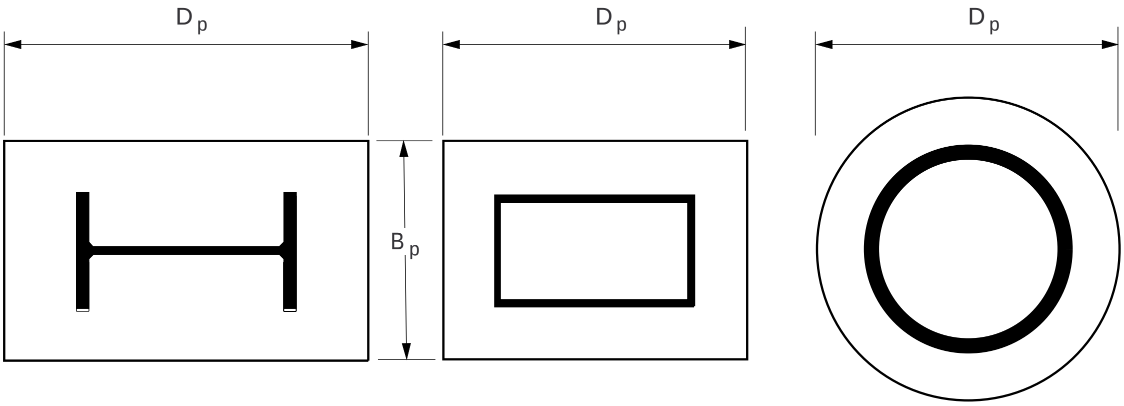
APPENDIX E

BASE PLATE DESIGN
(BS 5950: Part 1: 1990 METHOD)

This semi-empirical method may be used to determine initial plate dimensions. It assumes two way bending in the plate at the corner of **I** and RHS. It ignores conditions along the column web and flanges where, dependent upon the column depth/width ratio, the moments can be much greater due to cantilever action. The method can therefore, in certain circumstances, produce plate thicknesses less than would be obtained from the effective area method.

The effective area method should always be used for final design. It is described in Section 8.4. Full design procedures are given in Section 8.5 of this design guide.

STEP 1	Base plate size - BS 5950: Part 1: 1990 method



$A_p \geq A_{req}$

A_{req} = required area of baseplate = $\dfrac{F_c}{0.4 f_{cu}}$

A_p = area of base plate = $B_p\, D_p$ for UB, UC and RHS columns

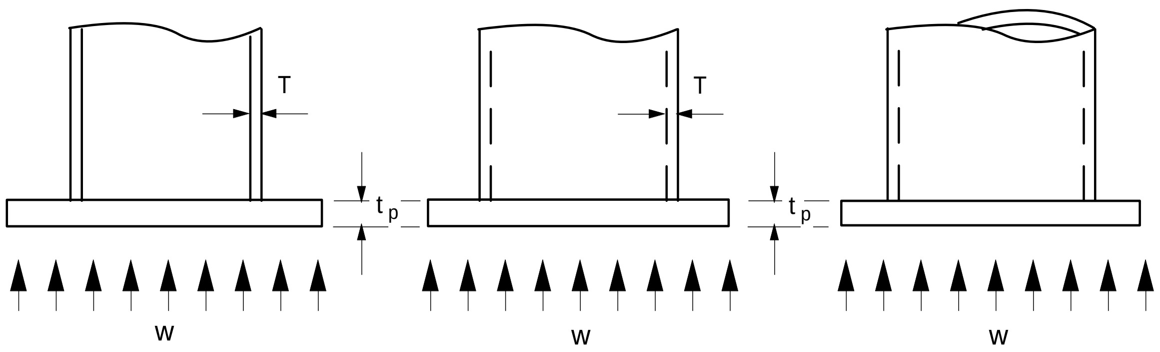
= $\dfrac{\pi D_p^2}{4}$ for CHS column

F_c = compressive force due to factored loads

$0.4\, f_{cu}$ = bearing strength (BS 5950: Part 1: 1990, cl. 4.13.1)

f_{cu} = the smaller of the characteristic cube strength at 28 days of the bedding material or the concrete base

STEP 2	Base plate thickness - BS 5950: Part 1: 1990 method



For UC, UB, RHS, channel and box columns

Note: This check should not be used when a < 5T
or with UB sections used as columns.

$$t_p \geq \left[\frac{2.5\,w}{p_{yp}} \left(a^2 - 0.3b^2\right) \right]^{1/2}$$

but not less than the flange thickness of the supported column

a = the greater projection of the plate beyond the column

b = the lesser projection of the plate beyond the column

w = the pressure on the underside of the plate assuming a uniform distribution

p_{yp} = design strength of the plate
≤ $270 N/mm^2$

For CHS columns

$$t_p \geq \left[\frac{w\,D_p}{2.4\,p_{yp}} \left(D_p - 0.9D\right) \right]^{1/2}$$

D_p = length or diameter of base plate

D = diameter of the column

APPENDIX F THERMAL DRILLING OF RHS

F.1 Introduction

Flowdrilling or Formdrilling is basically a thermal process which makes a hole through the wall of a structural hollow section without the removal of metal normally associated with a drilling process. The formed hole is then threaded by the use of a roll thread forming tool, leaving a threaded hole that will accept a standard fully threaded bolt.

F.2 Drilling tool and Process (see Figure F.1 & F.2)

The initial hole is made by the thermal drilling tool which consists of a tungsten carbide bit held in a taper collet adaptor (Figure F.1). The tool can be used in a conventional drilling machine or CNC machine as found in steelwork contractors works, provided that it has adequate power and spindle speed.

The process is shown in Figure F.2. During the 1st stage the tungsten carbide bit is brought into contact with the RHS wall where it generates sufficient heat to soften the steel. The bit is then advanced through the wall and in so doing the metal is redistributed (or flows) to form an internal bush. As well as drilling the initial hole, the tool is fitted with the means of removing any surplus material which may arise on the outside of the RHS section (see Figure F.2). The cycle time is similar to that for conventional drilling. However, if done on CNC machines the feed rate can be slow at the beginning, rapidly increasing as the material softens to improve efficiency.

The 2nd and final stage is to tap the formed bush. This is done by roll threading the bush with a Coldform tap. The complete cycle is shown in Figure F.2

F.3 Application & limitations

Flowdrill or Formdrill connections to RHS columns can be made using either double angle cleats or flexible end plates as shown in Figures F.3 and F.4.

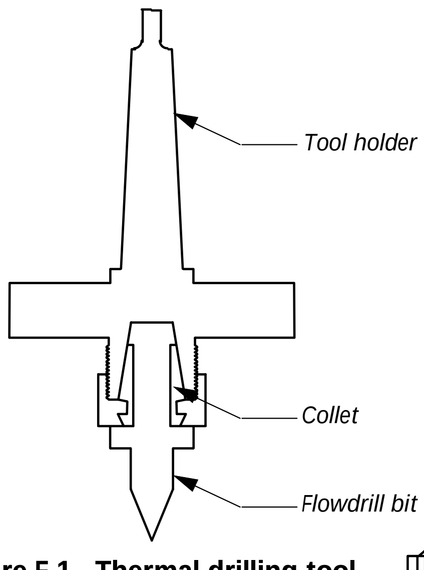


Figure F.1 Thermal drilling tool

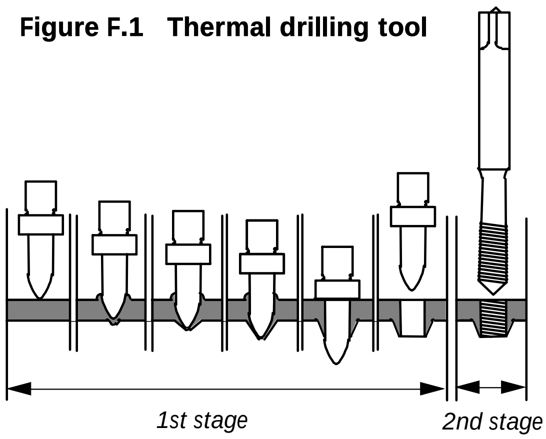


Figure F.2 The process

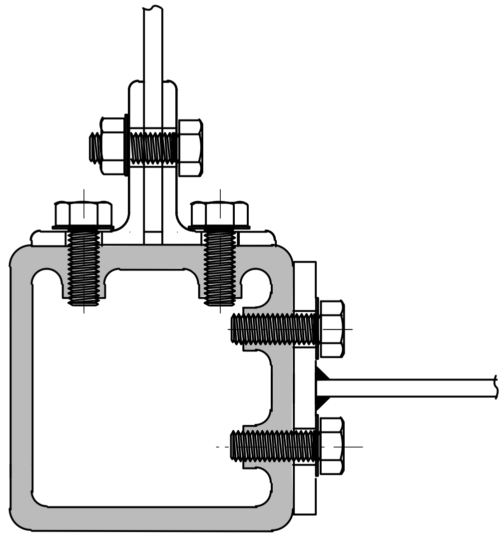


Figure F.3 Angle cleat and flexible end plate connections

When using Flowdrill or Formdrill connections it must be noted that:

- if used at locations exposed to the weather they should not be considered as water tight unless special protective measures are taken.
- they are not suitable for use with pre-galvanised materials.

F.4 Further information

Further information on thermal drilling, including drilling machine parameters and tool sizes can be obtained from the companies given below.

Flowdrill (U.K.) Limited
Unit 7, Hopewell Business Centre
105 Hopewell Drive
Chatham
Kent. ME5 7NP
Tel: 01634 309422

Formdrill
Robert Speck Ltd
Little Ridge
Whittlebury Road
Silverstone
Northants. NN12 8UD
Tel: 01327 857307

For detailing requirements and tension capacity of bolts see Tables H.60 and H.55 of the yellow pages.

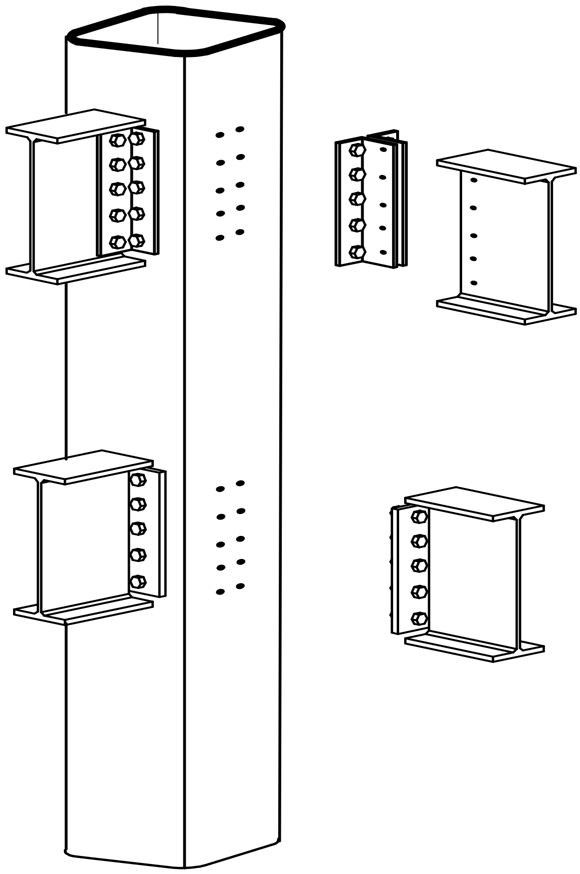


Figure F.4 Examples of Flowdrill connections to an RHS column

APPENDIX G HOLLO-BOLT - JOINTING OF RHS

G.1 Introduction

The Lindapter Hollo-Bolt is a patented method of fixing to RHS or to steelwork where access is only available from one side. The Hollo-Bolt is a pre-assembled three or five part fitting depending on the size of the Hollo-Bolt (see Table G.1).

TABLE G.1	**Hollo-Bolt Type**
Bolt Size	Hollo-bolt tpye
M8	3 Part
M10	3 Part
M12	3 Part
M16	5 Part
M20	5 Part

The pre-assembled Hollo-Bolt unit (Figure G.1) is inserted through normal tolerance holes in both the fixture and the RHS. As the bolt is tightened the cone is drawn into the body, spreading the legs, and forming a secure fixing. Once installed only the Hollo-Bolt head and collar are visible. Figures G.2 and G.3 show the Hollo-Bolt in the installed condition

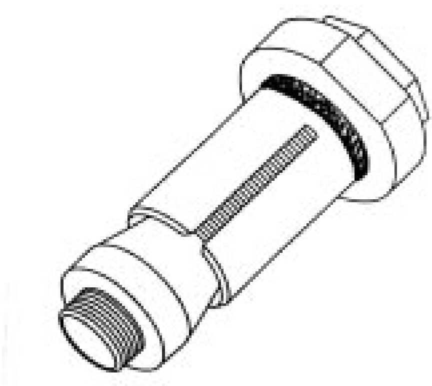


Figure G.1 Pre-assembled Hollo-Bolt unit

The M16 and M20 Hollo-Bolts have a rubber collapse mechanism under the collar which maximises the clamping force of the fastener.

Hollo-Bolt connections between universal beams and RHS columns can be made using either double angle cleats or flexible end plates as shown in Figure G.4.

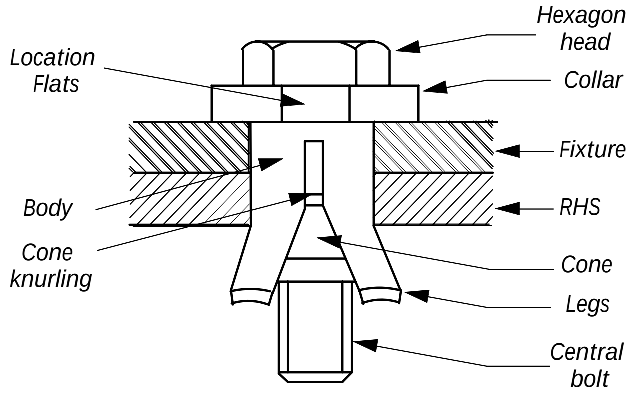


Figure G.2 Installed 3-part Hollo-Bolt (M8, M10 & M12)

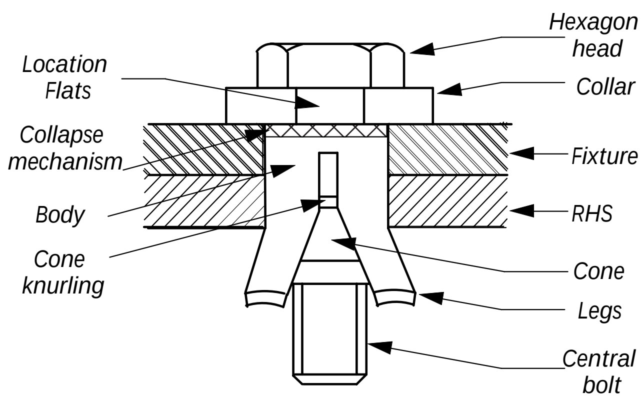


Figure G.3 Installed 5-part Hollo-Bolt (M16 & M20)

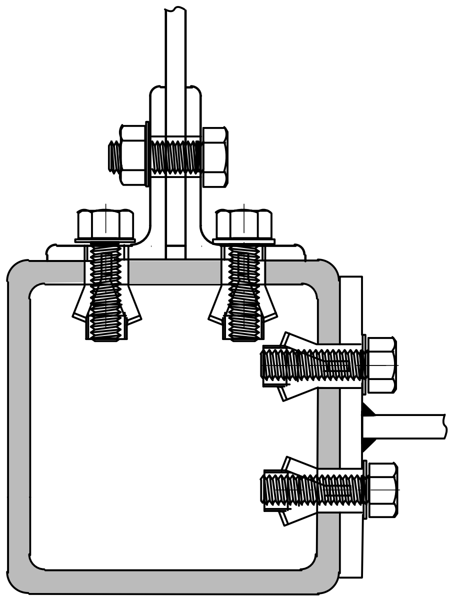


Figure G.4 Hollo-Bolts used for angle cleat and flexible end plate connections

G.2 Installation

Hollo-Bolt uses a plain drilled hole which can be made on site or in the fabrication shop using all normal drilling equipment.

The only tools required to fit Hollo-Bolt are two spanners - an open ended spanner to hold the collar and a torque wrench to tighten the central bolt. Alternatively a power operated electric hand tool can be used. The required tightening torques are given in Table G.2.

Should the steelwork need to be adjusted, the fixing can simply be removed and the hole reused with another Hollo-Bolt.

Table G.2 Required tightening torque

Bolt size	Tightening torque (Nm)
M8	23
M10	45
M12	80
M16	190
M20	300

G.3 Material Options

The standard product is manufactured from mild steel and is electro-zinc plated with the addition of JS500 1000 hour saltspray corrosion protection. The central fastener is a grade 8.8 bolt.

For special applications, the Hollo-Bolt is available manufactured from 316 stainless steel, with a grade A4-80 central bolt. This will not be a stocked item, and would be manufactured to order.

G.4 Sealing Options

In certain applications, it may be necessary to seal the Hollo-Bolt to prevent ingress of water or other corrosive agents. For details of sealing options available, please contact Lindapter.
Special Options (manufactured to order)

- Stainless steel
- Button head setscrew
- Countersunk setscrew/body
- Socket head capscrew
- Special body length

Further information on Hollo-Bolt is available from:

Lindapter International
Lindsay House
Brackenbeck Road
Bradford
West Yorkshire
England
BD7 2NF.

Telephone: 01274 5214444

Fax: 01274 5211130.

For detailing requirements and capacity tables for the Hollo-Bolt see Tables H.61 and H.56 of the yellow pages.

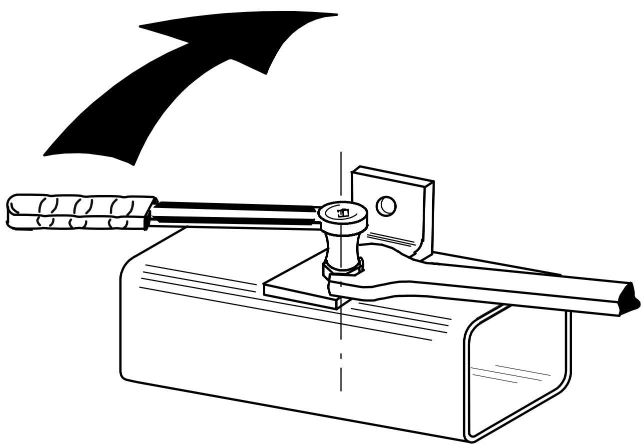


Figure G.5 Installation of Hollo-Bolt

APPENDIX H

Capacity Tables
Dimensions for Detailing
General Data

APPENDIX H CAPACITY TABLES, DIMENSIONS FOR DETAILING AND GENERAL DATA

Table H.1

Standard Fittings - Marks

The fitting range adopted in tables H.2 to H.5 and capacity tables H.6 to H.30 have identification marks as follows:

FITTING

Size and Hole Dimensions

E	B	*n*

CONNECTION

Types:

C = Double Angle Web cleat

E = Flexible End Plate

F = Fin Plate

NUMBER

Rows of bolts

Table H.2

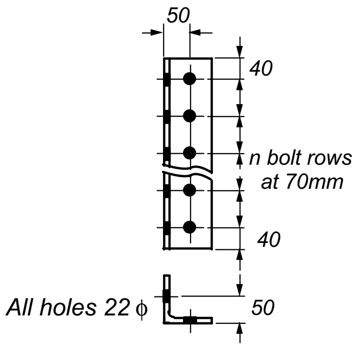
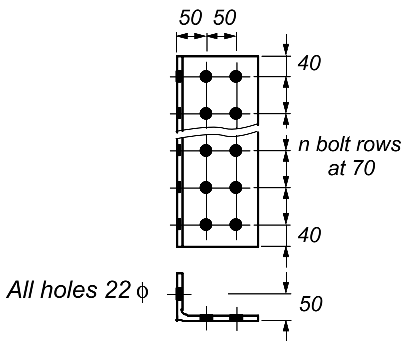
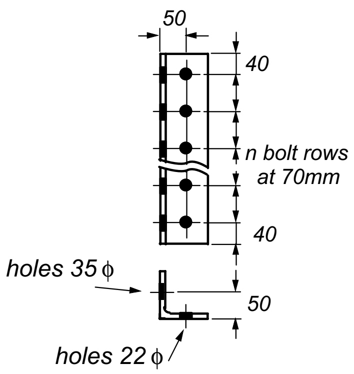
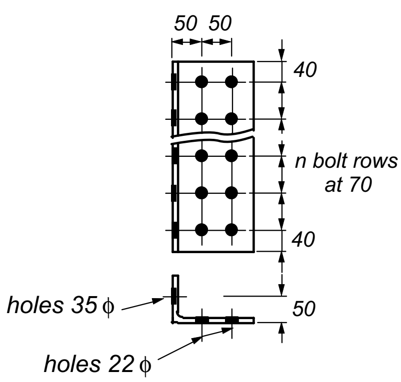
Standard Fittings - Details - Double Angle Web Cleats			
Connection	**Mark**	**Fitting Section**	**Dimensions and diameter of holes**
Cleats with single line of ordinary or Flowdrill bolts	CA*n*	90 x 90 x 10 EA	
Cleats with double line of ordinary or Flowdrill bolts	CB*n*	150 x 90 x 10 UEA	
Cleats with single line of Hollo-Bolts	CC*n*	90 x 90 x10 UEA	
Cleats with double line of Hollo-Bolts	CD*n*	150 x 90 x 10 UEA	

Table H.3

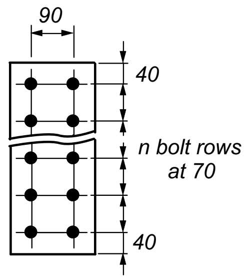
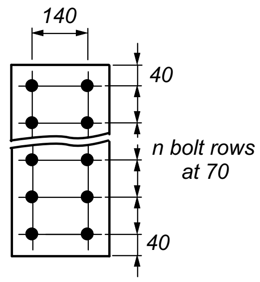
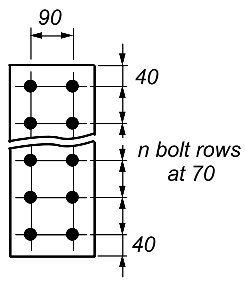
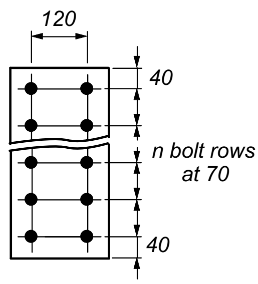
<table>
<tr><th colspan="4">Standard Fittings - Details - Flexible End Plates</th></tr>
<tr><th>Connection</th><th>Mark</th><th>Fitting Section</th><th>Dimensions and diameter of holes</th></tr>
<tr><td>Flexible End Plate
with ordinary or
Flowdrill bolts
For beams ≤ 457x191UB</td><td>EAn</td><td>150 x 8 Flat</td><td>All holes 22 ϕ

</td></tr>
<tr><td>Flexible End Plate
with ordinary or
Flowdrill bolts
For beams > 457x191UB</td><td>EBn</td><td>200 x 10 Flat</td><td>All holes 22 ϕ

</td></tr>
<tr><td>Flexible End Plate
with Hollo-Bolts
For beams ≤ 457x191UB</td><td>ECn</td><td>180 x 8 Flat</td><td>All holes 35ϕ

</td></tr>
<tr><td>Flexible End Plate
with Hollo-Bolts
For beams > 457x191UB</td><td>EDn</td><td>200 x 10 Flat</td><td>All holes 35ϕ

</td></tr>
</table>

Table H.4

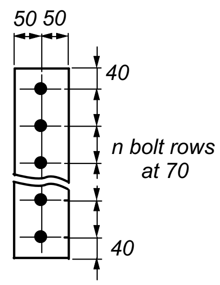
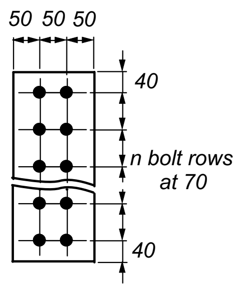
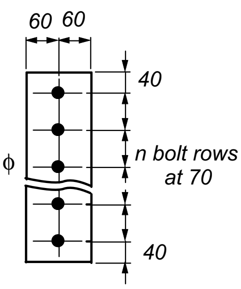
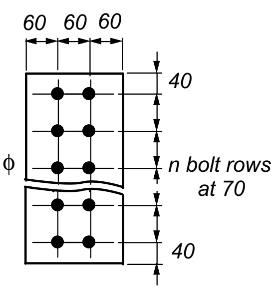
Standard Fittings - Details - Fin Plates			
Connection	**Mark**	**Fitting Section**	**Dimensions and diameter of holes**
Fin Plate with single line of ordinary bolts For beams ≤ 610 x 305 UB	FA*n*	100 x 10 Flat	*All holes 22 ϕ* 
Fin Plate with double line of ordinary bolts For beams ≤ 610 x 305 UB	FB*n*	150 x10 Flat	*All holes 22 ϕ* 
Fin Plate with single line of ordinary bolts For beams > 610 x 305 UB	FC*n*	120 x 10 Flat	*All holes 22 ϕ* 
Fin Plate with double line of ordinary bolts For beams > 610 x 305 UB	FD*n*	180 x 10 Flat	*All holes 22 ϕ* 

Table H.5

Standard Fittings - Masses

Angle Web Cleats - Single vertical line of bolts (CA or CC)

Mark	CA11	CA10	CA9	CA8	CA7	CA6	CA5	CA4	CA3	CA2
Size	90 x 90 x 10 Equal Angle (13.40 kg/m)									
Length (mm)	780	710	640	570	500	430	360	290	220	150
Mass each (kg)	10.45	9.51	8.58	7.64	6.70	5.76	4.82	3.89	2.95	2.01

Angle Web Cleats - Double vertical line of bolts (CB or CD)

Mark	CB11	CB10	CB9	CB8	CB7	CB6	CB5	CB4	CB3	CB2
Size	150 x 90 x 10 Unequal Angle (18.20 kg/m)									
Length (mm)	780	710	640	570	500	430	360	290	220	150
Mass each (kg)	14.20	12.92	11.65	10.37	9.10	7.83	6.55	5.28	4.00	2.73

Flexible End Plates - Light

Mark	EA5	EA4	EA3	EA2	EC5	EC4	EC3	EC2
Size	150 x 8 Flat (9.42 kg/m)				180 x 8 (11.30 kg/m)			
Length (mm)	360	290	220	150	360	290	220	150
Mass each (kg)	3.39	2.73	2.07	1.41	4.07	3.28	2.48	1.70

Flexible End Plates - Heavy (EB or ED)

Mark	EB11	EB10	EB9	EB8	EB7	EB6	EB5
Size	200 x 10 Flat (15.7 kg/m)						
Length (mm)	780	710	640	570	500	430	360
Mass each (kg)	12.25	11.15	10.05	8.95	7.85	6.75	5.65

Fin Plates - Single vertical line of bolts

Mark	FA7	FA6	FA5	FA4	FA3	FA2	FC11	FC10	FC9	FC8	FC7	FC6
Size	100 x 10 Flat (7.85 kg/m)						120 x 10 Flat (9.42 kg/m)					
Length (mm)	500	430	360	290	220	150	780	710	640	570	500	430
Mass each (kg)	3.93	3.38	2.83	2.28	1.73	1.18	7.35	6.69	6.03	5.37	4.71	4.05

Fin Plates - Double vertical line of bolts

Mark	FB7	FB6	FB5	FB4	FB3	FB2	FD11	FD10	FD9	FD8	FD7	FD6
Size	150 x 10 Flat (11.8 kg/m)						180 x 10 Flat (14.1 kg/m)					
Length (mm)	500	430	360	290	220	150	780	710	640	570	500	430
Mass each (kg)	5.90	5.07	4.25	3.42	2.60	1.77	11.00	10.01	9.02	8.04	7.05	6.06

Table H.6

Explanatory notes - DOUBLE ANGLE WEB CLEATS
Use of Capacity Tables

The following notes refer to the **suffix** numbers given at the top of the column descriptions for Tables H.9 to H.16.

The check numbers refer to those listed in Table H.8 and described in Section 4.5 Design procedures.

The capacity tables are based on the standard details given in Table H.7.

All universal beam sections which are suitable for the minimum standard fitting size and the standard notch size are shown in the tables (i.e. if T + r > 50 mm, section is not included).

(1) SHEAR CAPACITY OF THE BEAM

The value given in { } is the shear capacity of the beam, given by $0.6p_yt_wD$.

(2) SHEAR CAPACITY OF THE CONNECTION

This is the critical value of the design checks for the 'supported beam side' of the connection. i.e. the minimum capacity from CHECKS 2, 3(i), 3(ii), 4 (i), 4(ii), 4(iii), 8 and 9(ii).

For connections with Ordinary or Flowdrill bolts, Connection Shear Capacities are given for Un-notched or Single notched and for Double notched beams.

For connections with Hollo-Bolts, connection shear capacities are tabulated for Un-notched beams only, as Hollo-Bolts are used only with RHS columns and Un-notched beams.

(3) CRITICAL DESIGN CHECK

The check which gives the critical value of shear capacity. See Table H.8 for the description of the checks.

(4) FITTINGS

The length and type of Standard Double Angle fittings. See Tables H.1 and H.2 for details.

(5) MINIMUM SUPPORT THICKNESS

This is the minimum thickness of supporting column or beam element, that is needed to carry the given **SHEAR CAPACITY** (Un-notched or Single Notched) of the connection**.** It is derived from CHECK 10 and e_t has conservatively been taken as 90 mm.

For a symmetrical two sided connection, the minimum support thickness would be twice the tabulated value.

(6) MAXIMUM NOTCH LENGTH ($C + T_1$)

These are maximum lengths of notches for single and double notched beams that can be accommodated if the beam is to carry the tabulated corresponding **SHEAR CAPACITY** of the connection.

It is assumed that the beam is **fully restrained** against lateral torsional buckling, and the notched lengths are derived from CHECKS 5 and 6.

To provide a simple check for double notched beams, it has been assumed that the remaining web depth, 'y' (see Table H.7) is the same as the angle cleat length.

* Indicates that the condition from CHECK 6, i.e. $d_{c2} \leq D/5$ is <u>not</u> satisfied.

For Hollo-Bolts,Tables H.13 to H.16, no values are tabulated for maximum notch lengths, since Hollo-Bolts are used only with RHS columns and un-notched beams.

(7) TYING CAPACITY

This is the critical value of the design checks for the 'supported beam' side of the connection, i.e. the minimum capacity from CHECKS 11, 12(i), 12(ii) and 13.

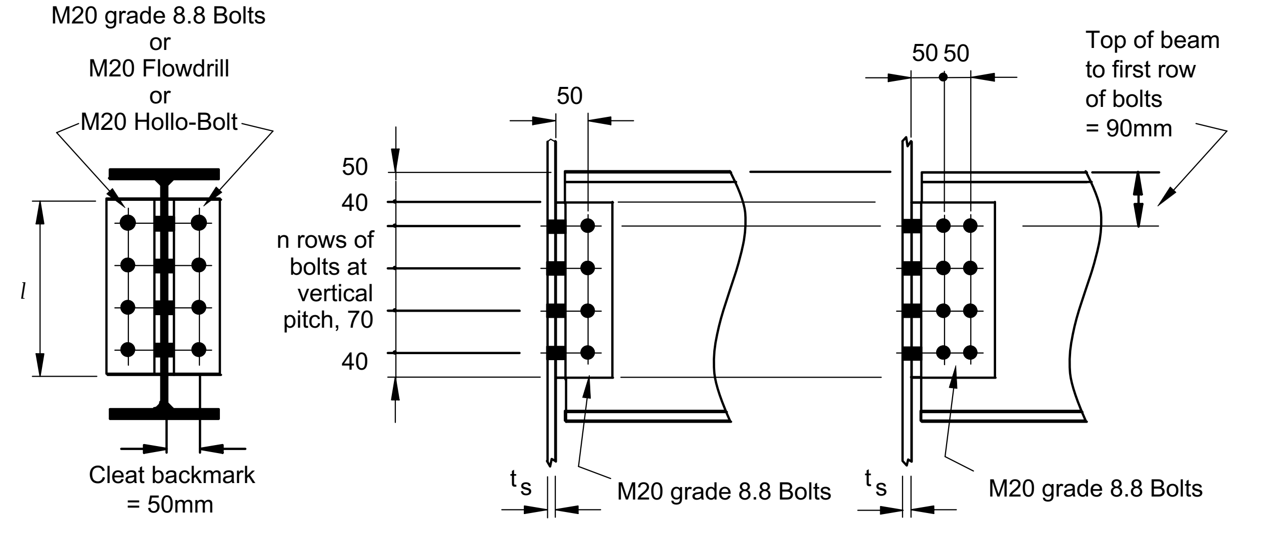
Values given in tables for CHECK 11 are those derived from the rigorous method of Appendix B and not the Simplified approach given in the design procedures. In CHECK 12 the supported beam is assumed to be double notched.

For Flowdrill connections, CHECK 13 will need to be carried out separately because the capacity is dependent on the thickness of the RHS.

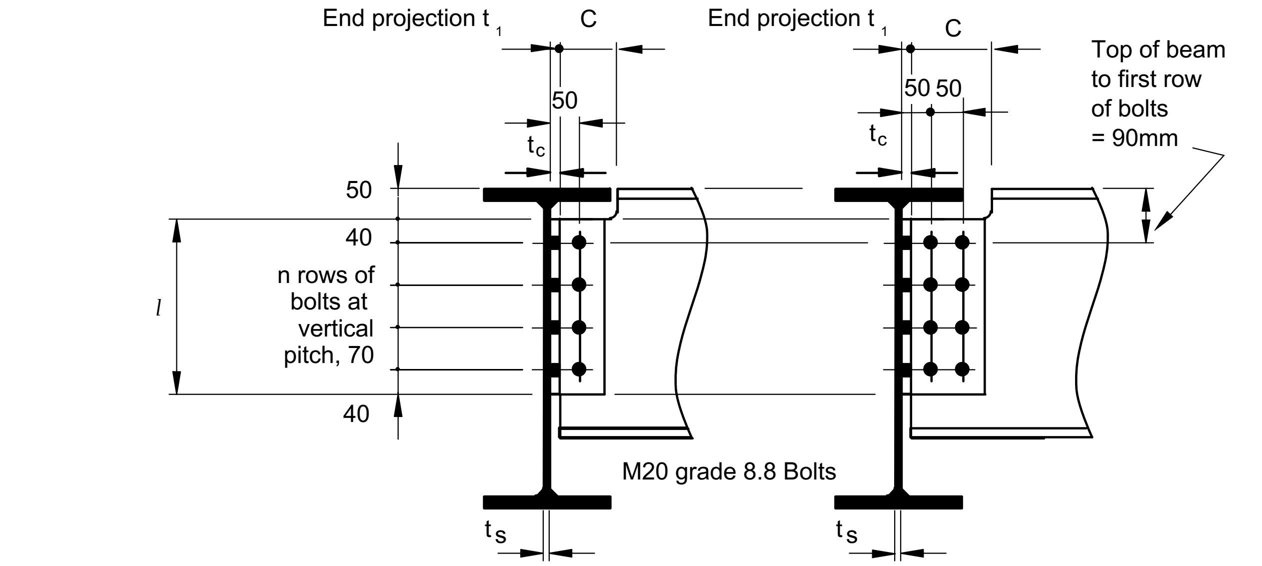
Separate checks will have to be carried out on the supporting members (see CHECKS 14 and 15).

Table H.7

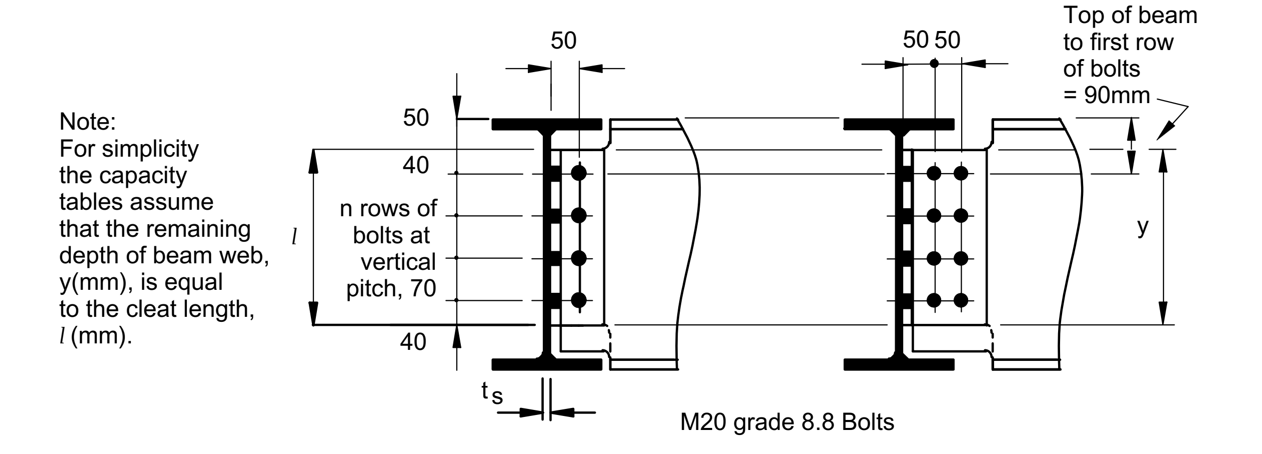
DOUBLE ANGLE WEB CLEATS
Standard details used in Capacity Tables



UN-NOTCHED BEAM



SINGLE NOTCHED BEAM



DOUBLE NOTCHED BEAM

Note: See Tables H.1 and H.2 Standard Fittings - Marks and details for Double Angle Cleats

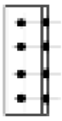
Table H.8

DOUBLE ANGLE WEB CLEATS Critical Check Description List			
		Check Number	**Description**
SHEAR CAPACITY	Supported beam side	2	Shear capacity of bolt group connected to supported beam
		3(i)	Shear capacity of angle cleats connected to supported beam
		3(ii)	Bearing capacity of angle cleats connected to supported beam
		4(i)	Shear capacity of supported beam at the connection
		4(ii)	Shear and bending interaction capacity of supported beam at the 2nd bolt line connection
	Supporting member side	4(iii)	Bearing capacity of supported beam web at the connection
		8	Shear capacity of bolt group connected to supporting member
		9(i)	Shear capacity of angle cleats connected to supporting member
STRUCTURAL INTEGRITY		9(ii)	Bearing capacity of angle cleats connected to supporting member
		11	Tension capacity of angle cleats (tables based on the rigorous approach of Appendix B)
		12(i)	Tension capacity of supported beam web at the connection
		12(ii)	Bearing capacity of supported beam web at the connection
		13	Tension capacity of bolts in presence of extreme prying

Note: This table only lists the critical checks.
For a full list of design checks and further information, see Section 4.5.

BEAM: S275
ANGLE CLEATS: S275
BOLTS: M20, 8.8

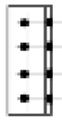
Table H.9



DOUBLE ANGLE CLEATS, ORDINARY or FLOWDRILL BOLTS

Single line of bolts

2 No. 90x90x10mm Equal Angles

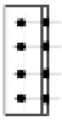


Beam Size { } [1]	Bolt Rows n	Un-notched or Single Notch: Shear[2] Capacity kN	Un-notched or Single Notch: Critical[3] Design Check	Double Notch: Shear[2] Capacity kN	Double Notch: Critical[3] Design Check	Fitting[4] Angle Cleats: Length ℓ mm	Fitting[4] Angle Cleats: Mark	Min. Support[5] Thickness: S275 mm	Min. Support[5] Thickness: S355 mm	Max Notch[6] Length $c+t_1$: Single mm	Max Notch[6] Length $c+t_1$: Double mm	Tying[7]: Capacity kN	Tying[7]: Critical Check
914x305x253	11	1650	4 (iii)	1650	4 (iii)	780	CA11	8.1	6.8	517	155	1200	11
{2530 kN}	10	1480	4 (iii)	1480	4 (iii)	710	CA10	8.1	6.7	575	154	1090	11
	9	1320	4 (iii)	1320	4 (iii)	640	CA9	8.0	6.6	648	152*	984	11
	8	1150	4 (iii)	1150	4 (iii)	570	CA8	7.8	6.5	742	150*	877	11
914x305x224	11	1520	4 (iii)	1520	4 (iii)	780	CA11	7.5	6.3	505	155	1200	11
{2300 kN}	10	1360	4 (iii)	1360	4 (iii)	710	CA10	7.4	6.2	561	154	1090	11
	9	1210	4 (iii)	1210	4 (iii)	640	CA9	7.3	6.1	632	152*	984	11
	8	1060	4 (iii)	1060	4 (iii)	570	CA8	7.2	6.0	724	150*	877	11
914x305x201	11	1440	4 (iii)	1440	4 (iii)	780	CA11	7.1	5.9	489	155	1200	11
{2170 kN}	10	1290	4 (iii)	1290	4 (iii)	710	CA10	7.0	5.9	543	154	1090	11
	9	1150	4 (iii)	1150	4 (iii)	640	CA9	6.9	5.8	612	152*	984	11
	8	1000	4 (iii)	1000	4 (iii)	570	CA8	6.8	5.7	686	150*	877	11
838x292x226	10	1380	4 (iii)	1380	4 (iii)	710	CA10	7.5	6.3	495	154	1090	11
{2180 kN}	9	1230	4 (iii)	1230	4 (iii)	640	CA9	7.4	6.2	557	152	984	11
	8	1070	4 (iii)	1070	4 (iii)	570	CA8	7.3	6.1	638	150*	877	11
838x292x194	10	1260	4 (iii)	1260	4 (iii)	710	CA10	6.8	5.7	476	154	1090	11
{1960 kN}	9	1120	4 (iii)	1120	4 (iii)	640	CA9	6.8	5.7	536	152	984	11
	8	977	4 (iii)	977	4 (iii)	570	CA8	6.6	5.6	614	150*	877	11
838x292x176	10	1200	4 (iii)	1200	4 (iii)	710	CA10	6.5	5.5	464	154	1090	11
{1860 kN}	9	1070	4 (iii)	1070	4 (iii)	640	CA9	6.4	5.4	522	152	984	11
	8	930	4 (iii)	930	4 (iii)	570	CA8	6.3	5.3	598	150*	877	11
762x267x197	9	1190	4 (iii)	1190	4 (iii)	640	CA9	7.2	6.0	449	152	984	11
{1910 kN}	8	1040	4 (iii)	1040	4 (iii)	570	CA8	7.0	5.9	514	150	877	11
	7	886	4 (iii)	886	4 (iii)	500	CA7	6.9	5.8	602	148*	769	11
762x267x173	9	1090	4 (iii)	1090	4 (iii)	640	CA9	6.6	5.5	436	152	984	11
{1730 kN}	8	950	4 (iii)	950	4 (iii)	570	CA8	6.5	5.4	500	150	877	11
	7	812	4 (iii)	812	4 (iii)	500	CA7	6.3	5.3	585	148*	769	11
762x267x147	9	974	4 (iii)	974	4 (iii)	640	CA9	5.9	4.9	422	152	984	11
{1530 kN}	8	851	4 (iii)	851	4 (iii)	570	CA8	5.8	4.8	484	150	877	11
	7	727	4 (iii)	727	4 (iii)	500	CA7	5.6	4.7	566	148*	769	11
762x267x134	9	913	4 (iii)	913	4 (iii)	640	CA9	5.5	4.6	430	181	984	11
{1490 kN}	8	797	4 (iii)	797	4 (iii)	570	CA8	5.4	4.5	493	175	877	11
	7	681	4 (iii)	681	4 (iii)	500	CA7	5.3	4.4	502	169*	769	11

For guidance on the use of tables see Explanatory notes in Table H.6

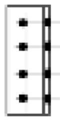
BEAM: S275
ANGLE CLEATS: S275
BOLTS: M20, 8.8

Table H.9 *Continued*



DOUBLE ANGLE CLEATS, ORDINARY or FLOWDRILL BOLTS

Single line of bolts
2 No. 90x90x10mm Equal Angles

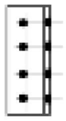


Beam Size { } [1]	Bolt Rows n	Un-notched or Single Notch		Double Notch		Fitting[4] Angle Cleats		Min. Support[5] Thickness		Max Notch[6] Length $c+t_1$		Tying[7]	
		Shear[2] Capacity kN	Critical[3] Design Check	Shear[2] Capacity kN	Critical[3] Design Check	Length ℓ mm	Mark	S275 mm	S355 mm	Single mm	Double mm	Capacity kN	Critical Check
686x254x170	8	964	4 (iii)	964	4 (iii)	570	CA8	6.5	5.5	414	150	877	11
{1600 kN}	7	823	4 (iii)	823	4 (iii)	500	CA7	6.4	5.3	485	148*	769	11
	6	683	4 (iii)	683	4 (iii)	430	CA6	6.2	5.2	584	145*	662	11
686x254x152	8	877	4 (iii)	877	4 (iii)	570	CA8	6.0	5.0	407	150	877	11
{1440 kN}	7	749	4 (iii)	749	4 (iii)	500	CA7	5.8	4.9	477	148	769	11
	6	621	4 (iii)	621	4 (iii)	430	CA6	5.6	4.7	575	145*	662	11
686x254x140	8	824	4 (iii)	824	4 (iii)	570	CA8	5.6	4.7	401	150	877	11
{1350 kN}	7	704	4 (iii)	704	4 (iii)	500	CA7	5.5	4.6	469	148	769	11
	6	584	4 (iii)	584	4 (iii)	430	CA6	5.3	4.4	566	145*	662	11
686x254x125	8	777	4 (iii)	777	4 (iii)	570	CA8	5.3	4.4	389	150	861	12 (ii)
{1260 kN}	7	664	4 (iii)	664	4 (iii)	500	CA7	5.2	4.3	456	148	753	12 (ii)
	6	551	4 (iii)	551	4 (iii)	430	CA6	5.0	4.2	550	145*	646	12 (ii)
610x305x238	7	1040	4 (iii)	1040	4 (iii)	500	CA7	8.1	6.8	416	148	769	11
{1860 kN}	6	866	4 (iii)	866	4 (iii)	430	CA6	7.8	6.6	502	145*	662	11
610x305x179	7	800	4 (iii)	800	4 (iii)	500	CA7	6.2	5.2	399	148	769	11
{1390 kN}	6	664	4 (iii)	664	4 (iii)	430	CA6	6.0	5.0	481	145*	662	11
610x305x149	7	670	4 (iii)	670	4 (iii)	500	CA7	5.2	4.3	391	148	760	12 (ii)
{1150 kN}	6	556	4 (iii)	556	4 (iii)	430	CA6	5.0	4.2	471	145*	651	12 (ii)
610x229x140	7	744	4 (iii)	744	4 (iii)	500	CA7	5.8	4.8	380	148	769	11
{1290 kN}	6	617	4 (iii)	617	4 (iii)	430	CA6	5.6	4.7	458	145*	662	11
610x229x125	7	676	4 (iii)	676	4 (iii)	500	CA7	5.2	4.4	373	148	766	12 (ii)
{1160 kN}	6	560	4 (iii)	560	4 (iii)	430	CA6	5.1	4.2	450	145*	657	12 (ii)
610x229x113	7	630	4 (iii)	630	4 (iii)	500	CA7	4.9	4.1	366	148	715	12 (ii)
{1070 kN}	6	523	4 (iii)	523	4 (iii)	430	CA6	4.7	4.0	441	145*	613	12 (ii)
610x229x101	7	596	4 (iii)	596	4 (iii)	500	CA7	4.6	3.9	369	169	676	12 (ii)
{1040 kN}	6	494	4 (iii)	494	4 (iii)	430	CA6	4.5	3.7	444	162*	580	12 (ii)

For guidance on the use of tables see Explanatory notes in Table H.6

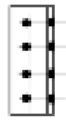
BEAM: S275
ANGLE CLEATS: S275
BOLTS: M20, 8.8

Table H.9 *Continued*



DOUBLE ANGLE CLEATS, ORDINARY or FLOWDRILL BOLTS

Single line of bolts
2 No. 90x90x10mm Equal Angles

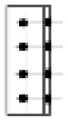


Beam Size	Bolt Rows	Un-notched or Single Notch		Double Notch		Fitting[4] Angle Cleats		Min. Support[5] Thickness		Max Notch[6] Length $c+t_1$		Tying[7]	
{ } [1]	n	Shear[2] Capacity kN	Critical[3] Design Check	Shear[2] Capacity kN	Critical[3] Design Check	Length ℓ mm	Mark	S275 mm	S355 mm	Single mm	Double mm	Capacity kN	Critical Check
533x210x122	6	598	4 (iii)	598	4 (iii)	430	CA6	5.4	4.5	349	145	662	11
{1100 kN}	5	475	4 (iii)	475	4 (iii)	360	CA5	5.2	4.3	439	142*	555	11
533x210x109	6	546	4 (iii)	546	4 (iii)	430	CA6	4.9	4.1	342	145	640	12 (ii)
{995 kN}	5	434	4 (iii)	434	4 (iii)	360	CA5	4.7	3.9	430	142*	534	12 (ii)
533x210x101	6	508	4 (iii)	508	4 (iii)	430	CA6	4.6	3.9	339	145	596	12 (ii)
{922 kN}	5	404	4 (iii)	404	4 (iii)	360	CA5	4.4	3.7	426	142*	497	12 (ii)
533x210x92	6	475	4 (iii)	475	4 (iii)	430	CA6	4.3	3.6	346	162	558	12 (ii)
{888 kN}	5	378	4 (iii)	378	4 (iii)	360	CA5	4.1	3.4	435	155*	465	12 (ii)
533x210x82	6	452	4 (iii)	452	4 (iii)	430	CA6	4.1	3.4	334	162	530	12 (ii)
{837 kN}	5	359	4 (iii)	359	4 (iii)	360	CA5	3.9	3.3	421	155*	442	12 (ii)
457x191x98	5	427	4 (iii)	427	4 (iii)	360	CA5	4.6	3.9	316	142	524	12 (ii)
{847 kN}	4	319	4 (iii)	319	4 (iii)	290	CA4	4.3	3.6	424	133*	420	12 (ii)
457x191x89	5	393	4 (iii)	393	4 (iii)	360	CA5	4.3	3.6	311	142	483	12 (ii)
{774 kN}	4	293	4 (iii)	293	4 (iii)	290	CA4	4.0	3.3	417	133*	386	12 (ii)
457x191x82	5	371	4 (iii)	371	4 (iii)	360	CA5	4.0	3.4	317	155	455	12 (ii)
{751 kN}	4	277	4 (iii)	277	4 (iii)	290	CA4	3.8	3.1	424	138*	364	12 (ii)
457x191x74	5	337	4 (iii)	337	4 (iii)	360	CA5	3.7	3.1	313	155	414	12 (ii)
{679 kN}	4	251	4 (iii)	251	4 (iii)	290	CA4	3.4	2.9	420	138*	331	12 (ii)
457x191x67	5	318	4 (iii)	318	4 (iii)	360	CA5	3.5	2.9	306	155	391	12 (ii)
{636 kN}	4	237	4 (iii)	237	4 (iii)	290	CA4	3.2	2.7	410	138*	313	12 (ii)

For guidance on the use of tables see Explanatory notes in Table H.6

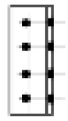
BEAM: S275
ANGLE CLEATS: S275
BOLTS: M20, 8.8

Table H.9 *Continued*



DOUBLE ANGLE CLEATS, ORDINARY or FLOWDRILL BOLTS

Single line of bolts
2 No. 90x90x10mm Equal Angles

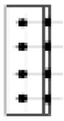


Beam Size { } [1]	Bolt Rows n	Un-notched or Single Notch: Shear[2] Capacity kN	Un-notched or Single Notch: Critical[3] Design Check	Double Notch: Shear[2] Capacity kN	Double Notch: Critical[3] Design Check	Fitting[4] Angle Cleats: Length ℓ mm	Fitting[4] Angle Cleats: Mark	Min. Support[5] Thickness: S275 mm	Min. Support[5] Thickness: S355 mm	Max Notch[6] Length $c+t_1$: Single mm	Max Notch[6] Length $c+t_1$: Double mm	Tying[7]: Capacity kN	Tying[7]: Critical Check
457x152x82	5	393	4 (iii)	393	4 (iii)	360	CA5	4.3	3.6	306	142	483	12 (ii)
{778 kN}	4	293	4 (iii)	293	4 (iii)	290	CA4	4.0	3.3	411	133*	386	12 (ii)
457x152x74	5	359	4 (iii)	359	4 (iii)	360	CA5	3.9	3.3	301	142	442	12 (ii)
{705 kN}	4	268	4 (iii)	268	4 (iii)	290	CA4	3.6	3.0	404	133*	353	12 (ii)
457x152x67	5	337	4 (iii)	337	4 (iii)	360	CA5	3.7	3.1	305	155	414	12 (ii)
{680 kN}	4	251	4 (iii)	251	4 (iii)	290	CA4	3.4	2.9	409	138*	331	12 (ii)
457x152x60	5	303	4 (iii)	303	4 (iii)	360	CA5	3.3	2.8	301	155	373	12 (ii)
{608 kN}	4	226	4 (iii)	226	4 (iii)	290	CA4	3.1	2.6	403	138*	298	12 (ii)
457x152x52	5	284	4 (iii)	284	4 (iii)	360	CA5	3.1	2.6	289	155	350	12 (ii)
{564 kN}	4	212	4 (iii)	212	4 (iii)	290	CA4	2.9	2.4	357	138*	280	12 (ii)
406x178x74 {647 kN}	4	265	4 (iii)	265	4 (iii)	290	CA4	3.6	3.0	336	138	350	12 (ii)
406x178x67 {594 kN}	4	246	4 (iii)	246	4 (iii)	290	CA4	3.3	2.8	329	138	324	12 (ii)
406x178x60 {530 kN}	4	221	4 (iii)	221	4 (iii)	290	CA4	3.0	2.5	325	138	291	12 (ii)
406x178x54 {512 kN}	4	215	4 (iii)	215	4 (iii)	290	CA4	2.9	2.4	314	138	283	12 (ii)
406x140x46 {452 kN}	4	190	4 (iii)	190	4 (iii)	290	CA4	2.6	2.2	311	138	250	12 (ii)
406x140x39 {420 kN}	4	179	4 (iii)	179	4 (iii)	290	CA4	2.4	2.0	275	138	236	12 (ii)
356x171x67 {546 kN}	3	171	4 (iii)	171	4 (iii)	220	CA3	3.1	2.6	373	118*	251	12 (ii)
356x171x57 {478 kN}	3	153	4 (iii)	153	4 (iii)	220	CA3	2.8	2.3	364	118*	224	12 (ii)
356x171x51 {433 kN}	3	139	4 (iii)	139	4 (iii)	220	CA3	2.5	2.1	357	118*	204	12 (ii)
356x171x45 {406 kN}	3	132	4 (iii)	132	4 (iii)	220	CA3	2.4	2.0	345	118*	193	12 (ii)
356x127x39 {385 kN}	3	124	4 (iii)	124	4 (iii)	220	CA3	2.3	1.9	340	118*	182	12 (ii)
356x127x33 {346 kN}	3	113	4 (iii)	113	4 (iii)	220	CA3	2.0	1.7	294	118*	166	12 (ii)

For guidance on the use of tables see Explanatory notes in Table H.6

BEAM: S275
ANGLE CLEATS: S275
BOLTS: M20, 8.8

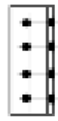
Table H.9 *Continued*



DOUBLE ANGLE CLEATS, ORDINARY or FLOWDRILL BOLTS

Single line of bolts

2 No. 90x90x10mm Equal Angles

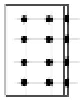


Beam Size { } (1)	Bolt Rows n	Un-notched or Single Notch		Double Notch		Fitting(4) Angle Cleats		Min. Support(5) Thickness		Max Notch(6) Length $c+t_1$		Tying(7)	
		Shear(2) Capacity kN	Critical(3) Design Check	Shear(2) Capacity kN	Critical(3) Design Check	Length ℓ mm	Mark	S275 mm	S355 mm	Single mm	Double mm	Capacity kN	Critical Check
305x165x54 {405 kN}	3	149	4 (iii)	149	4 (iii)	220	CA3	2.7	2.3	265	118	218	12 (ii)
305x165x46 {339 kN}	3	126	4 (iii)	126	4 (iii)	220	CA3	2.3	1.9	260	118	185	12 (ii)
305x165x40 {300 kN}	3	113	4 (iii)	113	4 (iii)	220	CA3	2.0	1.7	254	118	166	12 (ii)
305x127x48 {462 kN}	3	169	4 (iii)	169	4 (iii)	220	CA3	3.1	2.6	251	118	248	12 (ii)
305x127x42 {406 kN}	3	151	4 (iii)	151	4 (iii)	220	CA3	2.7	2.3	245	118	221	12 (ii)
305x127x37 {357 kN}	3	134	4 (iii)	134	4 (iii)	220	CA3	2.4	2.0	241	118	196	12 (ii)
305x102x33 {341 kN}	3	124	4 (iii)	124	4 (iii)	220	CA3	2.3	1.9	251	118	182	12 (ii)
305x102x28 {306 kN}	3	113	4 (iii)	113	4 (iii)	220	CA3	2.0	1.7	241	118	166	12 (ii)
305x102x25 {292 kN}	3	109	4 (iii)	109	4 (iii)	220	CA3	2.0	1.7	229	118	160	12 (ii)
254x146x43 {308 kN}	2	76	4 (iii)	76	4 (iii)	150	CA2	2.1	1.7	270	98*	132	12 (ii)
254x146x37 {266 kN}	2	66	4 (iii)	66	4 (iii)	150	CA2	1.8	1.5	266	98*	116	12 (ii)
254x146x31 {249 kN}	2	63	4 (iii)	63	4 (iii)	150	CA2	1.7	1.4	261	98*	110	12 (ii)
254x102x28 {271 kN}	2	66	4 (iii)	66	4 (iii)	150	CA2	1.8	1.5	270	98*	116	12 (ii)
254x102x25 {255 kN}	2	63	4 (iii)	63	4 (iii)	150	CA2	1.7	1.4	267	98*	110	12 (ii)
254x102x22 {239 kN}	2	60	4 (iii)	60	4 (iii)	150	CA2	1.6	1.4	264	98*	105	12 (ii)

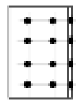
For guidance on the use of tables see Explanatory notes in Table H.6

BEAM: S275
ANGLE CLEATS: S275
BOLTS: M20, 8.8

Table H.10



DOUBLE ANGLE CLEATS, ORDINARY or FLOWDRILL BOLTS



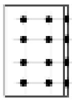
Double line of bolts
2 No.150x90x10mm Angles

Beam Size	Bolt Rows	Un-notched or Single Notch		Double Notch		Fitting[4] Angle Cleats		Min. Support[5] Thickness		Max Notch[6] Length $c+t_1$		Tying[7]	
Beam Size { } [1]	n	Shear[2] Capacity kN	Critical[3] Design Check	Shear[2] Capacity kN	Critical[3] Design Check	Length ℓ mm	Mark	S275 mm	S355 mm	Single mm	Double mm	Capacity kN	Critical Check
914x305x253	11	2020	8	1720	4(ii)	780	CB11	10.0	8.4	337	100	1200	11
{2530 kN}	10	1840	8	1550	4(ii)	710	CB10	10.0	8.4	445	100	1090	11
	9	1650	8	1390	4(ii)	640	CB9	10.0	8.4	516	100*	984	11
	8	1470	8	1230	4(ii)	570	CB8	10.0	8.4	580	100*	877	11
914x305x224	11	1950	4(ii)	1580	4(ii)	780	CB11	9.6	8.1	262	100	1200	11
{2300 kN}	10	1780	4(ii)	1430	4(ii)	710	CB10	9.7	8.1	372	100	1090	11
	9	1610	4(ii)	1280	4(ii)	640	CB9	9.7	8.1	476	100*	984	11
	8	1430	4(ii)	1130	4(ii)	570	CB8	9.7	8.2	533	100*	877	11
914x305x201	11	1850	4(ii)	1500	4(ii)	780	CB11	9.1	7.6	246	100	1200	11
{2170 kN}	10	1690	4(ii)	1360	4(ii)	710	CB10	9.2	7.7	354	100	1090	11
	9	1530	4(ii)	1210	4(ii)	640	CB9	9.2	7.7	461	100*	984	11
	8	1360	4(ii)	1070	4(ii)	570	CB8	9.3	7.7	517	100*	877	11
838x292x226	10	1790	4(ii)	1450	4(ii)	710	CB10	9.8	8.2	276	100	1090	11
{2180 kN}	9	1620	4(ii)	1290	4(ii)	640	CB9	9.8	8.2	386	100	984	11
	8	1450	4(ii)	1140	4(ii)	570	CB8	9.8	8.2	471	100*	877	11
838x292x194	10	1640	4(ii)	1320	4(ii)	710	CB10	8.9	7.4	256	100	1090	11
{1960 kN}	9	1480	4(ii)	1180	4(ii)	640	CB9	8.9	7.5	364	100	984	11
	8	1320	4(ii)	1040	4(ii)	570	CB8	9.0	7.5	454	100*	877	11
838x292x176	10	1560	4(ii)	1260	4(ii)	710	CB10	8.5	7.1	243	100	1090	11
{1860 kN}	9	1410	4(ii)	1130	4(ii)	640	CB9	8.5	7.1	351	100	984	11
	8	1260	4(ii)	992	4(ii)	570	CB8	8.6	7.2	442	100*	877	11
762x267x197	9	1560	4(ii)	1250	4(ii)	640	CB9	9.4	7.9	248	100	984	11
{1910 kN}	8	1400	4(ii)	1110	4(ii)	570	CB8	9.5	7.9	356	100	877	11
	7	1230	4(ii)	955	4(ii)	500	CB7	9.6	8.0	433	100*	769	11
762x267x173	9	1430	4(ii)	1150	4(ii)	640	CB9	8.6	7.2	234	100	984	11
{1730 kN}	8	1280	4(ii)	1010	4(ii)	570	CB8	8.7	7.3	341	100	877	11
	7	1130	4(ii)	875	4(ii)	500	CB7	8.8	7.3	421	100*	769	11
762x267x147	9	1280	4(ii)	1030	4(ii)	640	CB9	7.7	6.5	218	100	984	11
{1530 kN}	8	1140	4(ii)	907	4(ii)	570	CB8	7.8	6.5	325	100	877	11
	7	1010	4(ii)	784	4(ii)	500	CB7	7.8	6.5	408	100*	769	11
762x267x134	9	1240	4(ii)	1000	4(ii)	640	CB9	7.5	6.3	210	100	984	11
{1490 kN}	8	1110	4(ii)	882	4(ii)	570	CB8	7.6	6.3	316	100	877	11
	7	980	4(ii)	762	4(ii)	500	CB7	7.6	6.4	401	100*	769	11

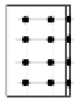
For guidance on the use of tables see Explanatory notes in Table H.6

BEAM: S275
ANGLE CLEATS: S275
BOLTS: M20, 8.8

Table H.10 *Continued*



DOUBLE ANGLE CLEATS, ORDINARY or FLOWDRILL BOLTS



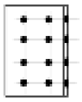
Double line of bolts
2 No.150x90x10mm Angles

Beam Size	Bolt Rows	Un-notched or Single Notch		Double Notch		Fitting[4] Angle Cleats		Min. Support[5] Thickness		Max Notch[6] Length $c+t_1$		Tying[7]	
Beam Size { }[1]	n	Shear[2] Capacity kN	Critical[3] Design Check	Shear[2] Capacity kN	Critical[3] Design Check	Length ℓ mm	Mark	S275 mm	S355 mm	Single mm	Double mm	Capacity kN	Critical Check
686x254x170	8	1290	4(ii)	1030	4(ii)	570	CB8	8.7	7.3	230	100	877	11
{1600 kN}	7	1140	4(ii)	888	4(ii)	500	CB7	8.8	7.4	338	100*	769	11
	6	982	4(ii)	747	4(ii)	430	CB6	8.9	7.4	406	100*	662	11
686x254x152	8	1170	4(ii)	935	4(ii)	570	CB8	8.0	6.7	221	100	877	11
{1440 kN}	7	1030	4(ii)	808	4(ii)	500	CB7	8.0	6.7	329	100	769	11
	6	894	4(ii)	680	4(ii)	430	CB6	8.1	6.8	399	100*	662	11
686x254x140	8	1100	4(ii)	878	4(ii)	570	CB8	7.5	6.2	214	100	877	11
{1350 kN}	7	971	4(ii)	759	4(ii)	500	CB7	7.5	6.3	321	100	769	11
	6	839	4(ii)	639	4(ii)	430	CB6	7.6	6.4	394	100*	662	11
686x254x125	8	1040	4(ii)	829	4(ii)	570	CB8	7.0	5.9	202	100	877	11
{1260 kN}	7	915	4(ii)	716	4(ii)	500	CB7	7.1	5.9	309	100	769	11
	6	791	4(ii)	603	4(ii)	430	CB6	7.2	6.0	383	100*	662	11
610x305x238	7	1290	8	1130	4(ii)	500	CB7	10.0	8.4	335	100	769	11
{1860 kN}	6	1100	8	948	4(ii)	430	CB6	10.0	8.4	395	100*	662	11
610x305x179	7	1100	4(ii)	863	4(ii)	500	CB7	8.5	7.1	225	100	769	11
{1390 kN}	6	949	4(ii)	726	4(ii)	430	CB6	8.6	7.2	336	100*	662	11
610x305x149	7	916	4(ii)	722	4(ii)	500	CB7	7.1	5.9	213	100	769	11
{1150 kN}	6	794	4(ii)	608	4(ii)	430	CB6	7.2	6.0	326	100*	662	11
610x229x140	7	1020	4(ii)	802	4(ii)	500	CB7	7.9	6.6	213	100	769	11
{1290 kN}	6	880	4(ii)	675	4(ii)	430	CB6	8.0	6.7	321	100*	662	11
610x229x125	7	921	4(ii)	729	4(ii)	500	CB7	7.2	6.0	205	100	769	11
{1160 kN}	6	799	4(ii)	613	4(ii)	430	CB6	7.2	6.1	313	100*	662	11
610x229x113	7	858	4(ii)	680	4(ii)	500	CB7	6.7	5.6	197	100	769	11
{1070 kN}	6	745	4(ii)	572	4(ii)	430	CB6	6.7	5.6	303	100*	662	11
610x229x101	7	841	4(ii)	667	4(ii)	500	CB7	6.5	5.5	187	100	769	11
{1040 kN}	6	730	4(ii)	561	4(ii)	430	CB6	6.6	5.5	292	100*	662	11

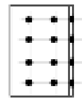
For guidance on the use of tables see Explanatory notes in Table H.6

BEAM: S275
ANGLE CLEATS: S275
BOLTS: M20, 8.8

Table H.10 *Continued*



DOUBLE ANGLE CLEATS, ORDINARY or FLOWDRILL BOLTS



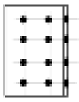
Double line of bolts
2 No.150x90x10mm Angles

Beam Size	Bolt Rows	Un-notched or Single Notch		Double Notch		Fitting[4] Angle Cleats		Min. Support[5] Thickness		Max Notch[6] Length $c+t_1$		Tying[7]	
	n	Shear[2] Capacity kN	Critical[3] Design Check	Shear[2] Capacity kN	Critical[3] Design Check	Length ℓ mm	Mark	S275 mm	S355 mm	Single mm	Double mm	Capacity kN	Critical Check
{ } [1]													
533x210x122	6	840	4(ii)	654	4(ii)	430	CB6	7.6	6.4	201	100	662	11
{1100 kN}	5	712	4(ii)	529	4(ii)	360	CB5	7.7	6.5	293	100*	555	11
533x210x109	6	766	4(ii)	597	4(ii)	430	CB6	6.9	5.8	193	100	662	11
{995 kN}	5	649	4(ii)	483	4(ii)	360	CB5	7.1	5.9	288	100*	555	11
533x210x101	6	713	4(ii)	556	4(ii)	430	CB6	6.5	5.4	189	100	662	11
{922 kN}	5	604	4(ii)	450	4(ii)	360	CB5	6.6	5.5	285	100*	555	11
533x210x92	6	691	4(ii)	540	4(ii)	430	CB6	6.3	5.2	183	100	662	11
{888 kN}	5	586	4(ii)	437	4(ii)	360	CB5	6.4	5.3	280	100*	555	11
533x210x82	6	655	4(ii)	513	4(ii)	430	CB6	5.9	5.0	174	100	662	11
{837 kN}	5	556	4(ii)	415	4(ii)	360	CB5	6.0	5.1	272	100*	555	11
457x191x98	5	624	4(ii)	475	4(ii)	360	CB5	6.8	5.7	183	100	555	11
{847 kN}	4	485	4 (iii)	361	4(ii)	290	CB4	6.6	5.5	278	100*	448	11
457x191x89	5	574	4(ii)	437	4(ii)	360	CB5	6.2	5.2	177	100	555	11
{774 kN}	4	447	4 (iii)	333	4(ii)	290	CB4	6.1	5.1	274	100*	448	11
457x191x82	5	561	4(ii)	428	4(ii)	360	CB5	6.1	5.1	172	100	555	11
{751 kN}	4	421	4 (iii)	325	4(ii)	290	CB4	5.7	4.8	279	100*	448	11
457x191x74	5	509	4(ii)	389	4(ii)	360	CB5	5.5	4.6	168	100	555	11
{679 kN}	4	383	4 (iii)	296	4(ii)	290	CB4	5.2	4.4	276	100*	448	11
457x191x67	5	480	4(ii)	367	4(ii)	360	CB5	5.2	4.4	162	100	555	11
{636 kN}	4	362	4 (iii)	279	4(ii)	290	CB4	4.9	4.1	269	100*	448	11

For guidance on the use of tables see Explanatory notes in Table H.6

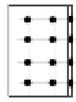
BEAM: S275
ANGLE CLEATS: S275
BOLTS: M20, 8.8

Table H.10 *Continued*



DOUBLE ANGLE CLEATS, ORDINARY or FLOWDRILL BOLTS

Double line of bolts
2 No.150x90x10mm Angles

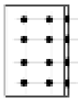


Beam Size	Bolt Rows	Un-notched or Single Notch		Double Notch		Fitting[4] Angle Cleats		Min. Support[5] Thickness		Max Notch[6] Length $c+t_1$		Tying[7]	
{ } [1]	n	Shear[2] Capacity kN	Critical[3] Design Check	Shear[2] Capacity kN	Critical[3] Design Check	Length ℓ mm	Mark	S275 mm	S355 mm	Single mm	Double mm	Capacity kN	Critical Check
457x152x82	5	573	4(ii)	437	4(ii)	360	CB5	6.2	5.2	178	100	555	11
{778 kN}	4	447	4 (iii)	333	4(ii)	290	CB4	6.1	5.1	270	100*	448	11
457x152x74	5	523	4(ii)	400	4(ii)	360	CB5	5.7	4.8	172	100	555	11
{705 kN}	4	408	4 (iii)	304	4(ii)	290	CB4	5.5	4.6	265	100*	448	11
457x152x67	5	508	4(ii)	389	4(ii)	360	CB5	5.5	4.6	166	100	555	11
{680 kN}	4	383	4 (iii)	296	4(ii)	290	CB4	5.2	4.4	269	100*	448	11
457x152x60	5	456	4(ii)	350	4(ii)	360	CB5	5.0	4.1	161	100	555	11
{608 kN}	4	345	4 (iii)	266	4(ii)	290	CB4	4.7	3.9	265	100*	448	11
457x152x52	5	426	4(ii)	329	4(ii)	360	CB5	4.6	3.9	153	100	555	11
{564 kN}	4	323	4 (iii)	250	4(ii)	290	CB4	4.4	3.7	255	100*	448	11
406x178x74	4	404	4 (iii)	312	4(ii)	290	CB4	5.5	4.6	221	100	448	11
{647 kN}													
406x178x67	4	374	4 (iii)	289	4(ii)	290	CB4	5.1	4.3	216	100	448	11
{594 kN}													
406x178x60	4	336	4 (iii)	260	4(ii)	290	CB4	4.6	3.8	214	100	448	11
{530 kN}													
406x178x54	4	328	4 (iii)	253	4(ii)	290	CB4	4.5	3.7	206	100	448	11
{512 kN}													
406x140x46	4	289	4 (iii)	224	4(ii)	290	CB4	3.9	3.3	204	100	415	12 (i)
{452 kN}													
406x140x39	4	272	4 (iii)	210	4(ii)	290	CB4	3.7	3.1	194	100	391	12 (i)
{420 kN}													
356x171x67	3	251	4 (iii)	202	4(ii)	220	CB3	4.5	3.8	258	100*	340	11
{546 kN}													
356x171x57	3	223	4 (iii)	180	4(ii)	220	CB3	4.0	3.4	249	100*	340	11
{478 kN}													
356x171x51	3	204	4 (iii)	164	4(ii)	220	CB3	3.7	3.1	244	100*	340	11
{433 kN}													
356x171x45	3	193	4 (iii)	155	4(ii)	220	CB3	3.5	2.9	236	100*	335	12 (i)
{406 kN}													
356x127x39	3	182	4 (iii)	146	4(ii)	220	CB3	3.3	2.8	232	100*	316	12 (i)
{385 kN}													
356x127x33	3	165	4 (iii)	133	4(ii)	220	CB3	3.0	2.5	223	100*	287	12 (i)
{346 kN}													

For guidance on the use of tables see Explanatory notes in Table H.6

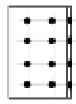
BEAM: S275
ANGLE CLEATS: S275
BOLTS: M20, 8.8

Table H.10 *Continued*



DOUBLE ANGLE CLEATS, ORDINARY or FLOWDRILL BOLTS

Double line of bolts
2 No.150x90x10mm Angles

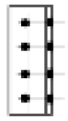


Beam Size { } (1)	Bolt Rows n	Un-notched or Single Notch		Double Notch		Fitting(4) Angle Cleats		Min. Support(5) Thickness		Max Notch(6) Length $c+t_1$		Tying(7)	
		Shear(2) Capacity kN	Critical(3) Design Check	Shear(2) Capacity kN	Critical(3) Design Check	Length ℓ mm	Mark	S275 mm	S355 mm	Single mm	Double mm	Capacity kN	Critical Check
305x165x54 {405 kN}	3	218	4 (iii)	175	4(ii)	220	CB3	3.9	3.3	181	100	340	11
305x165x46 {339 kN}	3	185	4 (iii)	149	4(ii)	220	CB3	3.3	2.8	178	100	321	12 (i)
305x165x40 {300 kN}	3	165	4 (iii)	133	4(ii)	220	CB3	3.0	2.5	173	100	287	12 (i)
305x127x48 {462 kN}	3	248	4 (iii)	200	4(ii)	220	CB3	4.5	3.8	172	100	340	11
305x127x42 {406 kN}	3	220	4 (iii)	177	4(ii)	220	CB3	4.0	3.3	167	100	340	11
305x127x37 {357 kN}	3	196	4 (iii)	158	4(ii)	220	CB3	3.5	3.0	164	100	340	12 (i)
305x102x33 {341 kN}	3	182	4 (iii)	146	4(ii)	220	CB3	3.3	2.8	172	100	316	12 (i)
305x102x28 {306 kN}	3	165	4 (iii)	133	4(ii)	220	CB3	3.0	2.5	165	100	287	12 (i)
305x102x25 {292 kN}	3	160	4 (iii)	129	4(ii)	220	CB3	2.9	2.4	157	100	278	12 (i)
254x146x43 {308 kN}	2	108	4 (iii)	74	4(ii)	150	CB2	2.9	2.4	218	100*	233	11
254x146x37 {266 kN}	2	94	4 (iii)	65	4(ii)	150	CB2	2.6	2.1	212	100*	218	12 (i)
254x146x31 {249 kN}	2	90	4 (iii)	62	4(ii)	150	CB2	2.4	2.0	200	100*	208	12 (i)
254x102x28 {271 kN}	2	94	4 (iii)	65	4(ii)	150	CB2	2.6	2.1	208	100*	218	12 (i)
254x102x25 {255 kN}	2	90	4 (iii)	62	4(ii)	150	CB2	2.4	2.0	200	100*	208	12 (i)
254x102x22 {239 kN}	2	85	4 (iii)	59	4(ii)	150	CB2	2.3	1.9	190	100*	198	12 (i)

For guidance on the use of tables see Explanatory notes in Table H.6

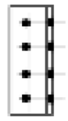
BEAM: S355
ANGLE CLEATS: S275
BOLTS: M20, 8.8

Table H.11



DOUBLE ANGLE CLEATS, ORDINARY or FLOWDRILL BOLTS

Single line of bolts
2 No. 90x90x10mm Equal Angles

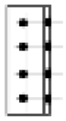


Beam Size	Bolt Rows	Un-notched or Single Notch		Double Notch		Fitting[4] Angle Cleats		Min. Support[5] Thickness		Max Notch[6] Length $c+t_1$		Tying[7]	
{ } [1]	n	Shear[2] Capacity kN	Critical[3] Design Check	Shear[2] Capacity kN	Critical[3] Design Check	Length ℓ mm	Mark	S275 mm	S355 mm	Single mm	Double mm	Capacity kN	Critical Check
914x305x253	11	1900	2	1900	2	780	CA11	9.4	7.9	584	207	1200	11
{3290 kN}	10	1710	2	1710	2	710	CA10	9.3	7.8	649	201	1090	11
	9	1520	2	1520	2	640	CA9	9.2	7.7	685	195*	984	11
	8	1330	2	1330	2	570	CA8	9.0	7.5	685	188*	877	11
914x305x224	11	1810	4 (iii)	1810	4 (iii)	780	CA11	9.0	7.5	543	166	1200	11
{3000 kN}	10	1630	4 (iii)	1630	4 (iii)	710	CA10	8.9	7.4	543	165	1090	11
	9	1450	4 (iii)	1450	4 (iii)	640	CA9	8.7	7.3	543	164*	984	11
	8	1260	4 (iii)	1260	4 (iii)	570	CA8	8.6	7.2	543	163*	877	11
914x305x201	11	1720	4 (iii)	1720	4 (iii)	780	CA11	8.5	7.1	474	166	1200	11
{2820 kN}	10	1550	4 (iii)	1550	4 (iii)	710	CA10	8.4	7.0	474	165	1090	11
	9	1370	4 (iii)	1370	4 (iii)	640	CA9	8.3	6.9	474	164*	984	11
	8	1200	4 (iii)	1200	4 (iii)	570	CA8	8.2	6.8	474	163*	877	11
838x292x226	10	1650	4 (iii)	1650	4 (iii)	710	CA10	9.0	7.5	539	165	1090	11
{2840 kN}	9	1470	4 (iii)	1470	4 (iii)	640	CA9	8.8	7.4	607	164	984	11
	8	1280	4 (iii)	1280	4 (iii)	570	CA8	8.7	7.3	644	163*	877	11
838x292x194	10	1510	4 (iii)	1510	4 (iii)	710	CA10	8.2	6.8	504	165	1090	11
{2560 kN}	9	1340	4 (iii)	1340	4 (iii)	640	CA9	8.1	6.8	504	164	984	11
	8	1170	4 (iii)	1170	4 (iii)	570	CA8	7.9	6.6	504	163*	877	11
838x292x176	10	1430	4 (iii)	1430	4 (iii)	710	CA10	7.8	6.5	443	165	1090	11
{2420 kN}	9	1270	4 (iii)	1270	4 (iii)	640	CA9	7.7	6.4	443	164	984	11
	8	1110	4 (iii)	1110	4 (iii)	570	CA8	7.6	6.3	443	163*	877	11
762x267x197	9	1420	4 (iii)	1420	4 (iii)	640	CA9	8.6	7.2	489	164	984	11
{2490 kN}	8	1240	4 (iii)	1240	4 (iii)	570	CA8	8.4	7.0	560	163	877	11
	7	1060	4 (iii)	1060	4 (iii)	500	CA7	8.2	6.9	655	160*	769	11
762x267x173	9	1300	4 (iii)	1300	4 (iii)	640	CA9	7.9	6.6	475	164	984	11
{2260 kN}	8	1140	4 (iii)	1140	4 (iii)	570	CA8	7.7	6.5	544	163	877	11
	7	971	4 (iii)	971	4 (iii)	500	CA7	7.5	6.3	564	160*	769	11
762x267x147	9	1160	4 (iii)	1160	4 (iii)	640	CA9	7.0	5.9	416	164	984	11
{2000 kN}	8	1020	4 (iii)	1020	4 (iii)	570	CA8	6.9	5.8	416	163	877	11
	7	869	4 (iii)	869	4 (iii)	500	CA7	6.7	5.6	416	160*	769	11
762x267x134	9	1090	4 (iii)	1090	4 (iii)	640	CA9	6.6	5.5	348	189	984	11
{1920 kN}	8	953	4 (iii)	953	4 (iii)	570	CA8	6.5	5.4	348	184	877	11
	7	814	4 (iii)	814	4 (iii)	500	CA7	6.3	5.3	348	178*	769	11

For guidance on the use of tables see Explanatory notes in Table H.6

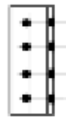
BEAM: S355
ANGLE CLEATS: S275
BOLTS: M20, 8.8

Table H.11 *Continued*



DOUBLE ANGLE CLEATS, ORDINARY or FLOWDRILL BOLTS

Single line of bolts
2 No. 90x90x10mm Equal Angles

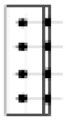


Beam Size	Bolt Rows	Un-notched or Single Notch		Double Notch		Fitting[4] Angle Cleats		Min. Support[5] Thickness		Max Notch[6] Length $c+t_1$		Tying[7]	
{ } [1]	n	Shear[2] Capacity kN	Critical[3] Design Check	Shear[2] Capacity kN	Critical[3] Design Check	Length ℓ mm	Mark	S275 mm	S355 mm	Single mm	Double mm	Capacity kN	Critical Check
686x254x170	8	1150	4 (iii)	1150	4 (iii)	570	CA8	7.8	6.5	451	163	877	11
{2080 kN}	7	984	4 (iii)	984	4 (iii)	500	CA7	7.6	6.4	528	160*	769	11
	6	816	4 (iii)	816	4 (iii)	430	CA6	7.4	6.2	636	158*	662	11
686x254x152	8	1050	4 (iii)	1050	4 (iii)	570	CA8	7.1	6.0	443	163	877	11
{1880 kN}	7	896	4 (iii)	896	4 (iii)	500	CA7	7.0	5.8	519	160	769	11
	6	743	4 (iii)	743	4 (iii)	430	CA6	6.7	5.6	545	158*	662	11
686x254x140	8	985	4 (iii)	985	4 (iii)	570	CA8	6.7	5.6	437	163	877	11
{1750 kN}	7	842	4 (iii)	842	4 (iii)	500	CA7	6.5	5.5	459	160	769	11
	6	698	4 (iii)	698	4 (iii)	430	CA6	6.3	5.3	459	158*	662	11
686x254x125	8	930	4 (iii)	930	4 (iii)	570	CA8	6.3	5.3	393	163	877	11
{1640 kN}	7	794	4 (iii)	794	4 (iii)	500	CA7	6.2	5.2	393	160	769	11
	6	659	4 (iii)	659	4 (iii)	430	CA6	6.0	5.0	393	158*	662	11
610x305x238	7	1130	2	1130	2	500	CA7	8.8	7.4	500	217	769	11
{2420 kN}	6	940	2	940	2	430	CA6	8.5	7.1	602	205*	662	11
610x305x179	7	957	4 (iii)	957	4 (iii)	500	CA7	7.4	6.2	434	160	769	11
{1810 kN}	6	794	4 (iii)	794	4 (iii)	430	CA6	7.2	6.0	524	158*	662	11
610x305x149	7	801	4 (iii)	801	4 (iii)	500	CA7	6.2	5.2	425	160	769	11
{1500 kN}	6	664	4 (iii)	664	4 (iii)	430	CA6	6.0	5.0	492	158*	662	11
610x229x140	7	889	4 (iii)	889	4 (iii)	500	CA7	6.9	5.8	413	160	769	11
{1670 kN}	6	737	4 (iii)	737	4 (iii)	430	CA6	6.7	5.6	498	158*	662	11
610x229x125	7	808	4 (iii)	808	4 (iii)	500	CA7	6.3	5.2	406	160	769	11
{1510 kN}	6	670	4 (iii)	670	4 (iii)	430	CA6	6.1	5.1	490	158*	662	11
610x229x113	7	753	4 (iii)	753	4 (iii)	500	CA7	5.8	4.9	398	160	769	11
{1400 kN}	6	625	4 (iii)	625	4 (iii)	430	CA6	5.7	4.7	417	158*	662	11
610x229x101	7	713	4 (iii)	713	4 (iii)	500	CA7	5.5	4.6	361	178	769	11
{1350 kN}	6	591	4 (iii)	591	4 (iii)	430	CA6	5.4	4.5	361	172*	662	11

For guidance on the use of tables see Explanatory notes in Table H.6

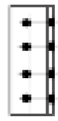
BEAM: S355
ANGLE CLEATS: S275
BOLTS: M20, 8.8

Table H.11 *Continued*



DOUBLE ANGLE CLEATS, ORDINARY or FLOWDRILL BOLTS

Single line of bolts
2 No. 90x90x10mm Equal Angles

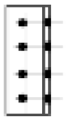


Beam Size { } (1)	Bolt Rows n	Un-notched or Single Notch		Double Notch		Fitting(4) Angle Cleats		Min. Support(5) Thickness		Max Notch(6) Length $c+t_1$		Tying(7)	
		Shear(2) Capacity kN	Critical(3) Design Check	Shear(2) Capacity kN	Critical(3) Design Check	Length ℓ mm	Mark	S275 mm	S355 mm	Single mm	Double mm	Capacity kN	Critical Check
533x210x122	6	715	4 (iii)	715	4 (iii)	430	CA6	6.5	5.4	380	158	662	11
{1430 kN}	5	568	4 (iii)	568	4 (iii)	360	CA5	6.2	5.2	478	154*	555	11
533x210x109	6	653	4 (iii)	653	4 (iii)	430	CA6	5.9	4.9	372	158	662	11
{1300 kN}	5	519	4 (iii)	519	4 (iii)	360	CA5	5.6	4.7	468	154*	555	11
533x210x101	6	608	4 (iii)	608	4 (iii)	430	CA6	5.5	4.6	369	158	662	11
{1200 kN}	5	483	4 (iii)	483	4 (iii)	360	CA5	5.3	4.4	464	154*	555	11
533x210x92	6	569	4 (iii)	569	4 (iii)	430	CA6	5.1	4.3	373	172	662	11
{1150 kN}	5	452	4 (iii)	452	4 (iii)	360	CA5	4.9	4.1	409	165*	555	11
533x210x82	6	540	4 (iii)	540	4 (iii)	430	CA6	4.9	4.1	359	172	634	12 (ii)
{1080 kN}	5	430	4 (iii)	430	4 (iii)	360	CA5	4.7	3.9	359	165*	528	12 (ii)
457x191x98	5	510	4 (iii)	510	4 (iii)	360	CA5	5.5	4.6	344	154	555	11
{1100 kN}	4	381	4 (iii)	381	4 (iii)	290	CA4	5.2	4.3	461	145*	448	11
457x191x89	5	470	4 (iii)	470	4 (iii)	360	CA5	5.1	4.3	339	154	555	11
{1010 kN}	4	351	4 (iii)	351	4 (iii)	290	CA4	4.8	4.0	454	145*	448	11
457x191x82	5	443	4 (iii)	443	4 (iii)	360	CA5	4.8	4.0	342	165	545	12 (ii)
{970 kN}	4	331	4 (iii)	331	4 (iii)	290	CA4	4.5	3.8	458	149*	436	12 (ii)
457x191x74	5	403	4 (iii)	403	4 (iii)	360	CA5	4.4	3.7	338	165	495	12 (ii)
{876 kN}	4	301	4 (iii)	301	4 (iii)	290	CA4	4.1	3.4	394	149*	396	12 (ii)
457x191x67	5	380	4 (iii)	380	4 (iii)	360	CA5	4.1	3.5	330	165	468	12 (ii)
{821 kN}	4	284	4 (iii)	284	4 (iii)	290	CA4	3.9	3.2	339	149*	374	12 (ii)

For guidance on the use of tables see Explanatory notes in Table H.6

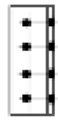
BEAM: S355
ANGLE CLEATS: S275
BOLTS: M20, 8.8

Table H.11 *Continued*



DOUBLE ANGLE CLEATS, ORDINARY or FLOWDRILL BOLTS

Single line of bolts
2 No. 90x90x10mm Equal Angles

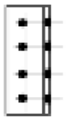


Beam Size { } [1]	Bolt Rows n	Un-notched or Single Notch		Double Notch		Fitting[4] Angle Cleats		Min. Support[5] Thickness		Max Notch[6] Length $c+t_1$		Tying[7]	
		Shear[2] Capacity kN	Critical[3] Design Check	Shear[2] Capacity kN	Critical[3] Design Check	Length ℓ mm	Mark	S275 mm	S355 mm	Single mm	Double mm	Capacity kN	Critical Check
457x152x82	5	470	4 (iii)	470	4 (iii)	360	CA5	5.1	4.3	334	154	555	11
{1010 kN}	4	351	4 (iii)	351	4 (iii)	290	CA4	4.8	4.0	447	145*	448	11
457x152x74	5	430	4 (iii)	430	4 (iii)	360	CA5	4.7	3.9	328	154	528	12 (ii)
{918 kN}	4	321	4 (iii)	321	4 (iii)	290	CA4	4.4	3.6	440	145*	422	12 (ii)
457x152x67	5	403	4 (iii)	403	4 (iii)	360	CA5	4.4	3.7	330	165	495	12 (ii)
{878 kN}	4	301	4 (iii)	301	4 (iii)	290	CA4	4.1	3.4	392	149*	396	12 (ii)
457x152x60	5	363	4 (iii)	363	4 (iii)	360	CA5	3.9	3.3	293	165	446	12 (ii)
{784 kN}	4	271	4 (iii)	271	4 (iii)	290	CA4	3.7	3.1	293	149*	356	12 (ii)
457x152x52	5	340	4 (iii)	340	4 (iii)	360	CA5	3.7	3.1	249	165	418	12 (ii)
{728 kN}	4	254	4 (iii)	254	4 (iii)	290	CA4	3.4	2.9	249	149*	334	12 (ii)
406x178x74	4	317	4 (iii)	317	4 (iii)	290	CA4	4.3	3.6	362	149	418	12 (ii)
{835 kN}													
406x178x67	4	294	4 (iii)	294	4 (iii)	290	CA4	4.0	3.3	356	149	387	12 (ii)
{767 kN}													
406x178x60	4	264	4 (iii)	264	4 (iii)	290	CA4	3.6	3.0	338	149	348	12 (ii)
{684 kN}													
406x178x54	4	257	4 (iii)	257	4 (iii)	290	CA4	3.5	2.9	320	149	339	12 (ii)
{660 kN}													
406x140x46	4	227	4 (iii)	227	4 (iii)	290	CA4	3.1	2.6	223	149	299	12 (ii)
{584 kN}													
406x140x39	4	214	4 (iii)	214	4 (iii)	290	CA4	2.9	2.4	192	149	282	12 (ii)
{543 kN}													
356x171x67	3	205	4 (iii)	205	4 (iii)	220	CA3	3.7	3.1	373	127*	300	12 (ii)
{704 kN}													
356x171x57	3	182	4 (iii)	182	4 (iii)	220	CA3	3.3	2.8	368	127*	267	12 (ii)
{618 kN}													
356x171x51	3	167	4 (iii)	167	4 (iii)	220	CA3	3.0	2.5	365	127*	244	12 (ii)
{560 kN}													
356x171x45	3	158	4 (iii)	158	4 (iii)	220	CA3	2.9	2.4	316	127*	231	12 (ii)
{524 kN}													
356x127x39	3	149	4 (iii)	149	4 (iii)	220	CA3	2.7	2.3	263	127*	218	12 (ii)
{497 kN}													
356x127x33	3	135	4 (iii)	135	4 (iii)	220	CA3	2.4	2.0	205	127*	198	12 (ii)
{446 kN}													

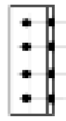
For guidance on the use of tables see Explanatory notes in Table H.6

BEAM: S355
ANGLE CLEATS: S275
BOLTS: M20, 8.8

Table H.11 *Continued*



DOUBLE ANGLE CLEATS, ORDINARY or FLOWDRILL BOLTS



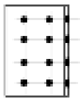
Single line of bolts
2 No. 90x90x10mm Equal Angles

	Bolt	Un-notched or Single Notch		Double Notch		Fitting[4] Angle Cleats		Min. Support[5] Thickness		Max Notch[6] Length $c+t_1$		Tying[7]	
Beam Size { } [1]	Rows n	Shear[2] Capacity kN	Critical[3] Design Check	Shear[2] Capacity kN	Critical[3] Design Check	Length ℓ mm	Mark	S275 mm	S355 mm	Single mm	Double mm	Capacity kN	Critical Check
305x165x54 {522 kN}	3	178	4 (iii)	178	4 (iii)	220	CA3	3.2	2.7	286	127	261	12 (ii)
305x165x46 {438 kN}	3	151	4 (iii)	151	4 (iii)	220	CA3	2.7	2.3	280	127	221	12 (ii)
305x165x40 {388 kN}	3	135	4 (iii)	135	4 (iii)	220	CA3	2.4	2.0	268	127	198	12 (ii)
305x127x48 {596 kN}	3	203	4 (iii)	203	4 (iii)	220	CA3	3.7	3.1	271	127	297	12 (ii)
305x127x42 {523 kN}	3	180	4 (iii)	180	4 (iii)	220	CA3	3.3	2.7	264	127	264	12 (ii)
305x127x37 {460 kN}	3	160	4 (iii)	160	4 (iii)	220	CA3	2.9	2.4	260	127	234	12 (ii)
305x102x33 {440 kN}	3	149	4 (iii)	149	4 (iii)	220	CA3	2.7	2.3	271	127	218	12 (ii)
305x102x28 {395 kN}	3	135	4 (iii)	135	4 (iii)	220	CA3	2.4	2.0	259	127	198	12 (ii)
305x102x25 {377 kN}	3	131	4 (iii)	131	4 (iii)	220	CA3	2.4	2.0	241	127	191	12 (ii)
254x146x43 {398 kN}	2	91	4 (iii)	91	4 (iii)	150	CA2	2.5	2.1	270	106*	158	12 (ii)
254x146x37 {344 kN}	2	79	4 (iii)	79	4 (iii)	150	CA2	2.2	1.8	266	106*	139	12 (ii)
254x146x31 {321 kN}	2	76	4 (iii)	76	4 (iii)	150	CA2	2.1	1.7	261	106*	132	12 (ii)
254x102x28 {349 kN}	2	79	4 (iii)	79	4 (iii)	150	CA2	2.2	1.8	270	106*	139	12 (ii)
254x102x25 {329 kN}	2	76	4 (iii)	76	4 (iii)	150	CA2	2.1	1.7	267	106*	132	12 (ii)
254x102x22 {308 kN}	2	72	4 (iii)	72	4 (iii)	150	CA2	2.0	1.6	264	106*	125	12 (ii)

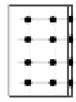
For guidance on the use of tables see Explanatory notes in Table H.6

BEAM: S355
ANGLE CLEATS: S275
BOLTS: M20, 8.8

Table H.12



DOUBLE ANGLE CLEATS, ORDINARY or FLOWDRILL BOLTS



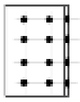
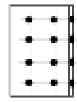
Double line of bolts
2 No.150x90x10mm Angles

Beam Size { } [(1)]	Bolt Rows n	Un-notched or Single Notch: Shear[(2)] Capacity kN	Un-notched or Single Notch: Critical[(3)] Design Check	Double Notch: Shear[(2)] Capacity kN	Double Notch: Critical[(3)] Design Check	Fitting[(4)] Angle Cleats: Length ℓ mm	Fitting[(4)] Angle Cleats: Mark	Min. Support[(5)] Thickness: S275 mm	Min. Support[(5)] Thickness: S355 mm	Max Notch[(6)] Length $c+t_1$: Single mm	Max Notch[(6)] Length $c+t_1$: Double mm	Tying[(7)]: Capacity kN	Tying[(7)]: Critical Check
914x305x253	11	2020	8	2020	8	780	CB11	10.0	8.4	550	132	1200	11
{3290 kN}	10	1840	8	1840	8	710	CB10	10.0	8.4	604	122	1090	11
	9	1650	8	1650	8	640	CB9	10.0	8.4	672	113*	984	11
	8	1470	8	1470	8	570	CB8	10.0	8.4	685	104*	877	11
914x305x224	11	2020	8	1890	4(ii)	780	CB11	10.0	8.4	468	100	1200	11
{3000 kN}	10	1840	8	1710	4(ii)	710	CB10	10.0	8.4	542	100	1090	11
	9	1650	8	1540	4(ii)	640	CB9	10.0	8.4	543	100*	984	11
	8	1470	8	1360	4(ii)	570	CB8	10.0	8.4	543	100*	877	11
914x305x201	11	2020	8	1800	4(ii)	780	CB11	10.0	8.4	384	100	1200	11
{2820 kN}	10	1840	8	1630	4(ii)	710	CB10	10.0	8.4	474	100	1090	11
	9	1650	8	1460	4(ii)	640	CB9	10.0	8.4	474	100*	984	11
	8	1470	8	1290	4(ii)	570	CB8	10.0	8.4	474	100*	877	11
838x292x226	10	1840	8	1740	4(ii)	710	CB10	10.0	8.4	484	100	1090	11
{2840 kN}	9	1650	8	1560	4(ii)	640	CB9	10.0	8.4	537	100	984	11
	8	1470	8	1370	4(ii)	570	CB8	10.0	8.4	605	100*	877	11
838x292x194	10	1840	8	1590	4(ii)	710	CB10	10.0	8.4	361	100	1090	11
{2560 kN}	9	1650	8	1420	4(ii)	640	CB9	10.0	8.4	472	100	984	11
	8	1470	8	1250	4(ii)	570	CB8	10.0	8.4	504	100*	877	11
838x292x176	10	1840	8	1510	4(ii)	710	CB10	10.0	8.4	287	100	1090	11
{2420 kN}	9	1650	8	1350	4(ii)	640	CB9	10.0	8.4	429	100	984	11
	8	1470	8	1190	4(ii)	570	CB8	10.0	8.4	443	100*	877	11
762x267x197	9	1650	8	1510	4(ii)	640	CB9	10.0	8.4	405	100	984	11
{2490 kN}	8	1470	8	1330	4(ii)	570	CB8	10.0	8.4	472	100	877	11
	7	1290	8	1150	4(ii)	500	CB7	10.0	8.4	539	100*	769	11
762x267x173	9	1650	8	1380	4(ii)	640	CB9	10.0	8.4	299	100	984	11
{2260 kN}	8	1470	8	1220	4(ii)	570	CB8	10.0	8.4	421	100	877	11
	7	1290	8	1060	4(ii)	500	CB7	10.0	8.4	481	100*	769	11
762x267x147	9	1650	8	1240	4(ii)	640	CB9	10.0	8.4	134	100	984	11
{2000 kN}	8	1470	8	1090	4(ii)	570	CB8	10.0	8.4	314	100	877	11
	7	1290	8	946	4(ii)	500	CB7	10.0	8.4	416	100*	769	11
762x267x134	9	1600	4(ii)	1190	4(ii)	640	CB9	9.6	8.1	118	100	984	11
{1920 kN}	8	1430	4(ii)	1050	4(ii)	570	CB8	9.7	8.1	297	100	877	11
	7	1260	4(ii)	912	4(ii)	500	CB7	9.8	8.2	348	100*	769	11

For guidance on the use of tables see Explanatory notes in Table H.6

BEAM: S355
ANGLE CLEATS: S275
BOLTS: M20, 8.8

Table H.12 *Continued*

DOUBLE ANGLE CLEATS, ORDINARY or FLOWDRILL BOLTS

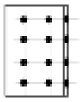
Double line of bolts
2 No.150x90x10mm Angles

Beam Size { } [1]	Bolt Rows n	Un-notched or Single Notch		Double Notch		Fitting[4] Angle Cleats		Min. Support[5] Thickness		Max Notch[6] Length $c+t_1$		Tying[7]	
		Shear[2] Capacity kN	Critical[3] Design Check	Shear[2] Capacity kN	Critical[3] Design Check	Length ℓ mm	Mark	S275 mm	S355 mm	Single mm	Double mm	Capacity kN	Critical Check
686x254x170	8	1470	8	1240	4(ii)	570	CB8	10.0	8.4	305	100	877	11
{2080 kN}	7	1290	8	1070	4(ii)	500	CB7	10.0	8.4	404	100*	769	11
	6	1100	8	904	4(ii)	430	CB6	10.0	8.4	471	100*	662	11
686x254x152	8	1470	8	1130	4(ii)	570	CB8	10.0	8.4	198	100	877	11
{1880 kN}	7	1290	8	975	4(ii)	500	CB7	10.0	8.4	353	100	769	11
	6	1100	8	823	4(ii)	430	CB6	10.0	8.4	422	100*	662	11
686x254x140	8	1420	4(ii)	1060	4(ii)	570	CB8	9.7	8.1	149	100	877	11
{1750 kN}	7	1260	4(ii)	916	4(ii)	500	CB7	9.7	8.2	312	100	769	11
	6	1080	4(ii)	773	4(ii)	430	CB6	9.8	8.2	397	100*	662	11
686x254x125	8	1340	4(ii)	998	4(ii)	570	CB8	9.1	7.6	131	100	877	11
{1640 kN}	7	1180	4(ii)	864	4(ii)	500	CB7	9.2	7.7	298	100	769	11
	6	1020	4(ii)	729	4(ii)	430	CB6	9.3	7.7	386	100*	662	11
610x305x238	7	1290	8	1290	8	500	CB7	10.0	8.4	440	141	769	11
{2420 kN}	6	1100	8	1100	8	430	CB6	10.0	8.4	514	124*	662	11
610x305x179	7	1290	8	1040	4(ii)	500	CB7	10.0	8.4	274	100	769	11
{1810 kN}	6	1100	8	879	4(ii)	430	CB6	10.0	8.4	377	100*	662	11
610x305x149	7	1180	4(ii)	872	4(ii)	500	CB7	9.2	7.7	164	100	769	11
{1500 kN}	6	1030	4(ii)	736	4(ii)	430	CB6	9.3	7.8	324	100*	662	11
610x229x140	7	1290	8	968	4(ii)	500	CB7	10.0	8.4	192	100	769	11
{1670 kN}	6	1100	8	817	4(ii)	430	CB6	10.0	8.4	333	100*	662	11
610x229x125	7	1190	4(ii)	879	4(ii)	500	CB7	9.3	7.7	159	100	769	11
{1510 kN}	6	1030	4(ii)	742	4(ii)	430	CB6	9.3	7.8	311	100*	662	11
610x229x113	7	1110	4(ii)	820	4(ii)	500	CB7	8.6	7.2	146	100	769	11
{1400 kN}	6	962	4(ii)	692	4(ii)	430	CB6	8.7	7.3	300	100*	662	11
610x229x101	7	1080	4(ii)	798	4(ii)	500	CB7	8.4	7.0	132	100	769	11
{1350 kN}	6	935	4(ii)	674	4(ii)	430	CB6	8.5	7.1	288	100*	662	11

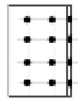
For guidance on the use of tables see Explanatory notes in Table H.6

BEAM: S355
ANGLE CLEATS: S275
BOLTS: M20, 8.8

Table H.12 *Continued*



DOUBLE ANGLE CLEATS, ORDINARY or FLOWDRILL BOLTS



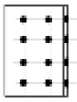
Double line of bolts
2 No.150x90x10mm Angles

Beam Size	Bolt Rows	Un-notched or Single Notch		Double Notch		Fitting[4] Angle Cleats		Min. Support[5] Thickness		Max Notch[6] Length $c+t_1$		Tying[7]	
{ }[1]	n	Shear[2] Capacity kN	Critical[3] Design Check	Shear[2] Capacity kN	Critical[3] Design Check	Length ℓ mm	Mark	S275 mm	S355 mm	Single mm	Double mm	Capacity kN	Critical Check
533x210x122	6	1090	4(ii)	792	4(ii)	430	CB6	9.8	8.2	173	100	662	11
{1430 kN}	5	901	4 (iii)	644	4(ii)	360	CB5	9.8	8.2	302	100*	555	11
533x210x109	6	990	4(ii)	723	4(ii)	430	CB6	9.0	7.5	161	100	662	11
{1300 kN}	5	823	4 (iii)	588	4(ii)	360	CB5	8.9	7.5	296	100*	555	11
533x210x101	6	921	4(ii)	673	4(ii)	430	CB6	8.3	7.0	155	100	662	11
{1200 kN}	5	766	4 (iii)	547	4(ii)	360	CB5	8.3	7.0	293	100*	555	11
533x210x92	6	885	4(ii)	648	4(ii)	430	CB6	8.0	6.7	146	100	662	11
{1150 kN}	5	716	4 (iii)	527	4(ii)	360	CB5	7.8	6.5	296	100*	555	11
533x210x82	6	839	4(ii)	616	4(ii)	430	CB6	7.6	6.4	134	100	662	11
{1080 kN}	5	681	4 (iii)	501	4(ii)	360	CB5	7.4	6.2	287	100*	555	11
457x191x98	5	805	4(ii)	578	4(ii)	360	CB5	8.8	7.3	164	100	555	11
{1100 kN}	4	580	4 (iii)	443	4(ii)	290	CB4	7.9	6.6	303	100*	448	11
457x191x89	5	741	4(ii)	532	4(ii)	360	CB5	8.1	6.7	156	100	555	11
{1010 kN}	4	534	4 (iii)	408	4(ii)	290	CB4	7.3	6.1	298	100*	448	11
457x191x82	5	702	4 (iii)	516	4(ii)	360	CB5	7.6	6.4	163	100	555	11
{970 kN}	4	504	4 (iii)	395	4(ii)	290	CB4	6.8	5.7	301	100*	448	11
457x191x74	5	638	4 (iii)	469	4(ii)	360	CB5	6.9	5.8	156	100	555	11
{876 kN}	4	458	4 (iii)	360	4(ii)	290	CB4	6.2	5.2	298	100*	448	11
457x191x67	5	603	4 (iii)	443	4(ii)	360	CB5	6.6	5.5	146	100	555	11
{821 kN}	4	432	4 (iii)	340	4(ii)	290	CB4	5.9	4.9	290	100*	448	11

For guidance on the use of tables see Explanatory notes in Table H.6

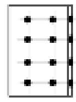
BEAM: S355
ANGLE CLEATS: S275
BOLTS: M20, 8.8

Table H.12 *Continued*



DOUBLE ANGLE CLEATS, ORDINARY or FLOWDRILL BOLTS

Double line of bolts
2 No.150x90x10mm Angles

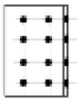


Beam Size { } (1)	Bolt Rows n	Un-notched or Single Notch		Double Notch		Fitting(4) Angle Cleats		Min. Support(5) Thickness		Max Notch(6) Length $c+t_1$		Tying(7)	
		Shear(2) Capacity kN	Critical(3) Design Check	Shear(2) Capacity kN	Critical(3) Design Check	Length ℓ mm	Mark	S275 mm	S355 mm	Single mm	Double mm	Capacity kN	Critical Check
457x152x82	5	740	4(ii)	532	4(ii)	360	CB5	8.0	6.7	158	100	555	11
{1010 kN}	4	534	4 (iii)	408	4(ii)	290	CB4	7.3	6.1	294	100*	448	11
457x152x74	5	675	4(ii)	486	4(ii)	360	CB5	7.3	6.1	151	100	555	11
{918 kN}	4	488	4 (iii)	373	4(ii)	290	CB4	6.6	5.5	289	100*	448	11
457x152x67	5	638	4 (iii)	469	4(ii)	360	CB5	6.9	5.8	154	100	555	11
{878 kN}	4	458	4 (iii)	360	4(ii)	290	CB4	6.2	5.2	290	100*	448	11
457x152x60	5	574	4 (iii)	422	4(ii)	360	CB5	6.2	5.2	146	100	555	11
{784 kN}	4	412	4 (iii)	324	4(ii)	290	CB4	5.6	4.7	286	100*	448	11
457x152x52	5	539	4 (iii)	396	4(ii)	360	CB5	5.9	4.9	132	100	555	11
{728 kN}	4	387	4 (iii)	304	4(ii)	290	CB4	5.3	4.4	249	100*	448	11
406x178x74	4	483	4 (iii)	379	4(ii)	290	CB4	6.6	5.5	238	100	448	11
{835 kN}													
406x178x67	4	448	4 (iii)	352	4(ii)	290	CB4	6.1	5.1	234	100	448	11
{767 kN}													
406x178x60	4	402	4 (iii)	316	4(ii)	290	CB4	5.5	4.6	231	100	448	11
{684 kN}													
406x178x54	4	392	4 (iii)	308	4(ii)	290	CB4	5.3	4.5	223	100	448	11
{660 kN}													
406x140x46	4	346	4 (iii)	272	4(ii)	290	CB4	4.7	3.9	221	100	448	11
{584 kN}													
406x140x39	4	325	4 (iii)	256	4(ii)	290	CB4	4.4	3.7	192	100	448	11
{543 kN}													
356x171x67	3	300	4 (iii)	251	4(ii)	220	CB3	5.4	4.5	278	100*	340	11
{704 kN}													
356x171x57	3	267	4 (iii)	224	4(ii)	220	CB3	4.8	4.0	268	100*	340	11
{618 kN}													
356x171x51	3	244	4 (iii)	204	4(ii)	220	CB3	4.4	3.7	263	100*	340	11
{560 kN}													
356x171x45	3	231	4 (iii)	193	4(ii)	220	CB3	4.2	3.5	255	100*	340	11
{524 kN}													
356x127x39	3	217	4 (iii)	182	4(ii)	220	CB3	3.9	3.3	251	100*	340	11
{497 kN}													
356x127x33	3	198	4 (iii)	166	4(ii)	220	CB3	3.6	3.0	205	100*	340	11
{446 kN}													

For guidance on the use of tables see Explanatory notes in Table H.6

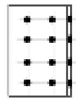
BEAM: S355
ANGLE CLEATS: S275
BOLTS: M20, 8.8

Table H.12 *Continued*



DOUBLE ANGLE CLEATS, ORDINARY or FLOWDRILL BOLTS

Double line of bolts
2 No.150x90x10mm Angles

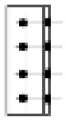


Beam Size { } (1)	Bolt Rows n	Un-notched or Single Notch: Shear(2) Capacity kN	Un-notched or Single Notch: Critical(3) Design Check	Double Notch: Shear(2) Capacity kN	Double Notch: Critical(3) Design Check	Fitting(4) Angle Cleats: Length ℓ mm	Fitting(4) Angle Cleats: Mark	Min. Support(5) Thickness: S275 mm	Min. Support(5) Thickness: S355 mm	Max Notch(6) Length $c+t_1$: Single mm	Max Notch(6) Length $c+t_1$: Double mm	Tying(7): Capacity kN	Tying(7): Critical Check
305x165x54 {522 kN}	3	260	4 (iii)	218	4(ii)	220	CB3	4.7	3.9	196	100	340	11
305x165x46 {438 kN}	3	221	4 (iii)	185	4(ii)	220	CB3	4.0	3.3	192	100	340	11
305x165x40 {388 kN}	3	198	4 (iii)	166	4(ii)	220	CB3	3.6	3.0	187	100	340	11
305x127x48 {596 kN}	3	296	4 (iii)	249	4(ii)	220	CB3	5.4	4.5	185	100	340	11
305x127x42 {523 kN}	3	263	4 (iii)	221	4(ii)	220	CB3	4.8	4.0	181	100	340	11
305x127x37 {460 kN}	3	234	4 (iii)	196	4(ii)	220	CB3	4.2	3.5	178	100	340	11
305x102x33 {440 kN}	3	217	4 (iii)	182	4(ii)	220	CB3	3.9	3.3	185	100	340	11
305x102x28 {395 kN}	3	198	4 (iii)	166	4(ii)	220	CB3	3.6	3.0	178	100	340	11
305x102x25 {377 kN}	3	191	4 (iii)	160	4(ii)	220	CB3	3.5	2.9	169	100	340	11
254x146x43 {398 kN}	2	129	4 (iii)	96	4(ii)	150	CB2	3.5	2.9	235	100*	233	11
254x146x37 {344 kN}	2	113	4 (iii)	84	4(ii)	150	CB2	3.1	2.6	229	100*	233	11
254x146x31 {321 kN}	2	107	4 (iii)	80	4(ii)	150	CB2	2.9	2.4	216	100*	233	11
254x102x28 {349 kN}	2	113	4 (iii)	84	4(ii)	150	CB2	3.1	2.6	225	100*	233	11
254x102x25 {329 kN}	2	107	4 (iii)	80	4(ii)	150	CB2	2.9	2.4	215	100*	233	11
254x102x22 {308 kN}	2	102	4 (iii)	76	4(ii)	150	CB2	2.8	2.3	205	100*	233	11

For guidance on the use of tables see Explanatory notes in Table H.6

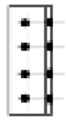
BEAM: S275
ANGLE CLEATS: S275
HOLLO-BOLTS: M20,8.8

Table H.13



DOUBLE ANGLE CLEATS, HOLLO-BOLTS

Single line of bolts
2 No. 90x90x10mm Equal Angles

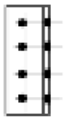


Beam Size { } [1]	Bolt Rows n	Un-notched: Shear[2] Capacity kN	Un-notched: Critical[3] Design Check	Fitting[4] Angle Cleats: Length ℓ mm	Fitting[4] Angle Cleats: Mark	Min. Support[5] Thickness: S275 mm	Min. Support[5] Thickness: S355 mm	Tying[7]: Capacity kN	Tying[7]: Critical Check
914x305x253	11	1460	9(i)	780	CC11	7.2	6.1	1050	11
{2530 kN}	10	1340	9(i)	710	CC10	7.3	6.1	953	11
	9	1210	9(i)	640	CC9	7.3	6.1	860	11
	8	1080	9(i)	570	CC8	7.3	6.1	766	11
914x305x224	11	1460	9(i)	780	CC11	7.2	6.1	1050	11
{2300 kN}	10	1340	9(i)	710	CC10	7.3	6.1	953	11
	9	1210	9(i)	640	CC9	7.3	6.1	860	11
	8	1060	4 (iii)	570	CC8	7.2	6.0	766	11
914x305x201	11	1440	4 (iii)	780	CC11	7.1	5.9	1050	11
{2170 kN}	10	1290	4 (iii)	710	CC10	7.0	5.9	953	11
	9	1150	4 (iii)	640	CC9	6.9	5.8	860	11
	8	1000	4 (iii)	570	CC8	6.8	5.7	766	11
838x292x226	10	1340	9(i)	710	CC10	7.3	6.1	953	11
{2180 kN}	9	1210	9(i)	640	CC9	7.3	6.1	860	11
	8	1070	4 (iii)	570	CC8	7.3	6.1	766	11
838x292x194	10	1260	4 (iii)	710	CC10	6.8	5.7	953	11
{1960 kN}	9	1120	4 (iii)	640	CC9	6.8	5.7	860	11
	8	977	4 (iii)	570	CC8	6.6	5.6	766	11
838x292x176	10	1200	4 (iii)	710	CC10	6.5	5.5	953	11
{1860 kN}	9	1070	4 (iii)	640	CC9	6.4	5.4	860	11
	8	930	4 (iii)	570	CC8	6.3	5.3	766	11
762x267x197	9	1190	4 (iii)	640	CC9	7.2	6.0	860	11
{1910 kN}	8	1040	4 (iii)	570	CC8	7.0	5.9	766	11
	7	886	4 (iii)	500	CC7	6.9	5.8	672	11
762x267x173	9	1090	4 (iii)	640	CC9	6.6	5.5	860	11
{1730 kN}	8	950	4 (iii)	570	CC8	6.5	5.4	766	11
	7	812	4 (iii)	500	CC7	6.3	5.3	672	11
762x267x147	9	974	4 (iii)	640	CC9	5.9	4.9	860	11
{1530 kN}	8	851	4 (iii)	570	CC8	5.8	4.8	766	11
	7	727	4 (iii)	500	CC7	5.6	4.7	672	11
762x267x134	9	913	4 (iii)	640	CC9	5.5	4.6	860	11
{1490 kN}	8	797	4 (iii)	570	CC8	5.4	4.5	766	11
	7	681	4 (iii)	500	CC7	5.3	4.4	672	11

For guidance on the use of tables see Explanatory notes in Table H.6

BEAM: S275
ANGLE CLEATS: S275
HOLLO-BOLTS: M20,8.8

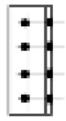
Table H.13 *Continued*



DOUBLE ANGLE CLEATS, HOLLO-BOLTS

Single line of bolts

2 No. 90x90x10mm Equal Angles

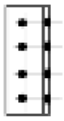


Beam Size { } (1)	Bolt Rows n	Un-notched		Fitting(4) Angle Cleats		Min. Support(5) Thickness		Tying(7)	
		Shear(2) Capacity kN	Critical(3) Design Check	Length ℓ mm	Mark	S275 mm	S355 mm	Capacity kN	Critical Check
686x254x170	8	964	4 (iii)	570	CC8	6.5	5.5	766	11
{1600 kN}	7	823	4 (iii)	500	CC7	6.4	5.3	672	11
	6	683	4 (iii)	430	CC6	6.2	5.2	580	11
686x254x152	8	877	4 (iii)	570	CC8	6.0	5.0	766	11
{1440 kN}	7	749	4 (iii)	500	CC7	5.8	4.9	672	11
	6	621	4 (iii)	430	CC6	5.6	4.7	580	11
686x254x140	8	824	4 (iii)	570	CC8	5.6	4.7	766	11
{1350 kN}	7	704	4 (iii)	500	CC7	5.5	4.6	672	11
	6	584	4 (iii)	430	CC6	5.3	4.4	580	11
686x254x125	8	777	4 (iii)	570	CC8	5.3	4.4	766	11
{1260 kN}	7	664	4 (iii)	500	CC7	5.2	4.3	672	11
	6	551	4 (iii)	430	CC6	5.0	4.2	580	11
610x305x238	7	947	9(i)	500	CC7	7.4	6.2	672	11
{1860 kN}	6	818	9(i)	430	CC6	7.4	6.2	580	11
610x305x179	7	800	4 (iii)	500	CC7	6.2	5.2	672	11
{1390 kN}	6	664	4 (iii)	430	CC6	6.0	5.0	580	11
610x305x149	7	670	4 (iii)	500	CC7	5.2	4.3	672	11
{1150 kN}	6	556	4 (iii)	430	CC6	5.0	4.2	580	11
610x229x140	7	744	4 (iii)	500	CC7	5.8	4.8	672	11
{1290 kN}	6	617	4 (iii)	430	CC6	5.6	4.7	580	11
610x229x125	7	676	4 (iii)	500	CC7	5.2	4.4	672	11
{1160 kN}	6	560	4 (iii)	430	CC6	5.1	4.2	580	11
610x229x113	7	630	4 (iii)	500	CC7	4.9	4.1	672	11
{1070 kN}	6	523	4 (iii)	430	CC6	4.7	4.0	580	11
610x229x101	7	596	4 (iii)	500	CC7	4.6	3.9	672	11
{1040 kN}	6	494	4 (iii)	430	CC6	4.5	3.7	580	12 (ii)

For guidance on the use of tables see Explanatory notes in Table H.6

BEAM: S275
ANGLE CLEATS: S275
HOLLO-BOLTS: M20,8.8

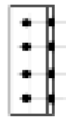
Table H.13 *Continued*



DOUBLE ANGLE CLEATS, HOLLO-BOLTS

Single line of bolts

2 No. 90x90x10mm Equal Angles

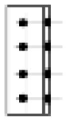


Beam Size { } (1)	Bolt Rows n	Un-notched: Shear(2) Capacity kN	Un-notched: Critical(3) Design Check	Fitting(4) Angle Cleats: Length ℓ mm	Fitting(4) Angle Cleats: Mark	Min. Support(5) Thickness: S275 mm	Min. Support(5) Thickness: S355 mm	Tying(7): Capacity kN	Tying(7): Critical Check
533x210x122	6	598	4 (iii)	430	CC6	5.4	4.5	580	11
{1100 kN}	5	475	4 (iii)	360	CC5	5.2	4.3	486	11
533x210x109	6	546	4 (iii)	430	CC6	4.9	4.1	580	11
{995 kN}	5	434	4 (iii)	360	CC5	4.7	3.9	486	11
533x210x101	6	508	4 (iii)	430	CC6	4.6	3.9	580	11
{922 kN}	5	404	4 (iii)	360	CC5	4.4	3.7	486	11
533x210x92	6	475	4 (iii)	430	CC6	4.3	3.6	558	12 (ii)
{888 kN}	5	378	4 (iii)	360	CC5	4.1	3.4	465	12 (ii)
533x210x82	6	452	4 (iii)	430	CC6	4.1	3.4	530	12 (ii)
{837 kN}	5	359	4 (iii)	360	CC5	3.9	3.3	442	12 (ii)
457x191x98	5	427	4 (iii)	360	CC5	4.6	3.9	486	11
{847 kN}	4	319	4 (iii)	290	CC4	4.3	3.6	393	11
457x191x89	5	393	4 (iii)	360	CC5	4.3	3.6	483	12 (ii)
{774 kN}	4	293	4 (iii)	290	CC4	4.0	3.3	386	12 (ii)
457x191x82	5	371	4 (iii)	360	CC5	4.0	3.4	455	12 (ii)
{751 kN}	4	277	4 (iii)	290	CC4	3.8	3.1	364	12 (ii)
457x191x74	5	337	4 (iii)	360	CC5	3.7	3.1	414	12 (ii)
{679 kN}	4	251	4 (iii)	290	CC4	3.4	2.9	331	12 (ii)
457x191x67	5	318	4 (iii)	360	CC5	3.5	2.9	391	12 (ii)
{636 kN}	4	237	4 (iii)	290	CC4	3.2	2.7	313	12 (ii)

For guidance on the use of tables see Explanatory notes in Table H.6

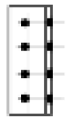
BEAM: S275
ANGLE CLEATS: S275
HOLLO-BOLTS: M20,8.8

Table H.13 *Continued*



DOUBLE ANGLE CLEATS, HOLLO-BOLTS

Single line of bolts
2 No. 90x90x10mm Equal Angles

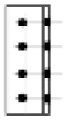


Beam Size { } (1)	Bolt Rows n	Un-notched: Shear(2) Capacity kN	Un-notched: Critical(3) Design Check	Fitting(4) Angle Cleats: Length ℓ mm	Fitting(4) Angle Cleats: Mark	Min. Support(5) Thickness: S275 mm	Min. Support(5) Thickness: S355 mm	Tying(7): Capacity kN	Tying(7): Critical Check
457x152x82	5	393	4 (iii)	360	CC5	4.3	3.6	483	12 (ii)
{778 kN}	4	293	4 (iii)	290	CC4	4.0	3.3	386	12 (ii)
457x152x74	5	359	4 (iii)	360	CC5	3.9	3.3	442	12 (ii)
{705 kN}	4	268	4 (iii)	290	CC4	3.6	3.0	353	12 (ii)
457x152x67	5	337	4 (iii)	360	CC5	3.7	3.1	414	12 (ii)
{680 kN}	4	251	4 (iii)	290	CC4	3.4	2.9	331	12 (ii)
457x152x60	5	303	4 (iii)	360	CC5	3.3	2.8	373	12 (ii)
{608 kN}	4	226	4 (iii)	290	CC4	3.1	2.6	298	12 (ii)
457x152x52	5	284	4 (iii)	360	CC5	3.1	2.6	350	12 (ii)
{564 kN}	4	212	4 (iii)	290	CC4	2.9	2.4	280	12 (ii)
406x178x74	4	265	4 (iii)	290	CC4	3.6	3.0	350	12 (ii)
{647 kN}									
406x178x67	4	246	4 (iii)	290	CC4	3.3	2.8	324	12 (ii)
{594 kN}									
406x178x60	4	221	4 (iii)	290	CC4	3.0	2.5	291	12 (ii)
{530 kN}									
406x178x54	4	215	4 (iii)	290	CC4	2.9	2.4	283	12 (ii)
{512 kN}									
406x140x46	4	190	4 (iii)	290	CC4	2.6	2.2	250	12 (ii)
{452 kN}									
406x140x39	4	179	4 (iii)	290	CC4	2.4	2.0	236	12 (ii)
{420 kN}									
356x171x67	3	171	4 (iii)	220	CC3	3.1	2.6	251	12 (ii)
{546 kN}									
356x171x57	3	153	4 (iii)	220	CC3	2.8	2.3	224	12 (ii)
{478 kN}									
356x171x51	3	139	4 (iii)	220	CC3	2.5	2.1	204	12 (ii)
{433 kN}									
356x171x45	3	132	4 (iii)	220	CC3	2.4	2.0	193	12 (ii)
{406 kN}									
356x127x39	3	124	4 (iii)	220	CC3	2.3	1.9	182	12 (ii)
{385 kN}									
356x127x33	3	113	4 (iii)	220	CC3	2.0	1.7	166	12 (ii)
{346 kN}									

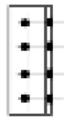
For guidance on the use of tables see Explanatory notes in Table H.6

BEAM: S275
ANGLE CLEATS: S275
HOLLO-BOLTS: M20,8.8

Table H.13 *Continued*



DOUBLE ANGLE CLEATS, HOLLO-BOLTS



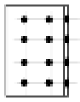
Single line of bolts
2 No. 90x90x10mm Equal Angles

Beam Size { } (1)	Bolt Rows n	Un-notched		Fitting(4) Angle Cleats		Min. Support(5) Thickness		Tying(7)	
		Shear(2) Capacity kN	Critical(3) Design Check	Length ℓ mm	Mark	S275 mm	S355 mm	Capacity kN	Critical Check
305x165x54 {405 kN}	3	149	4 (iii)	220	CC3	2.7	2.3	218	12 (ii)
305x165x46 {339 kN}	3	126	4 (iii)	220	CC3	2.3	1.9	185	12 (ii)
305x165x40 {300 kN}	3	113	4 (iii)	220	CC3	2.0	1.7	166	12 (ii)
305x127x48 {462 kN}	3	169	4 (iii)	220	CC3	3.1	2.6	248	12 (ii)
305x127x42 {406 kN}	3	151	4 (iii)	220	CC3	2.7	2.3	221	12 (ii)
305x127x37 {357 kN}	3	134	4 (iii)	220	CC3	2.4	2.0	196	12 (ii)
305x102x33 {341 kN}	3	124	4 (iii)	220	CC3	2.3	1.9	182	12 (ii)
305x102x28 {306 kN}	3	113	4 (iii)	220	CC3	2.0	1.7	166	12 (ii)
305x102x25 {292 kN}	3	109	4 (iii)	220	CC3	2.0	1.7	160	12 (ii)
254x146x43 {308 kN}	2	76	4 (iii)	150	CC2	2.1	1.7	132	12 (ii)
254x146x37 {266 kN}	2	66	4 (iii)	150	CC2	1.8	1.5	116	12 (ii)
254x146x31 {249 kN}	2	63	4 (iii)	150	CC2	1.7	1.4	110	12 (ii)
254x102x28 {271 kN}	2	66	4 (iii)	150	CC2	1.8	1.5	116	12 (ii)
254x102x25 {255 kN}	2	63	4 (iii)	150	CC2	1.7	1.4	110	12 (ii)
254x102x22 {239 kN}	2	60	4 (iii)	150	CC2	1.6	1.4	105	12 (ii)

For guidance on the use of tables see Explanatory notes in Table H.6

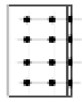
BEAM: S275
ANGLE CLEATS: S275
HOLLO-BOLTS: M20,8.8

Table H.14



DOUBLE ANGLE CLEATS, HOLLO-BOLTS

Double line of bolts
2 No.150x90x10mm Angles



Beam Size { } [1]	Bolt Rows n	Un-notched		Fitting[4] Angle Cleats		Min. Support[5] Thickness		Tying[7]	
		Shear[2] Capacity kN	Critical[3] Design Check	Length ℓ mm	Mark	S275 mm	S355 mm	Capacity kN	Critical Check
914x305x253	11	1460	9(i)	780	CD11	7.2	6.1	1050	11
{2530 kN}	10	1340	9(i)	710	CD10	7.3	6.1	953	11
	9	1210	9(i)	640	CD9	7.3	6.1	860	11
	8	1080	9(i)	570	CD8	7.3	6.1	766	11
914x305x224	11	1460	9(i)	780	CD11	7.2	6.1	1050	11
{2300 kN}	10	1340	9(i)	710	CD10	7.3	6.1	953	11
	9	1210	9(i)	640	CD9	7.3	6.1	860	11
	8	1080	9(i)	570	CD8	7.3	6.1	766	11
914x305x201	11	1460	9(i)	780	CD11	7.2	6.1	1050	11
{2170 kN}	10	1340	9(i)	710	CD10	7.3	6.1	953	11
	9	1210	9(i)	640	CD9	7.3	6.1	860	11
	8	1080	9(i)	570	CD8	7.3	6.1	766	11
838x292x226	10	1340	9(i)	710	CD10	7.3	6.1	953	11
{2180 kN}	9	1210	9(i)	640	CD9	7.3	6.1	860	11
	8	1080	9(i)	570	CD8	7.3	6.1	766	11
838x292x194	10	1340	9(i)	710	CD10	7.3	6.1	953	11
{1960 kN}	9	1210	9(i)	640	CD9	7.3	6.1	860	11
	8	1080	9(i)	570	CD8	7.3	6.1	766	11
838x292x176	10	1340	9(i)	710	CD10	7.3	6.1	953	11
{1860 kN}	9	1210	9(i)	640	CD9	7.3	6.1	860	11
	8	1080	9(i)	570	CD8	7.3	6.1	766	11
762x267x197	9	1210	9(i)	640	CD9	7.3	6.1	860	11
{1910 kN}	8	1080	9(i)	570	CD8	7.3	6.1	766	11
	7	947	9(i)	500	CD7	7.4	6.2	672	11
762x267x173	9	1210	9(i)	640	CD9	7.3	6.1	860	11
{1730 kN}	8	1080	9(i)	570	CD8	7.3	6.1	766	11
	7	947	9(i)	500	CD7	7.4	6.2	672	11
762x267x147	9	1210	9(i)	640	CD9	7.3	6.1	860	11
{1530 kN}	8	1080	9(i)	570	CD8	7.3	6.1	766	11
	7	947	9(i)	500	CD7	7.4	6.2	672	11
762x267x134	9	1210	9(i)	640	CD9	7.3	6.1	860	11
{1490 kN}	8	1080	9(i)	570	CD8	7.3	6.1	766	11
	7	947	9(i)	500	CD7	7.4	6.2	672	11

For guidance on the use of tables see Explanatory notes in Table H.6

BEAM: S275
ANGLE CLEATS: S275
HOLLO-BOLTS: M20,8.8

Table H.14 *Continued*

DOUBLE ANGLE CLEATS, HOLLO-BOLTS

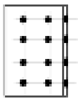
Double line of bolts
2 No.150x90x10mm Angles

Beam Size	Bolt Rows	Un-notched		Fitting[4] Angle Cleats		Min. Support[5] Thickness		Tying[7]	
{ } [1]	n	Shear[2] Capacity kN	Critical[3] Design Check	Length ℓ mm	Mark	S275 mm	S355 mm	Capacity kN	Critical Check
686x254x170	8	1080	9(i)	570	CD8	7.3	6.1	766	11
{1600 kN}	7	947	9(i)	500	CD7	7.4	6.2	672	11
	6	818	9(i)	430	CD6	7.4	6.2	580	11
686x254x152	8	1080	9(i)	570	CD8	7.3	6.1	766	11
{1440 kN}	7	947	9(i)	500	CD7	7.4	6.2	672	11
	6	818	9(i)	430	CD6	7.4	6.2	580	11
686x254x140	8	1080	9(i)	570	CD8	7.3	6.1	766	11
{1350 kN}	7	947	9(i)	500	CD7	7.4	6.2	672	11
	6	818	9(i)	430	CD6	7.4	6.2	580	11
686x254x125	8	1040	4(ii)	570	CD8	7.0	5.9	766	11
{1260 kN}	7	915	4(ii)	500	CD7	7.1	5.9	672	11
	6	791	4(ii)	430	CD6	7.2	6.0	580	11
610x305x238	7	947	9(i)	500	CD7	7.4	6.2	672	11
{1860 kN}	6	818	9(i)	430	CD6	7.4	6.2	580	11
610x305x179	7	947	9(i)	500	CD7	7.4	6.2	672	11
{1390 kN}	6	818	9(i)	430	CD6	7.4	6.2	580	11
610x305x149	7	916	4(ii)	500	CD7	7.1	5.9	672	11
{1150 kN}	6	794	4(ii)	430	CD6	7.2	6.0	580	11
610x229x140	7	947	9(i)	500	CD7	7.4	6.2	672	11
{1290 kN}	6	818	9(i)	430	CD6	7.4	6.2	580	11
610x229x125	7	921	4(ii)	500	CD7	7.2	6.0	672	11
{1160 kN}	6	799	4(ii)	430	CD6	7.2	6.1	580	11
610x229x113	7	858	4(ii)	500	CD7	6.7	5.6	672	11
{1070 kN}	6	745	4(ii)	430	CD6	6.7	5.6	580	11
610x229x101	7	841	4(ii)	500	CD7	6.5	5.5	672	11
{1040 kN}	6	730	4(ii)	430	CD6	6.6	5.5	580	11

For guidance on the use of tables see Explanatory notes in Table H.6

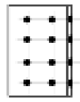
BEAM: S275
ANGLE CLEATS: S275
HOLLO-BOLTS: M20,8.8

Table H.14 *Continued*



DOUBLE ANGLE CLEATS, HOLLO-BOLTS

Double line of bolts
2 No.150x90x10mm Angles

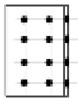


Beam Size { } (1)	Bolt Rows n	Un-notched: Shear(2) Capacity kN	Un-notched: Critical(3) Design Check	Fitting(4) Angle Cleats: Length ℓ mm	Fitting(4) Angle Cleats: Mark	Min. Support(5) Thickness: S275 mm	Min. Support(5) Thickness: S355 mm	Tying(7): Capacity kN	Tying(7): Critical Check
533x210x122	6	818	9(i)	430	CD6	7.4	6.2	580	11
{1100 kN}	5	688	9(i)	360	CD5	7.5	6.3	486	11
533x210x109	6	766	4(ii)	430	CD6	6.9	5.8	580	11
{995 kN}	5	649	4(ii)	360	CD5	7.1	5.9	486	11
533x210x101	6	713	4(ii)	430	CD6	6.5	5.4	580	11
{922 kN}	5	604	4(ii)	360	CD5	6.6	5.5	486	11
533x210x92	6	691	4(ii)	430	CD6	6.3	5.2	580	11
{888 kN}	5	586	4(ii)	360	CD5	6.4	5.3	486	11
533x210x82	6	655	4(ii)	430	CD6	5.9	5.0	580	11
{837 kN}	5	556	4(ii)	360	CD5	6.0	5.1	486	11
457x191x98	5	624	4(ii)	360	CD5	6.8	5.7	486	11
{847 kN}	4	485	4 (iii)	290	CD4	6.6	5.5	393	11
457x191x89	5	574	4(ii)	360	CD5	6.2	5.2	486	11
{774 kN}	4	447	4 (iii)	290	CD4	6.1	5.1	393	11
457x191x82	5	561	4(ii)	360	CD5	6.1	5.1	486	11
{751 kN}	4	421	4 (iii)	290	CD4	5.7	4.8	393	11
457x191x74	5	509	4(ii)	360	CD5	5.5	4.6	486	11
{679 kN}	4	383	4 (iii)	290	CD4	5.2	4.4	393	11
457x191x67	5	480	4(ii)	360	CD5	5.2	4.4	486	11
{636 kN}	4	362	4 (iii)	290	CD4	4.9	4.1	393	11

For guidance on the use of tables see Explanatory notes in Table H.6

BEAM: S275
ANGLE CLEATS: S275
HOLLO-BOLTS: M20,8.8

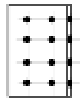
Table H.14 *Continued*



DOUBLE ANGLE CLEATS, HOLLO-BOLTS

Double line of bolts

2 No.150x90x10mm Angles

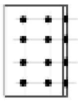


Beam Size { }[1]	Bolt Rows n	Un-notched		Fitting[4] Angle Cleats		Min. Support[5] Thickness		Tying[7]	
		Shear[2] Capacity kN	Critical[3] Design Check	Length ℓ mm	Mark	S275 mm	S355 mm	Capacity kN	Critical Check
457x152x82	5	573	4(ii)	360	CD5	6.2	5.2	486	11
{778 kN}	4	447	4 (iii)	290	CD4	6.1	5.1	393	11
457x152x74	5	523	4(ii)	360	CD5	5.7	4.8	486	11
{705 kN}	4	408	4 (iii)	290	CD4	5.5	4.6	393	11
457x152x67	5	508	4(ii)	360	CD5	5.5	4.6	486	11
{680 kN}	4	383	4 (iii)	290	CD4	5.2	4.4	393	11
457x152x60	5	456	4(ii)	360	CD5	5.0	4.1	486	11
{608 kN}	4	345	4 (iii)	290	CD4	4.7	3.9	393	11
457x152x52	5	426	4(ii)	360	CD5	4.6	3.9	486	11
{564 kN}	4	323	4 (iii)	290	CD4	4.4	3.7	393	11
406x178x74 {647 kN}	4	404	4 (iii)	290	CD4	5.5	4.6	393	11
406x178x67 {594 kN}	4	374	4 (iii)	290	CD4	5.1	4.3	393	11
406x178x60 {530 kN}	4	336	4 (iii)	290	CD4	4.6	3.8	393	11
406x178x54 {512 kN}	4	328	4 (iii)	290	CD4	4.5	3.7	393	11
406x140x46 {452 kN}	4	289	4 (iii)	290	CD4	3.9	3.3	393	11
406x140x39 {420 kN}	4	272	4 (iii)	290	CD4	3.7	3.1	393	11
356x171x67 {546 kN}	3	251	4 (iii)	220	CD3	4.5	3.8	299	11
356x171x57 {478 kN}	3	223	4 (iii)	220	CD3	4.0	3.4	299	11
356x171x51 {433 kN}	3	204	4 (iii)	220	CD3	3.7	3.1	299	11
356x171x45 {406 kN}	3	193	4 (iii)	220	CD3	3.5	2.9	299	11
356x127x39 {385 kN}	3	182	4 (iii)	220	CD3	3.3	2.8	299	11
356x127x33 {346 kN}	3	165	4 (iii)	220	CD3	3.0	2.5	299	11

For guidance on the use of tables see Explanatory notes in Table H.6

BEAM: S275
ANGLE CLEATS: S275
HOLLO-BOLTS: M20,8.8

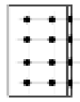
Table H.14 *Continued*



DOUBLE ANGLE CLEATS, HOLLO-BOLTS

Double line of bolts

2 No.150x90x10mm Angles

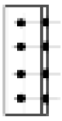


Beam Size { } (1)	Bolt Rows n	Un-notched		Fitting(4) Angle Cleats		Min. Support(5) Thickness		Tying(7)	
		Shear(2) Capacity kN	Critical(3) Design Check	Length ℓ mm	Mark	S275 mm	S355 mm	Capacity kN	Critical Check
305x165x54 {405 kN}	3	218	4 (iii)	220	CD3	3.9	3.3	299	11
305x165x46 {339 kN}	3	185	4 (iii)	220	CD3	3.3	2.8	299	11
305x165x40 {300 kN}	3	165	4 (iii)	220	CD3	3.0	2.5	299	11
305x127x48 {462 kN}	3	248	4 (iii)	220	CD3	4.5	3.8	299	11
305x127x42 {406 kN}	3	220	4 (iii)	220	CD3	4.0	3.3	299	11
305x127x37 {357 kN}	3	196	4 (iii)	220	CD3	3.5	3.0	299	11
305x102x33 {341 kN}	3	182	4 (iii)	220	CD3	3.3	2.8	299	11
305x102x28 {306 kN}	3	165	4 (iii)	220	CD3	3.0	2.5	299	11
305x102x25 {292 kN}	3	160	4 (iii)	220	CD3	2.9	2.4	299	11
254x146x43 {308 kN}	2	108	4 (iii)	150	CD2	2.9	2.4	206	11
254x146x37 {266 kN}	2	94	4 (iii)	150	CD2	2.6	2.1	206	11
254x146x31 {249 kN}	2	90	4 (iii)	150	CD2	2.4	2.0	206	11
254x102x28 {271 kN}	2	94	4 (iii)	150	CD2	2.6	2.1	206	11
254x102x25 {255 kN}	2	90	4 (iii)	150	CD2	2.4	2.0	206	11
254x102x22 {239 kN}	2	85	4 (iii)	150	CD2	2.3	1.9	206	11

For guidance on the use of tables see Explanatory notes in Table H.6

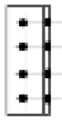
BEAM: S355
ANGLE CLEATS: S275
HOLLO-BOLTS: M20,8.8

Table H.15



DOUBLE ANGLE CLEATS, HOLLO-BOLTS

Single line of bolts
2 No. 90x90x10mm Equal Angles

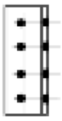


Beam Size	Bolt Rows	Un-notched		Fitting[4] Angle Cleats		Min. Support[5] Thickness		Tying[7]	
{ }[1]	n	Shear[2] Capacity kN	Critical[3] Design Check	Length ℓ mm	Mark	S275 mm	S355 mm	Capacity kN	Critical Check
914x305x253	11	1460	9(i)	780	CC11	7.2	6.1	1050	11
{3290 kN}	10	1340	9(i)	710	CC10	7.3	6.1	953	11
	9	1210	9(i)	640	CC9	7.3	6.1	860	11
	8	1080	9(i)	570	CC8	7.3	6.1	766	11
914x305x224	11	1460	9(i)	780	CC11	7.2	6.1	1050	11
{3000 kN}	10	1340	9(i)	710	CC10	7.3	6.1	953	11
	9	1210	9(i)	640	CC9	7.3	6.1	860	11
	8	1080	9(i)	570	CC8	7.3	6.1	766	11
914x305x201	11	1460	9(i)	780	CC11	7.2	6.1	1050	11
{2820 kN}	10	1340	9(i)	710	CC10	7.3	6.1	953	11
	9	1210	9(i)	640	CC9	7.3	6.1	860	11
	8	1080	9(i)	570	CC8	7.3	6.1	766	11
838x292x226	10	1340	9(i)	710	CC10	7.3	6.1	953	11
{2840 kN}	9	1210	9(i)	640	CC9	7.3	6.1	860	11
	8	1080	9(i)	570	CC8	7.3	6.1	766	11
838x292x194	10	1340	9(i)	710	CC10	7.3	6.1	953	11
{2560 kN}	9	1210	9(i)	640	CC9	7.3	6.1	860	11
	8	1080	9(i)	570	CC8	7.3	6.1	766	11
838x292x176	10	1340	9(i)	710	CC10	7.3	6.1	953	11
{2420 kN}	9	1210	9(i)	640	CC9	7.3	6.1	860	11
	8	1080	9(i)	570	CC8	7.3	6.1	766	11
762x267x197	9	1210	9(i)	640	CC9	7.3	6.1	860	11
{2490 kN}	8	1080	9(i)	570	CC8	7.3	6.1	766	11
	7	947	9(i)	500	CC7	7.4	6.2	672	11
762x267x173	9	1210	9(i)	640	CC9	7.3	6.1	860	11
{2260 kN}	8	1080	9(i)	570	CC8	7.3	6.1	766	11
	7	947	9(i)	500	CC7	7.4	6.2	672	11
762x267x147	9	1160	4 (iii)	640	CC9	7.0	5.9	860	11
{2000 kN}	8	1020	4 (iii)	570	CC8	6.9	5.8	766	11
	7	869	4 (iii)	500	CC7	6.7	5.6	672	11
762x267x134	9	1090	4 (iii)	640	CC9	6.6	5.5	860	11
{1920 kN}	8	953	4 (iii)	570	CC8	6.5	5.4	766	11
	7	814	4 (iii)	500	CC7	6.3	5.3	672	11

For guidance on the use of tables see Explanatory notes in Table H.6

BEAM: S355
ANGLE CLEATS: S275
HOLLO-BOLTS: M20,8.8

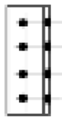
Table H.15 *Continued*



DOUBLE ANGLE CLEATS, HOLLO-BOLTS

Single line of bolts

2 No. 90x90x10mm Equal Angles

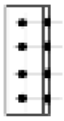


Beam Size { } (1)	Bolt Rows n	Un-notched		Fitting(4) Angle Cleats		Min. Support(5) Thickness		Tying(7)	
		Shear(2) Capacity kN	Critical(3) Design Check	Length ℓ mm	Mark	S275 mm	S355 mm	Capacity kN	Critical Check
686x254x170	8	1080	9(i)	570	CC8	7.3	6.1	766	11
{2080 kN}	7	947	9(i)	500	CC7	7.4	6.2	672	11
	6	816	4 (iii)	430	CC6	7.4	6.2	580	11
686x254x152	8	1050	4 (iii)	570	CC8	7.1	6.0	766	11
{1880 kN}	7	896	4 (iii)	500	CC7	7.0	5.8	672	11
	6	743	4 (iii)	430	CC6	6.7	5.6	580	11
686x254x140	8	985	4 (iii)	570	CC8	6.7	5.6	766	11
{1750 kN}	7	842	4 (iii)	500	CC7	6.5	5.5	672	11
	6	698	4 (iii)	430	CC6	6.3	5.3	580	11
686x254x125	8	930	4 (iii)	570	CC8	6.3	5.3	766	11
{1640 kN}	7	794	4 (iii)	500	CC7	6.2	5.2	672	11
	6	659	4 (iii)	430	CC6	6.0	5.0	580	11
610x305x238	7	947	9(i)	500	CC7	7.4	6.2	672	11
{2420 kN}	6	818	9(i)	430	CC6	7.4	6.2	580	11
610x305x179	7	947	9(i)	500	CC7	7.4	6.2	672	11
{1810 kN}	6	794	4 (iii)	430	CC6	7.2	6.0	580	11
610x305x149	7	801	4 (iii)	500	CC7	6.2	5.2	672	11
{1500 kN}	6	664	4 (iii)	430	CC6	6.0	5.0	580	11
610x229x140	7	889	4 (iii)	500	CC7	6.9	5.8	672	11
{1670 kN}	6	737	4 (iii)	430	CC6	6.7	5.6	580	11
610x229x125	7	808	4 (iii)	500	CC7	6.3	5.2	672	11
{1510 kN}	6	670	4 (iii)	430	CC6	6.1	5.1	580	11
610x229x113	7	753	4 (iii)	500	CC7	5.8	4.9	672	11
{1400 kN}	6	625	4 (iii)	430	CC6	5.7	4.7	580	11
610x229x101	7	713	4 (iii)	500	CC7	5.5	4.6	672	11
{1350 kN}	6	591	4 (iii)	430	CC6	5.4	4.5	580	11

For guidance on the use of tables see Explanatory notes in Table H.6

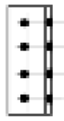
BEAM: S355
ANGLE CLEATS: S275
HOLLO-BOLTS: M20,8.8

Table H.15 *Continued*



DOUBLE ANGLE CLEATS, HOLLO-BOLTS

Single line of bolts
2 No. 90x90x10mm Equal Angles

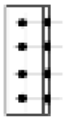


Beam Size { } (1)	Bolt Rows n	Un-notched		Fitting(4) Angle Cleats		Min. Support(5) Thickness		Tying(7)	
		Shear(2) Capacity kN	Critical(3) Design Check	Length ℓ mm	Mark	S275 mm	S355 mm	Capacity kN	Critical Check
533x210x122	6	715	4 (iii)	430	CC6	6.5	5.4	580	11
{1430 kN}	5	568	4 (iii)	360	CC5	6.2	5.2	486	11
533x210x109	6	653	4 (iii)	430	CC6	5.9	4.9	580	11
{1300 kN}	5	519	4 (iii)	360	CC5	5.6	4.7	486	11
533x210x101	6	608	4 (iii)	430	CC6	5.5	4.6	580	11
{1200 kN}	5	483	4 (iii)	360	CC5	5.3	4.4	486	11
533x210x92	6	569	4 (iii)	430	CC6	5.1	4.3	580	11
{1150 kN}	5	452	4 (iii)	360	CC5	4.9	4.1	486	11
533x210x82	6	540	4 (iii)	430	CC6	4.9	4.1	580	11
{1080 kN}	5	430	4 (iii)	360	CC5	4.7	3.9	486	11
457x191x98	5	510	4 (iii)	360	CC5	5.5	4.6	486	11
{1100 kN}	4	381	4 (iii)	290	CC4	5.2	4.3	393	11
457x191x89	5	470	4 (iii)	360	CC5	5.1	4.3	486	11
{1010 kN}	4	351	4 (iii)	290	CC4	4.8	4.0	393	11
457x191x82	5	443	4 (iii)	360	CC5	4.8	4.0	486	11
{970 kN}	4	331	4 (iii)	290	CC4	4.5	3.8	393	11
457x191x74	5	403	4 (iii)	360	CC5	4.4	3.7	486	11
{876 kN}	4	301	4 (iii)	290	CC4	4.1	3.4	393	11
457x191x67	5	380	4 (iii)	360	CC5	4.1	3.5	468	12 (ii)
{821 kN}	4	284	4 (iii)	290	CC4	3.9	3.2	374	12 (ii)

For guidance on the use of tables see Explanatory notes in Table H.6

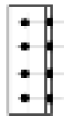
BEAM: S355
ANGLE CLEATS: S275
HOLLO-BOLTS: M20,8.8

Table H.15 *Continued*



DOUBLE ANGLE CLEATS, HOLLO-BOLTS

Single line of bolts
2 No. 90x90x10mm Equal Angles

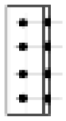


Beam Size	Bolt Rows	Un-notched		Fitting[4] Angle Cleats		Min. Support[5] Thickness		Tying[7]	
{ } [1]	n	Shear[2] Capacity kN	Critical[3] Design Check	Length ℓ mm	Mark	S275 mm	S355 mm	Capacity kN	Critical Check
457x152x82	5	470	4 (iii)	360	CC5	5.1	4.3	486	11
{1010 kN}	4	351	4 (iii)	290	CC4	4.8	4.0	393	11
457x152x74	5	430	4 (iii)	360	CC5	4.7	3.9	486	11
{918 kN}	4	321	4 (iii)	290	CC4	4.4	3.6	393	11
457x152x67	5	403	4 (iii)	360	CC5	4.4	3.7	486	11
{878 kN}	4	301	4 (iii)	290	CC4	4.1	3.4	393	11
457x152x60	5	363	4 (iii)	360	CC5	3.9	3.3	446	12 (ii)
{784 kN}	4	271	4 (iii)	290	CC4	3.7	3.1	356	12 (ii)
457x152x52	5	340	4 (iii)	360	CC5	3.7	3.1	418	12 (ii)
{728 kN}	4	254	4 (iii)	290	CC4	3.4	2.9	334	12 (ii)
406x178x74	4	317	4 (iii)	290	CC4	4.3	3.6	393	11
{835 kN}									
406x178x67	4	294	4 (iii)	290	CC4	4.0	3.3	387	12 (ii)
{767 kN}									
406x178x60	4	264	4 (iii)	290	CC4	3.6	3.0	348	12 (ii)
{684 kN}									
406x178x54	4	257	4 (iii)	290	CC4	3.5	2.9	339	12 (ii)
{660 kN}									
406x140x46	4	227	4 (iii)	290	CC4	3.1	2.6	299	12 (ii)
{584 kN}									
406x140x39	4	214	4 (iii)	290	CC4	2.9	2.4	282	12 (ii)
{543 kN}									
356x171x67	3	205	4 (iii)	220	CC3	3.7	3.1	299	11
{704 kN}									
356x171x57	3	182	4 (iii)	220	CC3	3.3	2.8	267	12 (ii)
{618 kN}									
356x171x51	3	167	4 (iii)	220	CC3	3.0	2.5	244	12 (ii)
{560 kN}									
356x171x45	3	158	4 (iii)	220	CC3	2.9	2.4	231	12 (ii)
{524 kN}									
356x127x39	3	149	4 (iii)	220	CC3	2.7	2.3	218	12 (ii)
{497 kN}									
356x127x33	3	135	4 (iii)	220	CC3	2.4	2.0	198	12 (ii)
{446 kN}									

For guidance on the use of tables see Explanatory notes in Table H.6

BEAM: S355
ANGLE CLEATS: S275
HOLLO-BOLTS: M20,8.8

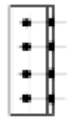
Table H.15 *Continued*



DOUBLE ANGLE CLEATS, HOLLO-BOLTS

Single line of bolts

2 No. 90x90x10mm Equal Angles

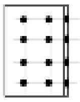


Beam Size	Bolt Rows	Un-notched		Fitting[4] Angle Cleats		Min. Support[5] Thickness		Tying[7]	
{ }[1]	n	Shear[2] Capacity kN	Critical[3] Design Check	Length ℓ mm	Mark	S275 mm	S355 mm	Capacity kN	Critical Check
305x165x54 {522 kN}	3	178	4 (iii)	220	CC3	3.2	2.7	261	12 (ii)
305x165x46 {438 kN}	3	151	4 (iii)	220	CC3	2.7	2.3	221	12 (ii)
305x165x40 {388 kN}	3	135	4 (iii)	220	CC3	2.4	2.0	198	12 (ii)
305x127x48 {596 kN}	3	203	4 (iii)	220	CC3	3.7	3.1	297	12 (ii)
305x127x42 {523 kN}	3	180	4 (iii)	220	CC3	3.3	2.7	264	12 (ii)
305x127x37 {460 kN}	3	160	4 (iii)	220	CC3	2.9	2.4	234	12 (ii)
305x102x33 {440 kN}	3	149	4 (iii)	220	CC3	2.7	2.3	218	12 (ii)
305x102x28 {395 kN}	3	135	4 (iii)	220	CC3	2.4	2.0	198	12 (ii)
305x102x25 {377 kN}	3	131	4 (iii)	220	CC3	2.4	2.0	191	12 (ii)
254x146x43 {398 kN}	2	91	4 (iii)	150	CC2	2.5	2.1	158	12 (ii)
254x146x37 {344 kN}	2	79	4 (iii)	150	CC2	2.2	1.8	139	12 (ii)
254x146x31 {321 kN}	2	76	4 (iii)	150	CC2	2.1	1.7	132	12 (ii)
254x102x28 {349 kN}	2	79	4 (iii)	150	CC2	2.2	1.8	139	12 (ii)
254x102x25 {329 kN}	2	76	4 (iii)	150	CC2	2.1	1.7	132	12 (ii)
254x102x22 {308 kN}	2	72	4 (iii)	150	CC2	2.0	1.6	125	12 (ii)

For guidance on the use of tables see Explanatory notes in Table H.6

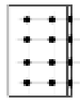
BEAM: S355
ANGLE CLEATS: S275
HOLLO-BOLTS: M20,8.8

Table H.16



DOUBLE ANGLE CLEATS, HOLLO-BOLTS

Double line of bolts
2 No.150x90x10mm Angles

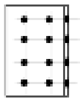


Beam Size { } (1)	Bolt Rows n	Un-notched: Shear(2) Capacity kN	Un-notched: Critical(3) Design Check	Fitting(4) Angle Cleats: Length ℓ mm	Fitting(4) Angle Cleats: Mark	Min. Support(5) Thickness: S275 mm	Min. Support(5) Thickness: S355 mm	Tying(7): Capacity kN	Tying(7): Critical Check
914x305x253	11	1460	9(i)	780	CD11	7.2	6.1	1050	11
{3290 kN}	10	1340	9(i)	710	CD10	7.3	6.1	953	11
	9	1210	9(i)	640	CD9	7.3	6.1	860	11
	8	1080	9(i)	570	CD8	7.3	6.1	766	11
914x305x224	11	1460	9(i)	780	CD11	7.2	6.1	1050	11
{3000 kN}	10	1340	9(i)	710	CD10	7.3	6.1	953	11
	9	1210	9(i)	640	CD9	7.3	6.1	860	11
	8	1080	9(i)	570	CD8	7.3	6.1	766	11
914x305x201	11	1460	9(i)	780	CD11	7.2	6.1	1050	11
{2820 kN}	10	1340	9(i)	710	CD10	7.3	6.1	953	11
	9	1210	9(i)	640	CD9	7.3	6.1	860	11
	8	1080	9(i)	570	CD8	7.3	6.1	766	11
838x292x226	10	1340	9(i)	710	CD10	7.3	6.1	953	11
{2840 kN}	9	1210	9(i)	640	CD9	7.3	6.1	860	11
	8	1080	9(i)	570	CD8	7.3	6.1	766	11
838x292x194	10	1340	9(i)	710	CD10	7.3	6.1	953	11
{2560 kN}	9	1210	9(i)	640	CD9	7.3	6.1	860	11
	8	1080	9(i)	570	CD8	7.3	6.1	766	11
838x292x176	10	1340	9(i)	710	CD10	7.3	6.1	953	11
{2420 kN}	9	1210	9(i)	640	CD9	7.3	6.1	860	11
	8	1080	9(i)	570	CD8	7.3	6.1	766	11
762x267x197	9	1210	9(i)	640	CD9	7.3	6.1	860	11
{2490 kN}	8	1080	9(i)	570	CD8	7.3	6.1	766	11
	7	947	9(i)	500	CD7	7.4	6.2	672	11
762x267x173	9	1210	9(i)	640	CD9	7.3	6.1	860	11
{2260 kN}	8	1080	9(i)	570	CD8	7.3	6.1	766	11
	7	947	9(i)	500	CD7	7.4	6.2	672	11
762x267x147	9	1210	9(i)	640	CD9	7.3	6.1	860	11
{2000 kN}	8	1080	9(i)	570	CD8	7.3	6.1	766	11
	7	947	9(i)	500	CD7	7.4	6.2	672	11
762x267x134	9	1210	9(i)	640	CD9	7.3	6.1	860	11
{1920 kN}	8	1080	9(i)	570	CD8	7.3	6.1	766	11
	7	947	9(i)	500	CD7	7.4	6.2	672	11

For guidance on the use of tables see Explanatory notes in Table H.6

BEAM: S355
ANGLE CLEATS: S275
HOLLO-BOLTS: M20,8.8

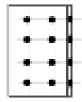
Table H.16 *Continued*



DOUBLE ANGLE CLEATS, HOLLO-BOLTS

Double line of bolts

2 No.150x90x10mm Angles

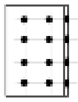


Beam Size { } (1)	Bolt Rows n	Un-notched: Shear(2) Capacity kN	Un-notched: Critical(3) Design Check	Fitting(4) Angle Cleats: Length ℓ mm	Fitting(4) Angle Cleats: Mark	Min. Support(5) Thickness: S275 mm	Min. Support(5) Thickness: S355 mm	Tying(7): Capacity kN	Tying(7): Critical Check
686x254x170	8	1080	9(i)	570	CD8	7.3	6.1	766	11
{2080 kN}	7	947	9(i)	500	CD7	7.4	6.2	672	11
	6	818	9(i)	430	CD6	7.4	6.2	580	11
686x254x152	8	1080	9(i)	570	CD8	7.3	6.1	766	11
{1880 kN}	7	947	9(i)	500	CD7	7.4	6.2	672	11
	6	818	9(i)	430	CD6	7.4	6.2	580	11
686x254x140	8	1080	9(i)	570	CD8	7.3	6.1	766	11
{1750 kN}	7	947	9(i)	500	CD7	7.4	6.2	672	11
	6	818	9(i)	430	CD6	7.4	6.2	580	11
686x254x125	8	1080	9(i)	570	CD8	7.3	6.1	766	11
{1640 kN}	7	947	9(i)	500	CD7	7.4	6.2	672	11
	6	818	9(i)	430	CD6	7.4	6.2	580	11
610x305x238	7	947	9(i)	500	CD7	7.4	6.2	672	11
{2420 kN}	6	818	9(i)	430	CD6	7.4	6.2	580	11
610x305x179	7	947	9(i)	500	CD7	7.4	6.2	672	11
{1810 kN}	6	818	9(i)	430	CD6	7.4	6.2	580	11
610x305x149	7	947	9(i)	500	CD7	7.4	6.2	672	11
{1500 kN}	6	818	9(i)	430	CD6	7.4	6.2	580	11
610x229x140	7	947	9(i)	500	CD7	7.4	6.2	672	11
{1670 kN}	6	818	9(i)	430	CD6	7.4	6.2	580	11
610x229x125	7	947	9(i)	500	CD7	7.4	6.2	672	11
{1510 kN}	6	818	9(i)	430	CD6	7.4	6.2	580	11
610x229x113	7	947	9(i)	500	CD7	7.4	6.2	672	11
{1400 kN}	6	818	9(i)	430	CD6	7.4	6.2	580	11
610x229x101	7	947	9(i)	500	CD7	7.4	6.2	672	11
{1350 kN}	6	818	9(i)	430	CD6	7.4	6.2	580	11

For guidance on the use of tables see Explanatory notes in Table H.6

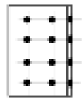
BEAM: S355
ANGLE CLEATS: S275
HOLLO-BOLTS: M20,8.8

Table H.16 *Continued*



DOUBLE ANGLE CLEATS, HOLLO-BOLTS

Double line of bolts
2 No.150x90x10mm Angles

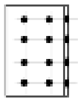


Beam Size	Bolt Rows	Un-notched		Fitting[4] Angle Cleats		Min. Support[5] Thickness		Tying[7]	
Beam Size { } [1]	n	Shear[2] Capacity kN	Critical[3] Design Check	Length ℓ mm	Mark	S275 mm	S355 mm	Capacity kN	Critical Check
533x210x122	6	818	9(i)	430	CD6	7.4	6.2	580	11
{1430 kN}	5	688	9(i)	360	CD5	7.5	6.3	486	11
533x210x109	6	818	9(i)	430	CD6	7.4	6.2	580	11
{1300 kN}	5	688	9(i)	360	CD5	7.5	6.3	486	11
533x210x101	6	818	9(i)	430	CD6	7.4	6.2	580	11
{1200 kN}	5	688	9(i)	360	CD5	7.5	6.3	486	11
533x210x92	6	818	9(i)	430	CD6	7.4	6.2	580	11
{1150 kN}	5	688	9(i)	360	CD5	7.5	6.3	486	11
533x210x82	6	818	9(i)	430	CD6	7.4	6.2	580	11
{1080 kN}	5	681	4 (iii)	360	CD5	7.4	6.2	486	11
457x191x98	5	688	9(i)	360	CD5	7.5	6.3	486	11
{1100 kN}	4	559	9(i)	290	CD4	7.6	6.4	393	11
457x191x89	5	688	9(i)	360	CD5	7.5	6.3	486	11
{1010 kN}	4	534	4 (iii)	290	CD4	7.3	6.1	393	11
457x191x82	5	688	9(i)	360	CD5	7.5	6.3	486	11
{970 kN}	4	504	4 (iii)	290	CD4	6.8	5.7	393	11
457x191x74	5	638	4 (iii)	360	CD5	6.9	5.8	486	11
{876 kN}	4	458	4 (iii)	290	CD4	6.2	5.2	393	11
457x191x67	5	603	4 (iii)	360	CD5	6.6	5.5	486	11
{821 kN}	4	432	4 (iii)	290	CD4	5.9	4.9	393	11

For guidance on the use of tables see Explanatory notes in Table H.6

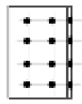
BEAM: S355
ANGLE CLEATS: S275
HOLLO-BOLTS: M20,8.8

Table H.16 *Continued*



DOUBLE ANGLE CLEATS, HOLLO-BOLTS

Double line of bolts
2 No.150x90x10mm Angles

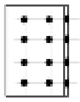


Beam Size { } (1)	Bolt Rows n	Un-notched Shear(2) Capacity kN	Un-notched Critical(3) Design Check	Fitting(4) Angle Cleats Length ℓ mm	Fitting(4) Angle Cleats Mark	Min. Support(5) Thickness S275 mm	Min. Support(5) Thickness S355 mm	Tying(7) Capacity kN	Tying(7) Critical Check
457x152x82	5	688	9(i)	360	CD5	7.5	6.3	486	11
{1010 kN}	4	534	4 (iii)	290	CD4	7.3	6.1	393	11
457x152x74	5	675	4(ii)	360	CD5	7.3	6.1	486	11
{918 kN}	4	488	4 (iii)	290	CD4	6.6	5.5	393	11
457x152x67	5	638	4 (iii)	360	CD5	6.9	5.8	486	11
{878 kN}	4	458	4 (iii)	290	CD4	6.2	5.2	393	11
457x152x60	5	574	4 (iii)	360	CD5	6.2	5.2	486	11
{784 kN}	4	412	4 (iii)	290	CD4	5.6	4.7	393	11
457x152x52	5	539	4 (iii)	360	CD5	5.9	4.9	486	11
{728 kN}	4	387	4 (iii)	290	CD4	5.3	4.4	393	11
406x178x74	4	483	4 (iii)	290	CD4	6.6	5.5	393	11
{835 kN}									
406x178x67	4	448	4 (iii)	290	CD4	6.1	5.1	393	11
{767 kN}									
406x178x60	4	402	4 (iii)	290	CD4	5.5	4.6	393	11
{684 kN}									
406x178x54	4	392	4 (iii)	290	CD4	5.3	4.5	393	11
{660 kN}									
406x140x46	4	346	4 (iii)	290	CD4	4.7	3.9	393	11
{584 kN}									
406x140x39	4	325	4 (iii)	290	CD4	4.4	3.7	393	11
{543 kN}									
356x171x67	3	300	4 (iii)	220	CD3	5.4	4.5	299	11
{704 kN}									
356x171x57	3	267	4 (iii)	220	CD3	4.8	4.0	299	11
{618 kN}									
356x171x51	3	244	4 (iii)	220	CD3	4.4	3.7	299	11
{560 kN}									
356x171x45	3	231	4 (iii)	220	CD3	4.2	3.5	299	11
{524 kN}									
356x127x39	3	217	4 (iii)	220	CD3	3.9	3.3	299	11
{497 kN}									
356x127x33	3	198	4 (iii)	220	CD3	3.6	3.0	299	11
{446 kN}									

For guidance on the use of tables see Explanatory notes in Table H.6

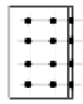
BEAM: S355
ANGLE CLEATS: S275
HOLLO-BOLTS: M20,8.8

Table H.16 *Continued*



DOUBLE ANGLE CLEATS, HOLLO-BOLTS

Double line of bolts
2 No.150x90x10mm Angles



Beam Size	Bolt Rows	Un-notched		Fitting[4] Angle Cleats		Min. Support[5] Thickness		Tying[7]	
{ } [1]	n	Shear[2] Capacity kN	Critical[3] Design Check	Length ℓ mm	Mark	S275 mm	S355 mm	Capacity kN	Critical Check
305x165x54 {522 kN}	3	260	4 (iii)	220	CD3	4.7	3.9	299	11
305x165x46 {438 kN}	3	221	4 (iii)	220	CD3	4.0	3.3	299	11
305x165x40 {388 kN}	3	198	4 (iii)	220	CD3	3.6	3.0	299	11
305x127x48 {596 kN}	3	296	4 (iii)	220	CD3	5.4	4.5	299	11
305x127x42 {523 kN}	3	263	4 (iii)	220	CD3	4.8	4.0	299	11
305x127x37 {460 kN}	3	234	4 (iii)	220	CD3	4.2	3.5	299	11
305x102x33 {440 kN}	3	217	4 (iii)	220	CD3	3.9	3.3	299	11
305x102x28 {395 kN}	3	198	4 (iii)	220	CD3	3.6	3.0	299	11
305x102x25 {377 kN}	3	191	4 (iii)	220	CD3	3.5	2.9	299	11
254x146x43 {398 kN}	2	129	4 (iii)	150	CD2	3.5	2.9	206	11
254x146x37 {344 kN}	2	113	4 (iii)	150	CD2	3.1	2.6	206	11
254x146x31 {321 kN}	2	107	4 (iii)	150	CD2	2.9	2.4	206	11
254x102x28 {349 kN}	2	113	4 (iii)	150	CD2	3.1	2.6	206	11
254x102x25 {329 kN}	2	107	4 (iii)	150	CD2	2.9	2.4	206	11
254x102x22 {308 kN}	2	102	4 (iii)	150	CD2	2.8	2.3	206	11

For guidance on the use of tables see Explanatory notes in Table H.6

Table H.17

Explanatory notes - FLEXIBLE END PLATES
Use of Capacity Tables

The following notes refer to the **suffix** numbers given at the top of the column descriptions for Tables H.20 to H.23.

The check numbers refer to those listed in Table H.19and described in Section 5.5 Design procedures.

The capacity tables are based on the standard details given in Table H.18.

All universal beam sections which are suitable for the minimum standard fitting size and the standard notch size are shown in the tables (i.e. if T + r > 50 mm, section is not included).

(1) SHEAR CAPACITY OF THE BEAM

The value given in { } is the shear capacity of the beam, given by $0.6p_yt_wD$

(2) SHEAR CAPACITY OF THE CONNECTION

This is the critical value of the design checks for the 'supported beam side' of the connection. i.e. the minimum capacity from CHECKS 2, 4, 8, 9(i) and 9(ii).

For connections with Ordinary or Flowdrill bolts, Connection Shear Capacities are given for Un-notched or Single notched and for Double notched beams.

For connections with Hollo-Bolts, connection shear capacities are tabulated for Un-notched beams only, as Hollo-Bolts are used only with RHS columns and Un-notched beams.

(3) CRITICAL DESIGN CHECK

The check which gives the critical value of shear capacity. See Table H.19 for the description of the checks.

(4) FITTINGS

The length and type of Standard End Plate fittings. See Tables H.1 and H.3 for details.

(5) MINIMUM SUPPORT THICKNESS

This is the minimum thickness of supporting column or beam element, that is needed to carry the given **SHEAR CAPACITY** (Un-notched or Single Notched) of the connection**.** It is derived from CHECK 10 and e_t has conservatively been taken as 90 mm.

For a symmetrical two sided connection, the minimum support thickness would be twice the tabulated value.

(6) MAXIMUM NOTCH LENGTH ($C + T_1$)

These are maximum lengths of notches for single and double notched beams that can be accommodated if the beam is to carry the tabulated corresponding **SHEAR CAPACITY** of the connection.

It is assumed that the beam is **fully restrained** against lateral torsional buckling, and the notched lengths are derived from CHECKS 5 and 6.

To provide a simple check for double notched beams, it has been assumed that the remaining web depth, 'y' (see Table H.18) is the same as the end plate length.

* Indicates that the condition from CHECK 6, i.e. $d_{c2} \le D/5$ is <u>not</u> satisfied.

For Hollo-Bolts, Tables H.22 to H.23, no values are tabulated for maximum notch lengths, since Hollo-Bolts are used only with RHS columns and Un-notched beams.

(7) TYING CAPACITY

This is the critical value of the design checks for the 'supported beam' side of the connection, i.e. the minimum capacity from CHECKS 11, 12, 13(i) and13(ii).

For Flowdrill connections, CHECK 13(i) will need to be carried out separately because the capacity is dependent on the thickness of the RHS.

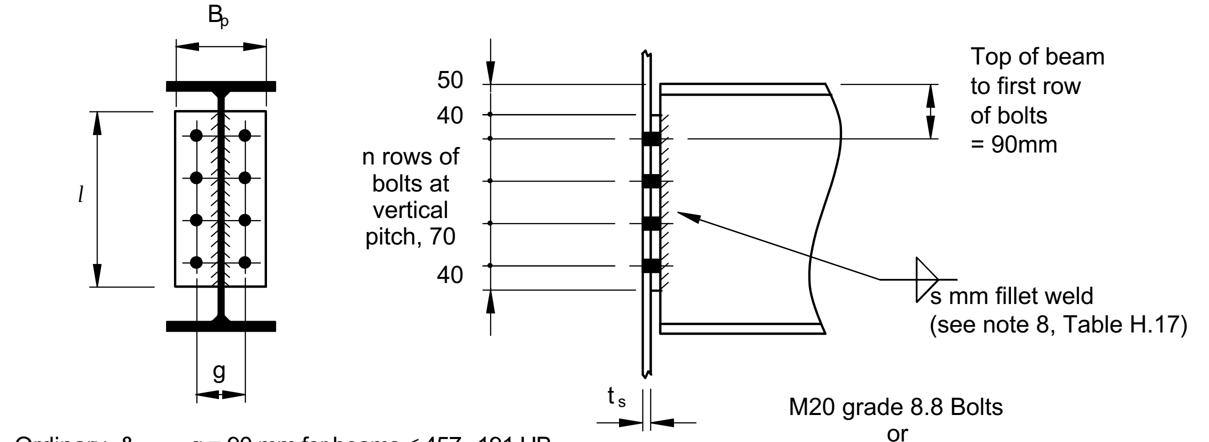
Separate checks will have to be carried out on the supporting members. (See CHECKS 14 and 15)

(8) WELD

The weld is either a 6, 8 or 10 mm fillet weld. The tabulated weld size has a capacity (CHECK 2) that is greater than the connection shear capacity.

Table H.18

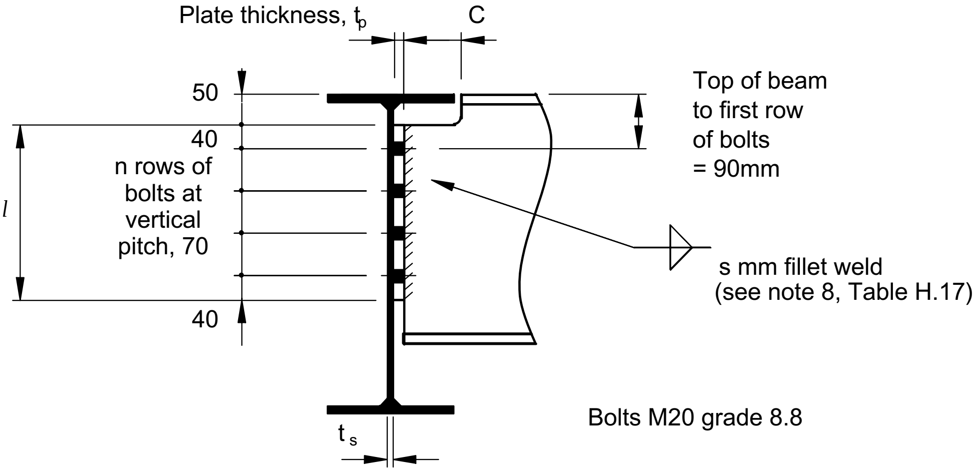
FLEXIBLE END PLATES Standard details used in Capacity Tables



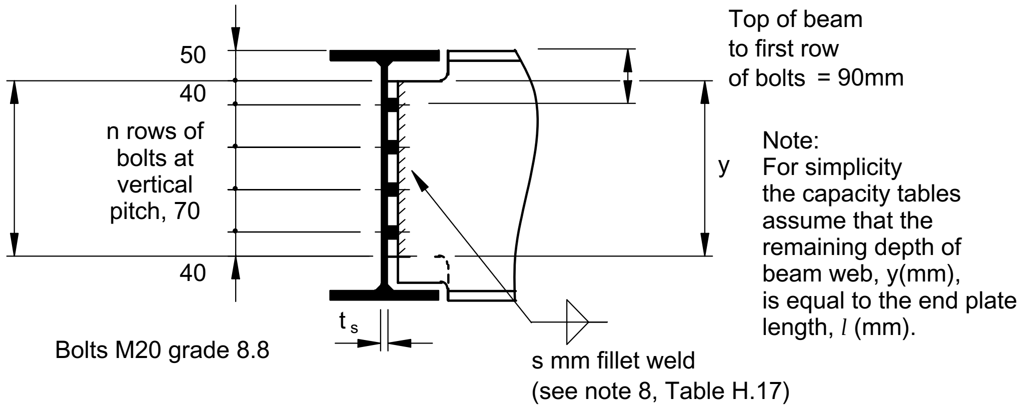
Ordinary & Flowdrill Bolts { g = 90 mm for beams ≤ 457 x 191 UB; g = 140 mm for beams > 457 x 191 UB

Hollo-Bolts { g = 90 mm for beams ≤ 457 x 191 UB; g = 120 mm for beams > 457 x 191 UB

UN-NOTCHED BEAM



SINGLE NOTCHED BEAM



DOUBLE NOTCHED BEAM

Note: See Tables H.1 and H.3 Standard Fittings - Marks and details for Flexible End Plates

Table H.19

FLEXIBLE END PLATES
Critical Check Description List

		Check Number	Description
SHEAR CAPACITY	Supported beam side	2	Capacity of fillet welds
		4	Shear capacity of supported beam at the connection
	Supporting member side	8	Shear capacity of bolt group connected to supporting member
		9(i)	Shear capacity of end plate
		9(ii)	Bearing capacity of end plate
STRUCTURAL INTEGRITY		11	Tension capacity of end plate
		12(i)	Tension capacity of supported beam web at the connection
		13(i)	Tension capacity of bolts in presence of extreme prying
		13(ii)	Tension capacity of weld

Note: This table only lists the critical checks.
For a full list of design checks and further information, see Section 5.5

Supplementary capacity tables for flexible end plates

Supplementary capacity tables for flexible end plates (pages 387-S to 396-S) are available on www.steelbiz.org as part of advisory desk note AD 291.

The supplementary tables give the double notch shear capacity with the corresponding critical design check, based on a standard double notch length.

BEAM: S275
END PLATE: S275
BOLTS: M20, 8.8

Table H.20

FLEXIBLE END PLATES, ORDINARY or FLOWDRILL BOLTS

200x10 or 150x8 mm End Plate

Beam Size	Bolt Rows	Un-notched or Single Notch		Double Notch		Fitting[4] End Plate					Fillet[8] weld	Min. Support[5] Thickness		Max. Notch Length[6] $c+t_p$		Tying[7]	
{ } [1]	n	Shear[2] Capacity kN	Critical[3] Design Check	Shear[2] Capacity kN	Critical[3] Design Check	Width mm	t_p mm	Length ℓ mm	Bolt[4] Gauge g mm	Mark[4]	s mm	S275 mm	S355 mm	Single mm	Double mm	Capacity kN	Critical Check
914x305x253	11	1930	4	1930	4	200	10	780	140	EB11	10	9.5	8.0	390	0	585	11
{2530 kN}	10	1760	4	1760	4	200	10	710	140	EB10	10	9.6	8.0	485	0	532	11
	9	1580	4	1580	4	200	10	640	140	EB9	10	9.6	8.0	538	0	480	11
	8	1410	4	1410	4	200	10	570	140	EB8	10	9.6	8.0	605	0	427	11
914x305x224	11	1770	4	1770	4	200	10	780	140	EB11	8	8.8	7.3	374	0	549	11
{2300 kN}	10	1620	4	1620	4	200	10	710	140	EB10	8	8.8	7.3	473	0	499	11
	9	1460	4	1460	4	200	10	640	140	EB9	8	8.8	7.4	525	0	450	11
	8	1300	4	1300	4	200	10	570	140	EB8	8	8.8	7.4	590	0	400	11
914x305x201	11	1690	4	1690	4	200	10	780	140	EB11	8	8.3	7.0	356	0	544	11
{2170 kN}	10	1530	4	1530	4	200	10	710	140	EB10	8	8.3	7.0	459	0	494	11
	9	1380	4	1380	4	200	10	640	140	EB9	8	8.4	7.0	509	0	445	11
	8	1230	4	1230	4	200	10	570	140	EB8	8	8.4	7.0	571	0	396	11
838x292x226	10	1640	4	1640	4	200	10	710	140	EB10	8	8.9	7.4	377	0	500	11
{2180 kN}	9	1470	4	1470	4	200	10	640	140	EB9	8	8.9	7.4	463	0	451	11
	8	1310	4	1310	4	200	10	570	140	EB8	8	8.9	7.5	520	0	401	11
838x292x194	10	1490	4	1490	4	200	10	710	140	EB10	8	8.1	6.8	355	0	492	11
{1960 kN}	9	1350	4	1350	4	200	10	640	140	EB9	8	8.1	6.8	446	0	443	11
	8	1200	4	1200	4	200	10	570	140	EB8	8	8.1	6.8	500	0	395	11
838x292x176	10	1420	4	1420	4	200	10	710	140	EB10	8	7.7	6.5	341	0	488	11
{1860 kN}	9	1280	4	1280	4	200	10	640	140	EB9	8	7.7	6.5	434	0	440	11
	8	1140	4	1140	4	200	10	570	140	EB8	8	7.8	6.5	487	0	391	11
762x267x197	9	1430	4	1430	4	200	10	640	140	EB9	8	8.6	7.2	335	0	448	11
{1910 kN}	8	1270	4	1270	4	200	10	570	140	EB8	8	8.6	7.2	419	0	399	11
	7	1120	4	1120	4	200	10	500	140	EB7	8	8.7	7.2	477	0	349	11
762x267x173	9	1310	4	1310	4	200	10	640	140	EB9	8	7.9	6.6	320	0	441	11
{1730 kN}	8	1170	4	1170	4	200	10	570	140	EB8	8	7.9	6.6	407	0	393	11
	7	1020	4	1020	4	200	10	500	140	EB7	8	7.9	6.6	464	0	344	11
762x267x147	9	1170	4	1170	4	200	10	640	140	EB9	8	7.1	5.9	303	0	434	11
{1530 kN}	8	1040	4	1040	4	200	10	570	140	EB8	8	7.1	5.9	394	0	386	11
	7	916	4	916	4	200	10	500	140	EB7	8	7.1	5.9	449	0	338	11
762x267x134	9	1140	4	1140	4	200	10	640	140	EB9	6	6.9	5.8	294	0	412	11
{1490 kN}	8	1020	4	1020	4	200	10	570	140	EB8	6	6.9	5.8	387	0	366	11
	7	891	4	891	4	200	10	500	140	EB7	6	6.9	5.8	441	0	321	11

For guidance on the use of tables see Explanatory notes in Table H.17

BEAM: S275
END PLATE: S275
BOLTS: M20, 8.8

Table H.20 *Continued*

FLEXIBLE END PLATES, ORDINARY or FLOWDRILL BOLTS
200x10 or 150x8 mm End Plate

Beam Size	Bolt Rows	Un-notched or Single Notch		Double Notch		Fitting[4] End Plate					Fillet[8] weld	Min. Support[5] Thickness		Max. Notch Length[6] $c+t_p$		Tying[7]	
Beam Size { } [1]	n	Shear[2] Capacity kN	Critical[3] Design Check	Shear[2] Capacity kN	Critical[3] Design Check	Width mm	t_p mm	Length ℓ mm	Bolt[4] Gauge g mm	Mark[4]	s mm	S275 mm	S355 mm	Single mm	Double mm	Capacity kN	Critical Check
686x254x170	8	1180	4	1180	4	200	10	570	140	EB8	8	8.0	6.7	305	0	394	11
{1600 kN}	7	1040	4	1040	4	200	10	500	140	EB7	8	8.1	6.7	385	0	345	11
	6	892	4	892	4	200	10	430	140	EB6	8	8.1	6.8	447	0	296	11
686x254x152	8	1080	4	1080	4	200	10	570	140	EB8	8	7.3	6.1	295	0	388	11
{1440 kN}	7	944	4	944	4	200	10	500	140	EB7	8	7.3	6.1	378	0	340	11
	6	812	4	812	4	200	10	430	140	EB6	8	7.4	6.2	440	0	292	11
686x254x140	8	1010	4	1010	4	200	10	570	140	EB8	6	6.9	5.7	288	0	368	11
{1350 kN}	7	887	4	887	4	200	10	500	140	EB7	6	6.9	5.8	372	0	322	11
	6	763	4	763	4	200	10	430	140	EB6	6	6.9	5.8	433	0	277	11
686x254x125	8	954	4	954	4	200	10	570	140	EB8	6	6.5	5.4	274	0	365	11
{1260 kN}	7	837	4	837	4	200	10	500	140	EB7	6	6.5	5.4	362	0	320	11
	6	720	4	720	4	200	10	430	140	EB6	6	6.5	5.5	421	0	275	11
610x305x238	7	1290	8	1290	8	200	10	500	140	EB7	10	10.0	8.4	335	51	379	11
{1860 kN}	6	1100	8	1100	8	200	10	430	140	EB6	10	10.0	8.4	395	47*	326	11
610x305x179	7	1010	4	1010	4	200	10	500	140	EB7	8	7.8	6.6	292	0	343	11
{1390 kN}	6	868	4	868	4	200	10	430	140	EB6	8	7.9	6.6	368	0	295	11
610x305x149	7	844	4	844	4	200	10	500	140	EB7	6	6.6	5.5	279	0	320	11
{1150 kN}	6	726	4	726	4	200	10	430	140	EB6	6	6.6	5.5	360	0	275	11
610x229x140	7	937	4	937	4	200	10	500	140	EB7	8	7.3	6.1	275	0	339	11
{1290 kN}	6	806	4	806	4	200	10	430	140	EB6	8	7.3	6.1	350	0	291	11
610x229x125	7	851	4	851	4	200	10	500	140	EB7	6	6.6	5.5	266	0	321	11
{1160 kN}	6	732	4	732	4	200	10	430	140	EB6	6	6.6	5.5	344	0	275	11
610x229x113	7	794	4	794	4	200	10	500	140	EB7	6	6.2	5.2	257	0	318	11
{1070 kN}	6	683	4	683	4	200	10	430	140	EB6	6	6.2	5.2	337	0	273	11
610x229x101	7	780	4	780	4	200	10	500	140	EB7	6	6.1	5.1	245	0	316	11
{1040 kN}	6	670	4	670	4	200	10	430	140	EB6	6	6.1	5.1	328	0	271	11

For guidance on the use of tables see Explanatory notes in Table H.17

BEAM: S275
END PLATE: S275
BOLTS: M20, 8.8

Table H.20 *Continued*

FLEXIBLE END PLATES, ORDINARY or FLOWDRILL BOLTS
200x10 or 150x8 mm End Plate

Beam Size	Bolt Rows	Un-notched or Single Notch		Double Notch		Fitting[4] End Plate					Fillet[8] weld	Min. Support[5] Thickness		Max. Notch Length[6] $c+t_p$		Tying[7]	
Beam Size { } [1]	n	Shear[2] Capacity kN	Critical[3] Design Check	Shear[2] Capacity kN	Critical[3] Design Check	Width mm	t_p mm	Length ℓ mm	Bolt[4] Gauge g mm	Mark[4]	s mm	S275 mm	S355 mm	Single mm	Double mm	Capacity kN	Critical Check
533x210x122	6	781	4	781	4	200	10	430	140	EB6	8	7.1	5.9	249	0	290	11
{1100 kN}	5	654	4	654	4	200	10	360	140	EB5	8	7.1	5.9	319	0	242	11
533x210x109	6	714	4	714	4	200	10	430	140	EB6	6	6.5	5.4	240	0	274	11
{995 kN}	5	598	4	598	4	200	10	360	140	EB5	6	6.5	5.4	313	0	229	11
533x210x101	6	665	4	665	4	200	10	430	140	EB6	6	6.0	5.0	236	0	272	11
{922 kN}	5	556	4	556	4	200	10	360	140	EB5	6	6.0	5.1	310	0	227	11
533x210x92	6	645	4	645	4	200	10	430	140	EB6	6	5.8	4.9	229	0	270	11
{888 kN}	5	540	4	540	4	200	10	360	140	EB5	6	5.9	4.9	304	0	226	11
533x210x82	6	613	4	613	4	200	10	430	140	EB6	6	5.6	4.6	217	0	269	11
{837 kN}	5	513	4	513	4	200	10	360	140	EB5	6	5.6	4.7	295	0	225	11
457x191x98	5	587	4	587	4	150	8	360	90	EA5	6	6.4	5.3	217	0	311	11
{847 kN}	4	473	4	473	4	150	8	290	90	EA4	6	6.4	5.4	285	0	249	11
457x191x89	5	541	4	541	4	150	8	360	90	EA5	6	5.9	4.9	211	0	304	11
{774 kN}	4	436	4	436	4	150	8	290	90	EA4	6	5.9	5.0	281	0	245	11
457x191x82	5	529	4	529	4	150	8	360	90	EA5	6	5.8	4.8	204	0	301	11
{751 kN}	4	426	4	426	4	150	8	290	90	EA4	6	5.8	4.8	275	0	241	11
457x191x74	5	481	4	481	4	150	8	360	90	EA5	6	5.2	4.4	199	0	295	11
{679 kN}	4	388	4	388	4	150	8	290	90	EA4	6	5.3	4.4	272	0	237	11
457x191x67	5	454	4	454	4	150	8	360	90	EA5	6	4.9	4.1	191	0	292	11
{636 kN}	4	366	4	366	4	150	8	290	90	EA4	6	5.0	4.2	266	0	234	11

For guidance on the use of tables see Explanatory notes in Table H.17

BEAM: S275
END PLATE: S275
BOLTS: M20, 8.8

Table H.20 *Continued*

FLEXIBLE END PLATES, ORDINARY or FLOWDRILL BOLTS

200x10 or 150x8 mm End Plate

Beam Size	Bolt Rows	Un-notched or Single Notch		Double Notch		Fitting[(4)] End Plate					Fillet[(8)] weld	Min. Support[(5)] Thickness		Max. Notch Length[(6)] c+t_p		Tying[(7)]	
	n	Shear[(2)] Capacity	Critical[(3)] Design Check	Shear[(2)] Capacity	Critical[(3)] Design Check	Width	t_p	Length ℓ	Bolt[(4)] Gauge g	Mark[(4)]	s	S275	S355	Single	Double	Capacity	Critical Check
{ } [(1)]		kN		kN		mm	mm	mm	mm		mm	mm	mm	mm	mm	kN	
457x152x82	5	541	4	541	4	150	8	360	90	EA5	6	5.9	4.9	209	0	304	11
{778 kN}	4	436	4	436	4	150	8	290	90	EA4	6	5.9	5.0	276	0	245	11
457x152x74	5	495	4	495	4	150	8	360	90	EA5	6	5.4	4.5	203	0	299	11
{705 kN}	4	398	4	398	4	150	8	290	90	EA4	6	5.4	4.5	272	0	240	11
457x152x67	5	481	4	481	4	150	8	360	90	EA5	6	5.2	4.4	195	0	295	11
{680 kN}	4	388	4	388	4	150	8	290	90	EA4	6	5.3	4.4	266	0	237	11
457x152x60	5	433	4	433	4	150	8	360	90	EA5	6	4.7	3.9	189	0	289	11
{608 kN}	4	349	4	349	4	150	8	290	90	EA4	6	4.7	4.0	262	0	232	11
457x152x52	5	406	4	406	4	150	8	360	90	EA5	6	4.4	3.7	178	0	286	11
{564 kN}	4	327	4	327	4	150	8	290	90	EA4	6	4.4	3.7	252	0	230	11
406x178x74	4	409	4	409	4	150	8	290	90	EA4	6	5.6	4.6	218	0	239	11
{647 kN}																	
406x178x67	4	379	4	379	4	150	8	290	90	EA4	6	5.1	4.3	214	0	236	11
{594 kN}																	
406x178x60	4	340	4	340	4	150	8	290	90	EA4	6	4.6	3.9	211	0	231	11
{530 kN}																	
406x178x54	4	332	4	332	4	150	8	290	90	EA4	6	4.5	3.8	204	0	230	11
{512 kN}																	
406x140x46	4	293	4	293	4	150	8	290	90	EA4	6	4.0	3.3	202	0	226	11
{452 kN}																	
406x140x39	4	276	4	276	4	150	8	290	90	EA4	6	3.7	3.1	191	0	224	11
{420 kN}																	
356x171x67	3	297	4	297	4	150	8	220	90	EA3	6	5.4	4.5	217	0	179	11
{546 kN}																	
356x171x57	3	265	4	265	4	150	8	220	90	EA3	6	4.8	4.0	210	0	175	11
{478 kN}																	
356x171x51	3	242	4	242	4	150	8	220	90	EA3	6	4.4	3.7	206	0	173	11
{433 kN}																	
356x171x45	3	229	4	229	4	150	8	220	90	EA3	6	4.1	3.5	199	0	171	11
{406 kN}																	
356x127x39	3	216	4	216	4	150	8	220	90	EA3	6	3.9	3.3	196	0	170	11
{385 kN}																	
356x127x33	3	196	4	196	4	150	8	220	90	EA3	6	3.6	3.0	188	0	168	11
{346 kN}																	

For guidance on the use of tables see Explanatory notes in Table H.17

BEAM: S275
END PLATE: S275
BOLTS: M20, 8.8

Table H.20 *Continued*

FLEXIBLE END PLATES, ORDINARY or FLOWDRILL BOLTS

200x10 or 150x8 mm End Plate

Beam Size { } [(1)]	Bolt Rows n	Un-notched or Single Notch Shear[(2)] Capacity kN	Un-notched or Single Notch Critical[(3)] Design Check	Double Notch Shear[(2)] Capacity kN	Double Notch Critical[(3)] Design Check	Fitting[(4)] End Plate Width mm	Fitting[(4)] End Plate t_p mm	Fitting[(4)] End Plate Length ℓ mm	Fitting[(4)] End Plate Bolt[(4)] Gauge g mm	Fitting[(4)] End Plate Mark[(4)]	Fillet[(8)] weld s mm	Min. Support[(5)] Thickness S275 mm	Min. Support[(5)] Thickness S355 mm	Max. Notch Length[(6)] $c+t_p$ Single mm	Max. Notch Length[(6)] $c+t_p$ Double mm	Tying[(7)] Capacity kN	Tying[(7)] Critical Check
305x165x54 {405 kN}	3	258	4	258	4	150	8	220	90	EA3	6	4.7	3.9	149	0	175	11
305x165x46 {339 kN}	3	219	4	219	4	150	8	220	90	EA3	6	4.0	3.3	143	0	170	11
305x165x40 {300 kN}	3	196	4	196	4	150	8	220	90	EA3	6	3.6	3.0	137	0	168	11
305x127x48 {462 kN}	3	294	4	294	4	150	8	220	90	EA3	6	5.3	4.5	141	0	179	11
305x127x42 {406 kN}	3	261	4	261	4	150	8	220	90	EA3	6	4.7	4.0	135	0	175	11
305x127x37 {357 kN}	3	232	4	232	4	150	8	220	90	EA3	6	4.2	3.5	131	0	172	11
305x102x33 {341 kN}	3	216	4	216	4	150	8	220	90	EA3	6	3.9	3.3	143	0	170	11
305x102x28 {306 kN}	3	196	4	196	4	150	8	220	90	EA3	6	3.6	3.0	134	0	168	11
305x102x25 {292 kN}	3	189	4	189	4	150	8	220	90	EA3	6	3.4	2.9	125	0	167	11
254x146x43 {308 kN}	2	160	4	160	4	150	8	150	90	EA2	6	4.4	3.6	146	0	116	11
254x146x37 {266 kN}	2	140	4	140	4	150	8	150	90	EA2	6	3.8	3.2	142	0	114	11
254x146x31 {249 kN}	2	134	4	134	4	150	8	150	90	EA2	6	3.6	3.0	134	0	113	11
254x102x28 {271 kN}	2	140	4	140	4	150	8	150	90	EA2	6	3.8	3.2	140	0	114	11
254x102x25 {255 kN}	2	134	4	134	4	150	8	150	90	EA2	6	3.6	3.0	134	0	113	11
254x102x22 {239 kN}	2	127	4	127	4	150	8	150	90	EA2	6	3.5	2.9	128	0	113	11

For guidance on the use of tables see Explanatory notes in Table H.17

BEAM: S355
END PLATE: S275
BOLTS: M20, 8.8

Table H.21

FLEXIBLE END PLATES, ORDINARY or FLOWDRILL BOLTS
200x10 or 150x8 mm End Plate

Beam Size { } [1]	Bolt Rows n	Un-notched or Single Notch: Shear[2] Capacity kN	Un-notched or Single Notch: Critical[3] Design Check	Double Notch: Shear[2] Capacity kN	Double Notch: Critical[3] Design Check	Fitting[4] End Plate: Width mm	t_p mm	Length ℓ mm	Bolt[4] Gauge g mm	Mark[4]	Fillet[8] welds mm	Min. Support[5] Thickness: S275 mm	S355 mm	Max. Notch Length[6] $c+t_p$: Single mm	Double mm	Tying[7]: Capacity kN	Critical Check
914x305x253	11	2020	8	2020	8	200	10	780	140	EB11	10	10.0	8.4	550	267	585	11
{3290 kN}	10	1840	8	1840	8	200	10	710	140	EB10	10	10.0	8.4	604	244	532	11
	9	1650	8	1650	8	200	10	640	140	EB9	10	10.0	8.4	672	221*	480	11
	8	1470	8	1470	8	200	10	570	140	EB8	10	10.0	8.4	685	198*	427	11
914x305x224	11	2020	8	2020	8	200	10	780	140	EB11	10	10.0	8.4	493	200	575	11
{3000 kN}	10	1840	8	1840	8	200	10	710	140	EB10	10	10.0	8.4	542	183	523	11
	9	1650	8	1650	8	200	10	640	140	EB9	10	10.0	8.4	543	166*	471	11
	8	1470	8	1470	8	200	10	570	140	EB8	10	10.0	8.4	543	149*	419	11
914x305x201	11	2020	8	2020	8	200	10	780	140	EB11	10	10.0	8.4	443	153	570	11
{2820 kN}	10	1840	8	1840	8	200	10	710	140	EB10	10	10.0	8.4	474	140	518	11
	9	1650	8	1650	8	200	10	640	140	EB9	10	10.0	8.4	474	128*	467	11
	8	1470	8	1470	8	200	10	570	140	EB8	10	10.0	8.4	474	115*	415	11
838x292x226	10	1840	8	1840	8	200	10	710	140	EB10	10	10.0	8.4	484	193	525	11
{2840 kN}	9	1650	8	1650	8	200	10	640	140	EB9	10	10.0	8.4	537	175	473	11
	8	1470	8	1470	8	200	10	570	140	EB8	10	10.0	8.4	605	157*	421	11
838x292x194	10	1840	8	1840	8	200	10	710	140	EB10	10	10.0	8.4	412	114	516	11
{2560 kN}	9	1650	8	1650	8	200	10	640	140	EB9	10	10.0	8.4	472	104	465	11
	8	1470	8	1470	8	200	10	570	140	EB8	10	10.0	8.4	504	94*	413	11
838x292x176	10	1840	8	1840	8	200	10	710	140	EB10	10	10.0	8.4	349	41	512	11
{2420 kN}	9	1650	8	1650	8	200	10	640	140	EB9	10	10.0	8.4	438	41	461	11
	8	1470	8	1470	8	200	10	570	140	EB8	10	10.0	8.4	443	40*	410	11
762x267x197	9	1650	8	1650	8	200	10	640	140	EB9	10	10.0	8.4	419	153	470	11
{2490 kN}	8	1470	8	1470	8	200	10	570	140	EB8	10	10.0	8.4	472	137	418	11
	7	1290	8	1290	8	200	10	500	140	EB7	10	10.0	8.4	539	122*	366	11
762x267x173	9	1650	8	1650	8	200	10	640	140	EB9	10	10.0	8.4	348	74	462	11
{2260 kN}	8	1470	8	1470	8	200	10	570	140	EB8	10	10.0	8.4	421	68	411	11
	7	1290	8	1290	8	200	10	500	140	EB7	10	10.0	8.4	481	62*	361	11
762x267x147	9	1530	4	1530	4	200	10	640	140	EB9	8	9.2	7.7	303	0	434	11
{2000 kN}	8	1360	4	1360	4	200	10	570	140	EB8	8	9.2	7.7	394	0	386	11
	7	1190	4	1190	4	200	10	500	140	EB7	8	9.3	7.7	416	0	338	11
762x267x134	9	1470	4	1470	4	200	10	640	140	EB9	8	8.9	7.4	294	0	430	11
{1920 kN}	8	1310	4	1310	4	200	10	570	140	EB8	8	8.9	7.5	348	0	383	11
	7	1150	4	1150	4	200	10	500	140	EB7	8	8.9	7.5	348	0	335	11

For guidance on the use of tables see Explanatory notes in Table H.17

BEAM: S355
END PLATE: S275
BOLTS: M20, 8.8

Table H.21 *Continued*

FLEXIBLE END PLATES, ORDINARY or FLOWDRILL BOLTS

200x10 or 150x8 mm End Plate

Beam Size { } (1)	Bolt Rows n	Un-notched or Single Notch: Shear(2) Capacity kN	Un-notched or Single Notch: Critical(3) Design Check	Double Notch: Shear(2) Capacity kN	Double Notch: Critical(3) Design Check	Fitting(4) End Plate: Width mm	Fitting(4) End Plate: t_p mm	Fitting(4) End Plate: Length ℓ mm	Fitting(4) End Plate: Bolt(4) Gauge g mm	Fitting(4) End Plate: Mark(4)	Fillet(8) weld s mm	Min. Support(5) Thickness: S275 mm	Min. Support(5) Thickness: S355 mm	Max. Notch Length(6) $c+t_p$: Single mm	Max. Notch Length(6) $c+t_p$: Double mm	Tying(7): Capacity kN	Tying(7): Critical Check
686x254x170	8	1470	8	1470	8	200	10	570	140	EB8	10	10.0	8.4	343	82	412	11
{2080 kN}	7	1290	8	1290	8	200	10	500	140	EB7	10	10.0	8.4	404	74*	361	11
	6	1100	8	1100	8	200	10	430	140	EB6	10	10.0	8.4	471	66*	310	11
686x254x152	8	1400	4	1400	4	200	10	570	140	EB8	10	9.5	8.0	295	0	406	11
{1880 kN}	7	1230	4	1230	4	200	10	500	140	EB7	10	9.5	8.0	378	0	356	11
	6	1060	4	1060	4	200	10	430	140	EB6	10	9.6	8.0	440	0	306	11
686x254x140	8	1320	4	1320	4	200	10	570	140	EB8	8	8.9	7.5	288	0	384	11
{1750 kN}	7	1160	4	1160	4	200	10	500	140	EB7	8	9.0	7.5	372	0	337	11
	6	993	4	993	4	200	10	430	140	EB6	8	9.0	7.5	433	0	289	11
686x254x125	8	1240	4	1240	4	200	10	570	140	EB8	8	8.4	7.1	274	0	381	11
{1640 kN}	7	1090	4	1090	4	200	10	500	140	EB7	8	8.5	7.1	362	0	334	11
	6	937	4	937	4	200	10	430	140	EB6	8	8.5	7.1	393	0	287	11
610x305x238	7	1290	8	1290	8	200	10	500	140	EB7	10	10.0	8.4	440	204	379	11
{2420 kN}	6	1100	8	1100	8	200	10	430	140	EB6	10	10.0	8.4	514	177*	326	11
610x305x179	7	1290	8	1290	8	200	10	500	140	EB7	10	10.0	8.4	308	48	360	11
{1810 kN}	6	1100	8	1100	8	200	10	430	140	EB6	10	10.0	8.4	377	44*	309	11
610x305x149	7	1100	4	1100	4	200	10	500	140	EB7	8	8.5	7.1	279	0	335	11
{1500 kN}	6	945	4	945	4	200	10	430	140	EB6	8	8.6	7.2	360	0	287	11
610x229x140	7	1220	4	1220	4	200	10	500	140	EB7	10	9.5	7.9	275	0	355	11
{1670 kN}	6	1050	4	1050	4	200	10	430	140	EB6	10	9.5	8.0	350	0	305	11
610x229x125	7	1110	4	1110	4	200	10	500	140	EB7	8	8.6	7.2	266	0	335	11
{1510 kN}	6	953	4	953	4	200	10	430	140	EB6	8	8.6	7.2	344	0	288	11
610x229x113	7	1030	4	1030	4	200	10	500	140	EB7	8	8.0	6.7	257	0	332	11
{1400 kN}	6	889	4	889	4	200	10	430	140	EB6	8	8.1	6.7	337	0	285	11
610x229x101	7	1010	4	1010	4	200	10	500	140	EB7	8	7.8	6.5	245	0	330	11
{1350 kN}	6	866	4	866	4	200	10	430	140	EB6	8	7.8	6.6	328	0	283	11

For guidance on the use of tables see Explanatory notes in Table H.17

BEAM: S355
END PLATE: S275
BOLTS: M20, 8.8

Table H.21 *Continued*

FLEXIBLE END PLATES, ORDINARY or FLOWDRILL BOLTS
200x10 or 150x8 mm End Plate

Beam Size { } (1)	Bolt Rows n	Un-notched or Single Notch		Double Notch		Fitting(4) End Plate					Fillet(8) weld s	Min. Support(5) Thickness		Max. Notch Length(6) $c+t_p$		Tying(7)	
		Shear(2) Capacity kN	Critical(3) Design Check	Shear(2) Capacity kN	Critical(3) Design Check	Width mm	t_p mm	Length ℓ mm	Bolt(4) Gauge g mm	Mark(4)	mm	S275 mm	S355 mm	Single mm	Double mm	Capacity kN	Critical Check
533x210x122	6	1020	4	1020	4	200	10	430	140	EB6	8	9.2	7.7	249	0	290	11
{1430 kN}	5	852	4	852	4	200	10	360	140	EB5	10	9.3	7.7	319	0	254	11
533x210x109	6	929	4	929	4	200	10	430	140	EB6	8	8.4	7.0	240	0	287	11
{1300 kN}	5	778	4	778	4	200	10	360	140	EB5	8	8.5	7.1	313	0	239	11
533x210x101	6	865	4	865	4	200	10	430	140	EB6	8	7.8	6.6	236	0	284	11
{1200 kN}	5	724	4	724	4	200	10	360	140	EB5	8	7.9	6.6	310	0	237	11
533x210x92	6	833	4	833	4	200	10	430	140	EB6	8	7.5	6.3	229	0	282	11
{1150 kN}	5	697	4	697	4	200	10	360	140	EB5	8	7.6	6.3	304	0	236	11
533x210x82	6	791	4	791	4	200	10	430	140	EB6	8	7.2	6.0	217	0	280	11
{1080 kN}	5	663	4	663	4	200	10	360	140	EB5	8	7.2	6.0	295	0	234	11
457x191x98	5	736	9(ii)	736	9(ii)	150	8	360	90	EA5	8	8.0	6.7	239	47	341	11
{1100 kN}	4	589	9(ii)	589	9(ii)	150	8	290	90	EA4	8	8.0	6.7	298	41*	274	11
457x191x89	5	704	4	704	4	150	8	360	90	EA5	8	7.7	6.4	211	0	334	11
{1010 kN}	4	567	4	567	4	150	8	290	90	EA4	8	7.7	6.4	281	0	268	11
457x191x82	5	683	4	683	4	150	8	360	90	EA5	8	7.4	6.2	204	0	329	11
{970 kN}	4	550	4	550	4	150	8	290	90	EA4	8	7.5	6.3	275	0	264	11
457x191x74	5	621	4	621	4	150	8	360	90	EA5	6	6.8	5.6	199	0	295	11
{876 kN}	4	500	4	500	4	150	8	290	90	EA4	6	6.8	5.7	272	0	237	11
457x191x67	5	587	4	587	4	150	8	360	90	EA5	6	6.4	5.3	191	0	292	11
{821 kN}	4	473	4	473	4	150	8	290	90	EA4	6	6.4	5.4	266	0	234	11

For guidance on the use of tables see Explanatory notes in Table H.17

BEAM: S355
END PLATE: S275
BOLTS: M20, 8.8

Table H.21 *Continued*

FLEXIBLE END PLATES, ORDINARY or FLOWDRILL BOLTS

200x10 or 150x8 mm End Plate

Beam Size	Bolt Rows	Un-notched or Single Notch		Double Notch		Fitting[4] End Plate					Fillet[8] weld	Min. Support[5] Thickness		Max. Notch Length[6] c+t_p		Tying[7]	
	n	Shear[2] Capacity	Critical[3] Design Check	Shear[2] Capacity	Critical[3] Design Check	Width	t_p	Length ℓ	Bolt[4] Gauge g	Mark[4]	s	S275	S355	Single	Double	Capacity	Critical Check
{ } [1]		kN		kN		mm	mm	mm	mm		mm	mm	mm	mm	mm	kN	
457x152x82	5	704	4	704	4	150	8	360	90	EA5	8	7.7	6.4	209	0	334	11
{1010 kN}	4	567	4	567	4	150	8	290	90	EA4	8	7.7	6.4	276	0	268	11
457x152x74	5	644	4	644	4	150	8	360	90	EA5	8	7.0	5.9	203	0	327	11
{918 kN}	4	519	4	519	4	150	8	290	90	EA4	8	7.0	5.9	272	0	262	11
457x152x67	5	621	4	621	4	150	8	360	90	EA5	6	6.8	5.6	195	0	295	11
{878 kN}	4	500	4	500	4	150	8	290	90	EA4	6	6.8	5.7	266	0	237	11
457x152x60	5	559	4	559	4	150	8	360	90	EA5	6	6.1	5.1	189	0	289	11
{784 kN}	4	450	4	450	4	150	8	290	90	EA4	6	6.1	5.1	262	0	232	11
457x152x52	5	524	4	524	4	150	8	360	90	EA5	6	5.7	4.8	178	0	286	11
{728 kN}	4	423	4	423	4	150	8	290	90	EA4	6	5.7	4.8	247	0	230	11
406x178x74	4	528	4	528	4	150	8	290	90	EA4	8	7.2	6.0	218	0	262	11
{835 kN}																	
406x178x67	4	489	4	489	4	150	8	290	90	EA4	6	6.6	5.6	214	0	236	11
{767 kN}																	
406x178x60	4	439	4	439	4	150	8	290	90	EA4	6	6.0	5.0	211	0	231	11
{684 kN}																	
406x178x54	4	428	4	428	4	150	8	290	90	EA4	6	5.8	4.9	204	0	230	11
{660 kN}																	
406x140x46	4	378	4	378	4	150	8	290	90	EA4	6	5.1	4.3	202	0	226	11
{584 kN}																	
406x140x39	4	356	4	356	4	150	8	290	90	EA4	6	4.8	4.0	190	0	224	11
{543 kN}																	
356x171x67	3	384	4	384	4	150	8	220	90	EA3	6	7.0	5.8	217	0	179	11
{704 kN}																	
356x171x57	3	342	4	342	4	150	8	220	90	EA3	6	6.2	5.2	210	0	175	11
{618 kN}																	
356x171x51	3	312	4	312	4	150	8	220	90	EA3	6	5.7	4.7	206	0	173	11
{560 kN}																	
356x171x45	3	295	4	295	4	150	8	220	90	EA3	6	5.3	4.5	199	0	171	11
{524 kN}																	
356x127x39	3	278	4	278	4	150	8	220	90	EA3	6	5.0	4.2	196	0	170	11
{497 kN}																	
356x127x33	3	253	4	253	4	150	8	220	90	EA3	6	4.6	3.8	188	0	168	11
{446 kN}																	

For guidance on the use of tables see Explanatory notes in Table H.17

BEAM: S355
END PLATE: S275
BOLTS: M20, 8.8

Table H.21 *Continued*

FLEXIBLE END PLATES, ORDINARY or FLOWDRILL BOLTS

200x10 or 150x8 mm End Plate

Beam Size	Bolt Rows	Un-notched or Single Notch		Double Notch		Fitting[4] End Plate					Fillet[8] weld	Min. Support[5] Thickness		Max. Notch Length[6] $c+t_p$		Tying[7]	
{ } [1]	n	Shear[2] Capacity kN	Critical[3] Design Check	Shear[2] Capacity kN	Critical[3] Design Check	Width mm	t_p mm	Length ℓ mm	Bolt[4] Gauge g mm	Mark[4]	s mm	S275 mm	S355 mm	Single mm	Double mm	Capacity kN	Critical Check
305x165x54 {522 kN}	3	333	4	333	4	150	8	220	90	EA3	6	6.0	5.0	149	0	175	11
305x165x46 {438 kN}	3	283	4	283	4	150	8	220	90	EA3	6	5.1	4.3	143	0	170	11
305x165x40 {388 kN}	3	253	4	253	4	150	8	220	90	EA3	6	4.6	3.8	137	0	168	11
305x127x48 {596 kN}	3	380	4	380	4	150	8	220	90	EA3	6	6.9	5.8	141	0	179	11
305x127x42 {523 kN}	3	337	4	337	4	150	8	220	90	EA3	6	6.1	5.1	135	0	175	11
305x127x37 {460 kN}	3	299	4	299	4	150	8	220	90	EA3	6	5.4	4.5	131	0	172	11
305x102x33 {440 kN}	3	278	4	278	4	150	8	220	90	EA3	6	5.0	4.2	143	0	170	11
305x102x28 {395 kN}	3	253	4	253	4	150	8	220	90	EA3	6	4.6	3.8	134	0	168	11
305x102x25 {377 kN}	3	245	4	245	4	150	8	220	90	EA3	6	4.4	3.7	125	0	167	11
254x146x43 {398 kN}	2	207	4	207	4	150	8	150	90	EA2	6	5.6	4.7	146	0	116	11
254x146x37 {344 kN}	2	181	4	181	4	150	8	150	90	EA2	6	4.9	4.1	142	0	114	11
254x146x31 {321 kN}	2	173	4	173	4	150	8	150	90	EA2	6	4.7	3.9	134	0	113	11
254x102x28 {349 kN}	2	181	4	181	4	150	8	150	90	EA2	6	4.9	4.1	140	0	114	11
254x102x25 {329 kN}	2	173	4	173	4	150	8	150	90	EA2	6	4.7	3.9	134	0	113	11
254x102x22 {308 kN}	2	164	4	164	4	150	8	150	90	EA2	6	4.5	3.7	128	0	113	11

For guidance on the use of tables see Explanatory notes in Table H.17

BEAM: S275
END PLATE: S275
HOLLO-BOLTS: M20, 8.8

Table H.22

FLEXIBLE END PLATES, HOLLO-BOLTS

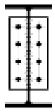
200x10 or 180x8 mm End Plate

Beam Size	Bolt Rows	Un-notched		Fitting[4] End Plate					Fillet[8] weld	Min. Support[5] Thickness		Tying[7]	
{ } [1]	n	Shear[2] Capacity kN	Critical[3] Design Check	Width mm	t_p mm	Length ℓ mm	Bolt[4] Gauge g mm	Mark[4]	s mm	S275 mm	S355 mm	Capacity kN	Critical Check
914x305x253	11	1460	9(i)	200	10	780	120	ED11	8	7.6	6.5	990	11
{2530 kN}	10	1340	9(i)	200	10	710	120	ED10	8	7.6	6.4	901	11
	9	1210	9(i)	200	10	640	120	ED9	8	7.6	6.4	812	11
	8	1080	9(i)	200	10	570	120	ED8	8	7.5	6.4	723	11
914x305x224	11	1460	9(i)	200	10	780	120	ED11	8	7.6	6.5	964	11
{2300 kN}	10	1340	9(i)	200	10	710	120	ED10	8	7.6	6.4	877	11
	9	1210	9(i)	200	10	640	120	ED9	8	7.6	6.4	791	11
	8	1080	9(i)	200	10	570	120	ED8	8	7.5	6.4	704	11
914x305x201	11	1460	9(i)	200	10	780	120	ED11	8	7.6	6.5	949	11
{2170 kN}	10	1340	9(i)	200	10	710	120	ED10	8	7.6	6.4	864	11
	9	1210	9(i)	200	10	640	120	ED9	8	7.6	6.4	779	11
	8	1080	9(i)	200	10	570	120	ED8	8	7.5	6.4	694	11
838x292x226	10	1340	9(i)	200	10	710	120	ED10	8	7.6	6.4	880	11
{2180 kN}	9	1210	9(i)	200	10	640	120	ED9	8	7.6	6.4	794	11
	8	1080	9(i)	200	10	570	120	ED8	8	7.5	6.4	707	11
838x292x194	10	1340	9(i)	200	10	710	120	ED10	8	7.6	6.4	858	11
{1960 kN}	9	1210	9(i)	200	10	640	120	ED9	8	7.6	6.4	773	11
	8	1080	9(i)	200	10	570	120	ED8	8	7.5	6.4	689	11
838x292x176	10	1340	9(i)	200	10	710	120	ED10	8	7.6	6.4	847	11
{1860 kN}	9	1210	9(i)	200	10	640	120	ED9	8	7.6	6.4	763	11
	8	1080	9(i)	200	10	570	120	ED8	8	7.5	6.4	680	11
762x267x197	9	1210	9(i)	200	10	640	120	ED9	8	7.6	6.4	786	11
{1910 kN}	8	1080	9(i)	200	10	570	120	ED8	8	7.5	6.4	700	11
	7	947	9(i)	200	10	500	120	ED7	8	7.5	6.3	614	11
762x267x173	9	1210	9(i)	200	10	640	120	ED9	8	7.6	6.4	768	11
{1730 kN}	8	1080	9(i)	200	10	570	120	ED8	8	7.5	6.4	684	11
	7	947	9(i)	200	10	500	120	ED7	8	7.5	6.3	600	11
762x267x147	9	1170	4	200	10	640	120	ED9	8	7.4	6.2	747	11
{1530 kN}	8	1040	4	200	10	570	120	ED8	8	7.3	6.2	665	11
	7	916	4	200	10	500	120	ED7	8	7.2	6.1	584	11
762x267x134	9	1140	4	200	10	640	120	ED9	6	7.2	6.0	688	11
{1490 kN}	8	1020	4	200	10	570	120	ED8	6	7.1	6.0	613	11
	7	891	4	200	10	500	120	ED7	6	7.0	5.9	538	11

For guidance on the use of tables see Explanatory notes in Table H.17

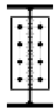
BEAM: S275
END PLATE: S275
HOLLO-BOLTS: M20, 8.8

Table H.22 *Continued*



FLEXIBLE END PLATES, HOLLO-BOLTS

200x10 or 180x8 mm End Plate

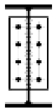


Beam Size	Bolt Rows	Un-notched		Fitting[4] End Plate					Fillet[8] weld	Min. Support[5] Thickness		Tying[7]	
{ } [1]	n	Shear[2] Capacity kN	Critical[3] Design Check	Width mm	t_p mm	Length ℓ mm	Bolt[4] Gauge g mm	Mark[4]	s mm	S275 mm	S355 mm	Capacity kN	Critical Check
686x254x170	8	1080	9(i)	200	10	570	120	ED8	8	7.5	6.4	686	11
{1600 kN}	7	947	9(i)	200	10	500	120	ED7	8	7.5	6.3	602	11
	6	818	9(i)	200	10	430	120	ED6	8	7.4	6.2	518	11
686x254x152	8	1080	9(i)	200	10	570	120	ED8	8	7.5	6.4	670	11
{1440 kN}	7	944	4	200	10	500	120	ED7	8	7.4	6.3	588	11
	6	812	4	200	10	430	120	ED6	8	7.4	6.2	506	11
686x254x140	8	1010	4	200	10	570	120	ED8	6	7.1	6.0	617	11
{1350 kN}	7	887	4	200	10	500	120	ED7	6	7.0	5.9	541	11
	6	763	4	200	10	430	120	ED6	6	6.9	5.8	465	11
686x254x125	8	954	4	200	10	570	120	ED8	6	6.7	5.6	610	11
{1260 kN}	7	837	4	200	10	500	120	ED7	6	6.6	5.6	535	11
	6	720	4	200	10	430	120	ED6	6	6.5	5.5	460	11
610x305x238	7	947	9(i)	200	10	500	120	ED7	8	7.5	6.3	648	11
{1860 kN}	6	818	9(i)	200	10	430	120	ED6	8	7.4	6.2	557	11
610x305x179	7	947	9(i)	200	10	500	120	ED7	8	7.5	6.3	597	11
{1390 kN}	6	818	9(i)	200	10	430	120	ED6	8	7.4	6.2	514	11
610x305x149	7	844	4	200	10	500	120	ED7	6	6.6	5.6	536	11
{1150 kN}	6	726	4	200	10	430	120	ED6	6	6.6	5.5	461	11
610x229x140	7	937	4	200	10	500	120	ED7	8	7.4	6.2	587	11
{1290 kN}	6	806	4	200	10	430	120	ED6	8	7.3	6.1	505	11
610x229x125	7	851	4	200	10	500	120	ED7	6	6.7	5.7	537	11
{1160 kN}	6	732	4	200	10	430	120	ED6	6	6.6	5.6	462	11
610x229x113	7	794	4	200	10	500	120	ED7	6	6.3	5.3	530	11
{1070 kN}	6	683	4	200	10	430	120	ED6	6	6.2	5.2	456	11
610x229x101	7	780	4	200	10	500	120	ED7	6	6.1	5.2	525	11
{1040 kN}	6	670	4	200	10	430	120	ED6	6	6.1	5.1	451	11

For guidance on the use of tables see Explanatory notes in Table H.17

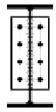
BEAM: S275
END PLATE: S275
HOLLO-BOLTS: M20, 8.8

Table H.22 *Continued*



FLEXIBLE END PLATES, HOLLO-BOLTS

200x10 or 180x8 mm End Plate

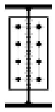


Beam Size { } (1)	Bolt Rows n	Un-notched Shear(2) Capacity kN	Un-notched Critical(3) Design Check	Fitting(4) End Plate Width mm	Fitting(4) End Plate t_p mm	Fitting(4) End Plate Length ℓ mm	Fitting(4) End Plate Bolt(4) Gauge g mm	Fitting(4) End Plate Mark(4)	Fillet(8) weld s mm	Min. Support(5) Thickness S275 mm	Min. Support(5) Thickness S355 mm	Tying(7) Capacity kN	Tying(7) Critical Check
533x210x122	6	781	4	200	10	430	120	ED6	8	7.1	6.0	501	11
{1100 kN}	5	654	4	200	10	360	120	ED5	8	7.1	5.9	419	11
533x210x109	6	714	4	200	10	430	120	ED6	6	6.5	5.4	459	11
{995 kN}	5	598	4	200	10	360	120	ED5	6	6.5	5.4	385	11
533x210x101	6	665	4	200	10	430	120	ED6	6	6.0	5.1	454	11
{922 kN}	5	556	4	200	10	360	120	ED5	6	6.0	5.1	380	11
533x210x92	6	645	4	200	10	430	120	ED6	6	5.8	4.9	448	11
{888 kN}	5	540	4	200	10	360	120	ED5	6	5.9	4.9	375	11
533x210x82	6	613	4	200	10	430	120	ED6	6	5.6	4.7	445	11
{837 kN}	5	513	4	200	10	360	120	ED5	6	5.6	4.7	372	11
457x191x98	5	551	9(i)	180	8	360	90	EC5	6	6.3	5.3	478	11
{847 kN}	4	447	9(i)	180	8	290	90	EC4	6	6.2	5.3	385	11
457x191x89	5	541	4	180	8	360	90	EC5	6	6.2	5.2	465	11
{774 kN}	4	436	4	180	8	290	90	EC4	6	6.1	5.1	375	11
457x191x82	5	529	4	180	8	360	90	EC5	6	6.0	5.1	457	11
{751 kN}	4	426	4	180	8	290	90	EC4	6	6.0	5.0	368	11
457x191x74	5	481	4	180	8	360	90	EC5	6	5.5	4.6	445	11
{679 kN}	4	388	4	180	8	290	90	EC4	6	5.4	4.6	358	11
457x191x67	5	454	4	180	8	360	90	EC5	6	5.2	4.4	438	11
{636 kN}	4	366	4	180	8	290	90	EC4	6	5.1	4.3	353	11

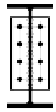
For guidance on the use of tables see Explanatory notes in Table H.17

BEAM: S275
END PLATE: S275
HOLLO-BOLTS: M20, 8.8

Table H.22 *Continued*



FLEXIBLE END PLATES, HOLLO-BOLTS
200x10 or 180x8 mm End Plate

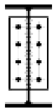


Beam Size { } (1)	Bolt Rows n	Un-notched Shear(2) Capacity kN	Un-notched Critical(3) Design Check	Fitting(4) End Plate Width mm	Fitting(4) End Plate t_p mm	Fitting(4) End Plate Length ℓ mm	Fitting(4) End Plate Bolt(4) Gauge g mm	Fitting(4) End Plate Mark(4)	Fillet(8) weld s mm	Min. Support(5) Thickness S275 mm	Min. Support(5) Thickness S355 mm	Tying(7) Capacity kN	Tying(7) Critical Check
457x152x82	5	541	4	180	8	360	90	EC5	6	6.2	5.2	465	11
{778 kN}	4	436	4	180	8	290	90	EC4	6	6.1	5.1	375	11
457x152x74	5	495	4	180	8	360	90	EC5	6	5.6	4.8	453	11
{705 kN}	4	398	4	180	8	290	90	EC4	6	5.6	4.7	365	11
457x152x67	5	481	4	180	8	360	90	EC5	6	5.5	4.6	445	11
{680 kN}	4	388	4	180	8	290	90	EC4	6	5.4	4.6	358	11
457x152x60	5	433	4	180	8	360	90	EC5	6	4.9	4.2	433	11
{608 kN}	4	349	4	180	8	290	90	EC4	6	4.9	4.1	349	11
457x152x52	5	406	4	180	8	360	90	EC5	6	4.6	3.9	427	11
{564 kN}	4	327	4	180	8	290	90	EC4	6	4.6	3.9	344	11
406x178x74	4	409	4	180	8	290	90	EC4	6	5.7	4.8	363	11
{647 kN}													
406x178x67	4	379	4	180	8	290	90	EC4	6	5.3	4.5	356	11
{594 kN}													
406x178x60	4	340	4	180	8	290	90	EC4	6	4.8	4.0	347	11
{530 kN}													
406x178x54	4	332	4	180	8	290	90	EC4	6	4.6	3.9	345	11
{512 kN}													
406x140x46	4	293	4	180	8	290	90	EC4	6	4.1	3.5	336	11
{452 kN}													
406x140x39	4	276	4	180	8	290	90	EC4	6	3.8	3.3	333	11
{420 kN}													
356x171x67	3	297	4	180	8	220	90	EC3	6	5.4	4.5	272	11
{546 kN}													
356x171x57	3	265	4	180	8	220	90	EC3	6	4.8	4.0	265	11
{478 kN}													
356x171x51	3	242	4	180	8	220	90	EC3	6	4.4	3.7	259	11
{433 kN}													
356x171x45	3	229	4	180	8	220	90	EC3	6	4.1	3.5	257	11
{406 kN}													
356x127x39	3	216	4	180	8	220	90	EC3	6	3.9	3.3	254	11
{385 kN}													
356x127x33	3	196	4	180	8	220	90	EC3	6	3.6	3.0	250	11
{346 kN}													

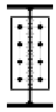
For guidance on the use of tables see Explanatory notes in Table H.17

BEAM: S275
END PLATE: S275
HOLLO-BOLTS: M20, 8.8

Table H.22 *Continued*



FLEXIBLE END PLATES, HOLLO-BOLTS
200x10 or 180x8 mm End Plate



Beam Size { } (1)	Bolt Rows n	Un-notched: Shear(2) Capacity kN	Un-notched: Critical(3) Design Check	Fitting(4) End Plate: Width mm	Fitting(4) End Plate: t_p mm	Fitting(4) End Plate: Length ℓ mm	Fitting(4) End Plate: Bolt(4) Gauge g mm	Fitting(4) End Plate: Mark(4)	Fillet(8) weld s mm	Min. Support(5) Thickness: S275 mm	Min. Support(5) Thickness: S355 mm	Tying(7): Capacity kN	Tying(7): Critical Check
305x165x54 {405 kN}	3	258	4	180	8	220	90	EC3	6	4.7	3.9	263	11
305x165x46 {339 kN}	3	219	4	180	8	220	90	EC3	6	4.0	3.3	254	11
305x165x40 {300 kN}	3	196	4	180	8	220	90	EC3	6	3.6	3.0	250	11
305x127x48 {462 kN}	3	294	4	180	8	220	90	EC3	6	5.3	4.5	272	11
305x127x42 {406 kN}	3	261	4	180	8	220	90	EC3	6	4.7	4.0	264	11
305x127x37 {357 kN}	3	232	4	180	8	220	90	EC3	6	4.2	3.5	257	11
305x102x33 {341 kN}	3	216	4	180	8	220	90	EC3	6	3.9	3.3	254	11
305x102x28 {306 kN}	3	196	4	180	8	220	90	EC3	6	3.6	3.0	250	11
305x102x25 {292 kN}	3	189	4	180	8	220	90	EC3	6	3.4	2.9	248	11
254x146x43 {308 kN}	2	160	4	180	8	150	90	EC2	6	4.4	3.6	176	11
254x146x37 {266 kN}	2	140	4	180	8	150	90	EC2	6	3.8	3.2	172	11
254x146x31 {249 kN}	2	134	4	180	8	150	90	EC2	6	3.6	3.0	170	11
254x102x28 {271 kN}	2	140	4	180	8	150	90	EC2	6	3.8	3.2	172	11
254x102x25 {255 kN}	2	134	4	180	8	150	90	EC2	6	3.6	3.0	170	11
254x102x22 {239 kN}	2	127	4	180	8	150	90	EC2	6	3.5	2.9	169	11

For guidance on the use of tables see Explanatory notes in Table H.17

BEAM: S355
END PLATE: S275
HOLLO-BOLTS: M20, 8.8

Table H.23

FLEXIBLE END PLATES, HOLLO-BOLTS

200x10 or 180x8 mm End Plate

Beam Size	Bolt Rows	Un-notched		Fitting[4] End Plate					Fillet[8] weld	Min. Support[5] Thickness		Tying[7]	
{ } [1]	n	Shear[2] Capacity kN	Critical[3] Design Check	Width mm	t_p mm	Length ℓ mm	Bolt[4] Gauge g mm	Mark[4]	s mm	S275 mm	S355 mm	Capacity kN	Critical Check
914x305x253	11	1460	9(i)	200	10	780	120	ED11	8	7.6	6.5	990	11
{3290 kN}	10	1340	9(i)	200	10	710	120	ED10	8	7.6	6.4	901	11
	9	1210	9(i)	200	10	640	120	ED9	8	7.6	6.4	812	11
	8	1080	9(i)	200	10	570	120	ED8	8	7.5	6.4	723	11
914x305x224	11	1460	9(i)	200	10	780	120	ED11	8	7.6	6.5	964	11
{3000 kN}	10	1340	9(i)	200	10	710	120	ED10	8	7.6	6.4	877	11
	9	1210	9(i)	200	10	640	120	ED9	8	7.6	6.4	791	11
	8	1080	9(i)	200	10	570	120	ED8	8	7.5	6.4	704	11
914x305x201	11	1460	9(i)	200	10	780	120	ED11	8	7.6	6.5	949	11
{2820 kN}	10	1340	9(i)	200	10	710	120	ED10	8	7.6	6.4	864	11
	9	1210	9(i)	200	10	640	120	ED9	8	7.6	6.4	779	11
	8	1080	9(i)	200	10	570	120	ED8	8	7.5	6.4	694	11
838x292x226	10	1340	9(i)	200	10	710	120	ED10	8	7.6	6.4	880	11
{2840 kN}	9	1210	9(i)	200	10	640	120	ED9	8	7.6	6.4	794	11
	8	1080	9(i)	200	10	570	120	ED8	8	7.5	6.4	707	11
838x292x194	10	1340	9(i)	200	10	710	120	ED10	8	7.6	6.4	858	11
{2560 kN}	9	1210	9(i)	200	10	640	120	ED9	8	7.6	6.4	773	11
	8	1080	9(i)	200	10	570	120	ED8	8	7.5	6.4	689	11
838x292x176	10	1340	9(i)	200	10	710	120	ED10	8	7.6	6.4	847	11
{2420 kN}	9	1210	9(i)	200	10	640	120	ED9	8	7.6	6.4	763	11
	8	1080	9(i)	200	10	570	120	ED8	8	7.5	6.4	680	11
762x267x197	9	1210	9(i)	200	10	640	120	ED9	8	7.6	6.4	786	11
{2490 kN}	8	1080	9(i)	200	10	570	120	ED8	8	7.5	6.4	700	11
	7	947	9(i)	200	10	500	120	ED7	8	7.5	6.3	614	11
762x267x173	9	1210	9(i)	200	10	640	120	ED9	8	7.6	6.4	768	11
{2260 kN}	8	1080	9(i)	200	10	570	120	ED8	8	7.5	6.4	684	11
	7	947	9(i)	200	10	500	120	ED7	8	7.5	6.3	600	11
762x267x147	9	1210	9(i)	200	10	640	120	ED9	8	7.6	6.4	747	11
{2000 kN}	8	1080	9(i)	200	10	570	120	ED8	8	7.5	6.4	665	11
	7	947	9(i)	200	10	500	120	ED7	8	7.5	6.3	584	11
762x267x134	9	1210	9(i)	200	10	640	120	ED9	8	7.6	6.4	737	11
{1920 kN}	8	1080	9(i)	200	10	570	120	ED8	8	7.5	6.4	656	11
	7	947	9(i)	200	10	500	120	ED7	8	7.5	6.3	575	11

For guidance on the use of tables see Explanatory notes in Table H.17

BEAM: S355
END PLATE: S275
HOLLO-BOLTS: M20, 8.8

Table H.23 *Continued*

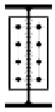
FLEXIBLE END PLATES, HOLLO-BOLTS
200x10 or 180x8 mm End Plate

Beam Size { } (1)	Bolt Rows n	Un-notched Shear(2) Capacity kN	Un-notched Critical(3) Design Check	Fitting(4) End Plate Width mm	Fitting(4) End Plate t_p mm	Fitting(4) End Plate Length ℓ mm	Fitting(4) End Plate Bolt(4) Gauge g mm	Fitting(4) End Plate Mark(4)	Fillet(8) weld s mm	Min. Support(5) Thickness S275 mm	Min. Support(5) Thickness S355 mm	Tying(7) Capacity kN	Tying(7) Critical Check
686x254x170	8	1080	9(i)	200	10	570	120	ED8	8	7.5	6.4	686	11
{2080 kN}	7	947	9(i)	200	10	500	120	ED7	8	7.5	6.3	602	11
	6	818	9(i)	200	10	430	120	ED6	8	7.4	6.2	518	11
686x254x152	8	1080	9(i)	200	10	570	120	ED8	8	7.5	6.4	670	11
{1880 kN}	7	947	9(i)	200	10	500	120	ED7	8	7.5	6.3	588	11
	6	818	9(i)	200	10	430	120	ED6	8	7.4	6.2	506	11
686x254x140	8	1080	9(i)	200	10	570	120	ED8	8	7.5	6.4	661	11
{1750 kN}	7	947	9(i)	200	10	500	120	ED7	8	7.5	6.3	580	11
	6	818	9(i)	200	10	430	120	ED6	8	7.4	6.2	498	11
686x254x125	8	1080	9(i)	200	10	570	120	ED8	8	7.5	6.4	653	11
{1640 kN}	7	947	9(i)	200	10	500	120	ED7	8	7.5	6.3	572	11
	6	818	9(i)	200	10	430	120	ED6	8	7.4	6.2	492	11
610x305x238	7	947	9(i)	200	10	500	120	ED7	8	7.5	6.3	648	11
{2420 kN}	6	818	9(i)	200	10	430	120	ED6	8	7.4	6.2	557	11
610x305x179	7	947	9(i)	200	10	500	120	ED7	8	7.5	6.3	597	11
{1810 kN}	6	818	9(i)	200	10	430	120	ED6	8	7.4	6.2	514	11
610x305x149	7	947	9(i)	200	10	500	120	ED7	8	7.5	6.3	573	11
{1500 kN}	6	818	9(i)	200	10	430	120	ED6	8	7.4	6.2	493	11
610x229x140	7	947	9(i)	200	10	500	120	ED7	8	7.5	6.3	587	11
{1670 kN}	6	818	9(i)	200	10	430	120	ED6	8	7.4	6.2	505	11
610x229x125	7	947	9(i)	200	10	500	120	ED7	8	7.5	6.3	574	11
{1510 kN}	6	818	9(i)	200	10	430	120	ED6	8	7.4	6.2	494	11
610x229x113	7	947	9(i)	200	10	500	120	ED7	8	7.5	6.3	566	11
{1400 kN}	6	818	9(i)	200	10	430	120	ED6	8	7.4	6.2	487	11
610x229x101	7	947	9(i)	200	10	500	120	ED7	8	7.5	6.3	561	11
{1350 kN}	6	818	9(i)	200	10	430	120	ED6	8	7.4	6.2	482	11

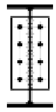
For guidance on the use of tables see Explanatory notes in Table H.17

BEAM: S355
END PLATE: S275
HOLLO-BOLTS: M20, 8.8

Table H.23 *Continued*



FLEXIBLE END PLATES, HOLLO-BOLTS
200x10 or 180x8 mm End Plate

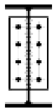


Beam Size	Bolt Rows	Un-notched		Fitting[4] End Plate					Fillet[8] weld	Min. Support[5] Thickness		Tying[7]	
	n	Shear[2] Capacity	Critical[3] Design Check	Width	t_p	Length ℓ	Bolt[4] Gauge g	Mark[4]	s	S275	S355	Capacity	Critical Check
{ } [1]		kN		mm	mm	mm	mm		mm	mm	mm	kN	
533x210x122	6	818	9(i)	200	10	430	120	ED6	8	7.4	6.2	501	11
{1430 kN}	5	688	9(i)	200	10	360	120	ED5	8	7.5	6.3	419	11
533x210x109	6	818	9(i)	200	10	430	120	ED6	8	7.4	6.2	491	11
{1300 kN}	5	688	9(i)	200	10	360	120	ED5	8	7.5	6.3	411	11
533x210x101	6	818	9(i)	200	10	430	120	ED6	8	7.4	6.2	485	11
{1200 kN}	5	688	9(i)	200	10	360	120	ED5	8	7.5	6.3	406	11
533x210x92	6	818	9(i)	200	10	430	120	ED6	8	7.4	6.2	479	11
{1150 kN}	5	688	9(i)	200	10	360	120	ED5	8	7.5	6.3	401	11
533x210x82	6	791	4	200	10	430	120	ED6	8	7.2	6.0	475	11
{1080 kN}	5	663	4	200	10	360	120	ED5	8	7.2	6.0	398	11
457x191x98	5	551	9(i)	180	8	360	90	EC5	6	6.3	5.3	478	11
{1100 kN}	4	447	9(i)	180	8	290	90	EC4	6	6.2	5.3	385	11
457x191x89	5	551	9(i)	180	8	360	90	EC5	6	6.3	5.3	465	11
{1010 kN}	4	447	9(i)	180	8	290	90	EC4	6	6.2	5.3	375	11
457x191x82	5	551	9(i)	180	8	360	90	EC5	6	6.3	5.3	457	11
{970 kN}	4	447	9(i)	180	8	290	90	EC4	6	6.2	5.3	368	11
457x191x74	5	551	9(i)	180	8	360	90	EC5	6	6.3	5.3	445	11
{876 kN}	4	447	9(i)	180	8	290	90	EC4	6	6.2	5.3	358	11
457x191x67	5	551	9(i)	180	8	360	90	EC5	6	6.3	5.3	438	11
{821 kN}	4	447	9(i)	180	8	290	90	EC4	6	6.2	5.3	353	11

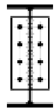
For guidance on the use of tables see Explanatory notes in Table H.17

BEAM: S355
END PLATE: S275
HOLLO-BOLTS: M20, 8.8

Table H.23 *Continued*



FLEXIBLE END PLATES, HOLLO-BOLTS
200x10 or 180x8 mm End Plate

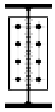


Beam Size	Bolt Rows	Un-notched		Fitting[4] End Plate					Fillet[8] welds	Min. Support[5] Thickness		Tying[7]	
	n	Shear[2] Capacity kN	Critical[3] Design Check	Width	t_p	Length ℓ	Bolt[4] Gauge g	Mark[4]		S275	S355	Capacity	Critical Check
{ } [1]				mm	mm	mm	mm		mm	mm	mm	kN	
457x152x82	5	551	9(i)	180	8	360	90	EC5	6	6.3	5.3	465	11
{1010 kN}	4	447	9(i)	180	8	290	90	EC4	6	6.2	5.3	375	11
457x152x74	5	551	9(i)	180	8	360	90	EC5	6	6.3	5.3	453	11
{918 kN}	4	447	9(i)	180	8	290	90	EC4	6	6.2	5.3	365	11
457x152x67	5	551	9(i)	180	8	360	90	EC5	6	6.3	5.3	445	11
{878 kN}	4	447	9(i)	180	8	290	90	EC4	6	6.2	5.3	358	11
457x152x60	5	551	9(i)	180	8	360	90	EC5	6	6.3	5.3	433	11
{784 kN}	4	447	9(i)	180	8	290	90	EC4	6	6.2	5.3	349	11
457x152x52	5	524	4	180	8	360	90	EC5	6	6.0	5.0	427	11
{728 kN}	4	423	4	180	8	290	90	EC4	6	5.9	5.0	344	11
406x178x74	4	447	9(i)	180	8	290	90	EC4	6	6.2	5.3	363	11
{835 kN}													
406x178x67	4	447	9(i)	180	8	290	90	EC4	6	6.2	5.3	356	11
{767 kN}													
406x178x60	4	439	4	180	8	290	90	EC4	6	6.1	5.2	347	11
{684 kN}													
406x178x54	4	428	4	180	8	290	90	EC4	6	6.0	5.1	345	11
{660 kN}													
406x140x46	4	378	4	180	8	290	90	EC4	6	5.3	4.5	336	11
{584 kN}													
406x140x39	4	356	4	180	8	290	90	EC4	6	5.0	4.2	333	11
{543 kN}													
356x171x67	3	344	9(i)	180	8	220	90	EC3	6	6.2	5.2	272	11
{704 kN}													
356x171x57	3	342	4	180	8	220	90	EC3	6	6.2	5.2	265	11
{618 kN}													
356x171x51	3	312	4	180	8	220	90	EC3	6	5.7	4.8	259	11
{560 kN}													
356x171x45	3	295	4	180	8	220	90	EC3	6	5.3	4.5	257	11
{524 kN}													
356x127x39	3	278	4	180	8	220	90	EC3	6	5.0	4.2	254	11
{497 kN}													
356x127x33	3	253	4	180	8	220	90	EC3	6	4.6	3.9	250	11
{446 kN}													

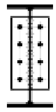
For guidance on the use of tables see Explanatory notes in Table H.17

BEAM: S355
END PLATE: S275
HOLLO-BOLTS: M20, 8.8

Table H.23 *Continued*



FLEXIBLE END PLATES, HOLLO-BOLTS
200x10 or 180x8 mm End Plate



Beam Size	Bolt Rows	Un-notched		Fitting[4] End Plate					Fillet[8] weld	Min. Support[5] Thickness		Tying[7]	
	n	Shear[2] Capacity kN	Critical[3] Design Check	Width	t_p	Length ℓ	Bolt[4] Gauge g	Mark[4]	s	S275	S355	Capacity	Critical Check
{ } [1]				mm	mm	mm	mm		mm	mm	mm	kN	
305x165x54 {522 kN}	3	333	4	180	8	220	90	EC3	6	6.0	5.1	263	11
305x165x46 {438 kN}	3	283	4	180	8	220	90	EC3	6	5.1	4.3	254	11
305x165x40 {388 kN}	3	253	4	180	8	220	90	EC3	6	4.6	3.9	250	11
305x127x48 {596 kN}	3	344	9(i)	180	8	220	90	EC3	6	6.2	5.2	272	11
305x127x42 {523 kN}	3	337	4	180	8	220	90	EC3	6	6.1	5.1	264	11
305x127x37 {460 kN}	3	299	4	180	8	220	90	EC3	6	5.4	4.6	257	11
305x102x33 {440 kN}	3	278	4	180	8	220	90	EC3	6	5.0	4.2	254	11
305x102x28 {395 kN}	3	253	4	180	8	220	90	EC3	6	4.6	3.9	250	11
305x102x25 {377 kN}	3	245	4	180	8	220	90	EC3	6	4.4	3.7	248	11
254x146x43 {398 kN}	2	207	4	180	8	150	90	EC2	6	5.6	4.7	176	11
254x146x37 {344 kN}	2	181	4	180	8	150	90	EC2	6	4.9	4.1	172	11
254x146x31 {321 kN}	2	173	4	180	8	150	90	EC2	6	4.7	3.9	170	11
254x102x28 {349 kN}	2	181	4	180	8	150	90	EC2	6	4.9	4.1	172	11
254x102x25 {329 kN}	2	173	4	180	8	150	90	EC2	6	4.7	3.9	170	11
254x102x22 {308 kN}	2	164	4	180	8	150	90	EC2	6	4.5	3.7	169	11

For guidance on the use of tables see Explanatory notes in Table H.17

Table H.24

Explanatory notes - FIN PLATES
Use of Capacity Tables

The following notes refer to the **suffix** numbers given at the top of the column descriptions for Tables H.27 to H.30.

The check numbers refer to those listed in Table H.26 and described in Section 6.5 Design procedures.

The capacity tables are based on the standard details given in Table H.25.

All universal beam sections which are suitable for the minimum standard fitting size and the standard notch size are shown in the tables (i.e. if T + r > 50 mm, section is not included).

SHADED PORTION OF TABLES:

Connections for beams in the shaded portion of the tables may only be used when:

- the supported beam span/depth ratio ≤ 20
- the gap between the supported beam end and the supporting element = 20 mm
- the maximum number of bolt rows = 8 (i.e. (n – 1)p = 490 mm, which does not exceed the limit of 530 mm).

(1) SHEAR CAPACITY OF THE BEAM

The value given in { } is the shear capacity of the beam, given by $0.6p_yt_wD$.

(2) SHEAR CAPACITY OF THE CONNECTION

This is the critical value of the design checks for the 'supported beam side' of the connection. i.e. the minimum capacity from CHECKS 2, 3(i), 3(ii), 4(i), 4(ii) and 8.

(3) CRITICAL DESIGN CHECK

The check which gives the critical value of shear capacity. See Table H.26 for the description of the checks.

(4) FITTINGS

The length and type of Standard Fin Plate fittings. See Tables H.1 and H.4 for details

(5) MINIMUM SUPPORT THICKNESS

This is the minimum thickness of supporting column or beam element, that is needed to carry the given **SHEAR CAPACITY** (Un-notched or Single Notched) of the connection. It is derived from CHECK 10.

For a symmetrical two sided connection, the minimum support thickness would be twice the tabulated value.

(6) MAXIMUM NOTCH LENGTH ($C + T_1$)

These are maximum lengths of notches for single and double notched beams that can be accommodated if the beam is to carry the tabulated corresponding **SHEAR CAPACITY** of the connection.

It is assumed that the beam is fully restrained against lateral torsional buckling, and the notched lengths are derived from CHECKS 5 and 6.

To provide a simple check for double notched beams, it has been assumed that the remaining web depth, 'y' (see Table H.25) is the same as the end plate length.

* Indicates that the condition from CHECK 6, i.e. $d_{c2} \leq D/5$ is <u>not</u> satisfied.

(7) TYING CAPACITY

This is the critical value of the design checks for the 'supported beam' side of the connection, i.e. the minimum capacity from CHECKS 11(i), 11(ii), 12(i) and12(ii).

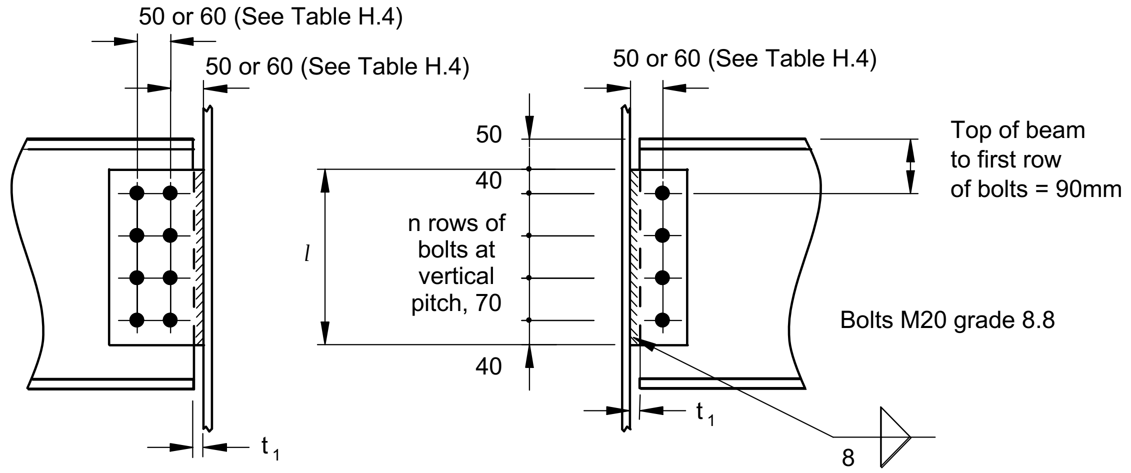
In CHECK 12 the supported beam is assumed to be a double notched member.

Separate checks will have to be carried out on the supporting member. (See CHECKS 14, 15 and 16).

Table H.25

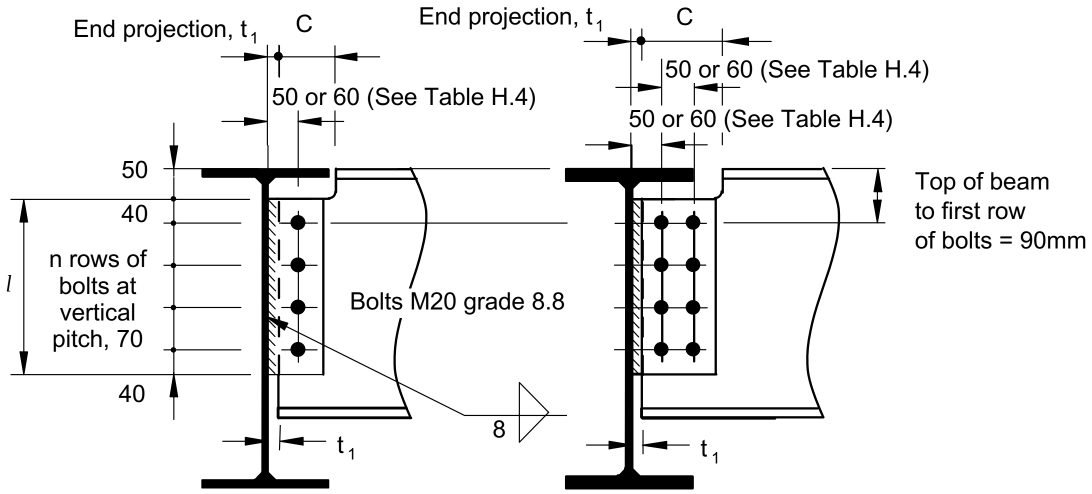
FIN PLATES

Standard details used in Capacity Tables

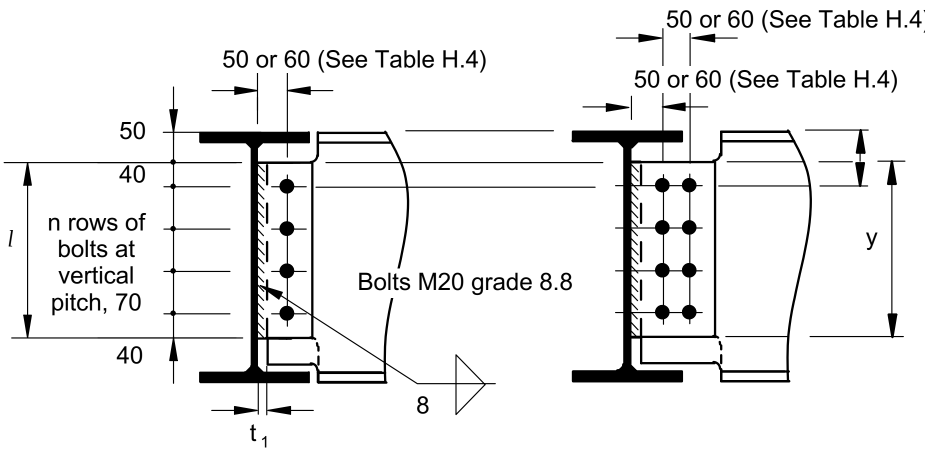


t_1 = 10 mm for supported beams ≤ 610 x 305 UB
t_1 = 20 mm for supported beams > 610 x 305 UB

UN-NOTCHED BEAM



SINGLE NOTCHED BEAM



Top of beam to first row of bolts = 90mm

Note:
For simplicity the capacity tables assume that the remaining depth of beam web, y(mm), is equal to the fin plate length, l (mm).

DOUBLE NOTCHED BEAM

Note: See Tables H.1 and H.4 Standard Fittings, Marks and details for Fin Plates

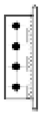
Table H.26

FIN PLATES Critical Check Description List			
		Check Number	**Description**
SHEAR CAPACITY	Supported beam side	2	Capacity of bolt group
		3(i)	Shear capacity of fin plate
		3(ii)	Shear and bending interaction capacity of fin plate
		4(i)	Shear capacity of supported beam at the connection
		4(ii)	Shear and bending interaction capacity of supported beam at the 2nd bolt line connection
	Supporting member side	8	Capacity of fillet weld
STRUCTURAL INTEGRITY		11(i)	Tension capacity of fin plate
		11(ii)	Bearing capacity of fin plate
		12(i)	Tension capacity of supported beam web at the connection
		12(ii)	Bearing capacity of supported beam web at the connection

Note: This table only lists the critical checks.
For a full list of design checks and further information, see Section 6.5.

BEAM: S275
FIN PLATE: S275
BOLTS: M20, 8.8

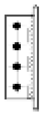
Table H.27



FIN PLATES, ORDINARY BOLTS

Single line of bolts

120x10mm or 100x10mm Fin Plate



Beam Size { } (1)	Bolt Rows n	Un-notched or Single Notch: Shear(2) Capacity kN	Un-notched or Single Notch: Critical(3) Design Check	Double Notch: Shear(2) Capacity kN	Double Notch: Critical(3) Design Check	Fitting(4) Fin Plate: Width mm	Fitting(4) Fin Plate: t_p mm	Fitting(4) Fin Plate: Length ℓ mm	Fitting(4) Fin Plate: Mark	Min. Support(5) Thickness: S275 mm	Min. Support(5) Thickness: S355 mm	Max. Notch Length(6) $c+t_1$: Single mm	Max. Notch Length(6) $c+t_1$: Double mm	Tying(7): Capacity kN	Tying(7): Critical Check
914x305x253	8	639	2	639	2	120	10	570	FC8	3.8	2.9	928	388*	735	13
{2530 kN}															
914x305x224	8	639	2	639	2	120	10	570	FC8	3.8	2.9	786	357*	735	13
{2300 kN}															
914x305x201	8	639	2	639	2	120	10	570	FC8	3.8	2.9	686	339*	735	13
{2170 kN}															
838x292x226	8	639	2	639	2	120	10	570	FC8	3.8	2.9	861	362*	735	13
{2180 kN}															
838x292x194	8	639	2	639	2	120	10	570	FC8	3.8	2.9	729	330*	735	13
{1960 kN}															
838x292x176	8	639	2	639	2	120	10	570	FC8	3.8	2.9	640	314*	735	13
{1860 kN}															
762x267x197	8	639	2	639	2	120	10	570	FC8	3.8	2.9	780	350	735	13
{1910 kN}	7	542	2	542	2	120	10	500	FC7	3.6	2.8	780	318*	643	13
762x267x173	8	639	2	639	2	120	10	570	FC8	3.8	2.9	743	321	735	13
{1730 kN}	7	542	2	542	2	120	10	500	FC7	3.6	2.8	772	291*	643	13
762x267x147	8	639	2	639	2	120	10	570	FC8	3.8	2.9	600	287	735	13
{1530 kN}	7	542	2	542	2	120	10	500	FC7	3.6	2.8	600	261*	643	13
762x267x134	8	639	2	639	2	120	10	570	FC8	3.8	2.9	502	280	735	13
{1490 kN}	7	542	2	542	2	120	10	500	FC7	3.6	2.8	502	254*	643	13

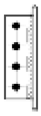
For guidance on the use of tables see Explanatory notes in Table H.24

Note: (7) The minimum capacity from Checks 11(i), 11(ii), 12(i), 12(ii) and 13 (Shear capacity of the bolt group)

Incorporating corrigendum 1, 2002 and corrigendum 2, 2005. (See page 491)

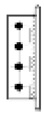
BEAM: S275
FIN PLATE: S275
BOLTS: M20, 8.8

Table H.27 *Continued*



FIN PLATES, ORDINARY BOLTS

Single line of bolts
120x10mm or 100x10mm Fin Plate



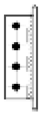
Beam Size	Bolt Rows	Un-notched or Single Notch		Double Notch		Fitting[4] Fin Plate				Min. Support[5] Thickness		Max. Notch Length[6] $c+t_1$		Tying[7]	
{ }[1]	n	Shear[2] Capacity kN	Critical[3] Design Check	Shear[2] Capacity kN	Critical[3] Design Check	Width mm	t_p mm	Length ℓ mm	Mark	S275 mm	S355 mm	Single mm	Double mm	Capacity kN	Critical Check
686x254x170	8	639	2	639	2	120	10	570	FC8	3.8	2.9	624	326	735	13
{1600 kN}	7	542	2	542	2	120	10	500	FC7	3.6	2.8	703	296*	643	13
	6	445	2	445	2	120	10	430	FC6	3.5	2.7	703	266*	551	13
686x254x152	8	639	2	639	2	120	10	570	FC8	3.8	2.9	559	296	735	13
{1440 kN}	7	542	2	542	2	120	10	500	FC7	3.6	2.8	659	269	643	13
	6	445	2	445	2	120	10	430	FC6	3.5	2.7	698	242*	551	13
686x254x140	8	639	2	639	2	120	10	570	FC8	3.8	2.9	517	278	735	13
{1350 kN}	7	542	2	542	2	120	10	500	FC7	3.6	2.8	610	253	643	13
	6	445	2	445	2	120	10	430	FC6	3.5	2.7	663	228*	551	13
686x254x125	8	639	2	639	2	120	10	570	FC8	3.8	2.9	474	263	735	13
{1260 kN}	7	542	2	542	2	120	10	500	FC7	3.6	2.8	559	238	643	13
	6	445	2	445	2	120	10	430	FC6	3.5	2.7	568	215*	551	13
610x305x238	7	568	2	568	2	100	10	500	FA7	3.8	3.0	646	358	643	13
{1860 kN}	6	471	2	471	2	100	10	430	FA6	3.7	2.9	646	319*	551	13
610x305x179	7	568	2	568	2	100	10	500	FA7	3.8	3.0	563	274	643	13
{1390 kN}	6	471	2	471	2	100	10	430	FA6	3.7	2.9	630	245*	551	13
610x305x149	7	568	2	568	2	100	10	500	FA7	3.8	3.0	461	230	643	13
{1150 kN}	6	471	2	471	2	100	10	430	FA6	3.7	2.9	556	205*	551	13
610x229x140	7	568	2	568	2	100	10	500	FA7	3.8	3.0	497	255	643	13
{1290 kN}	6	471	2	471	2	100	10	430	FA6	3.7	2.9	600	227*	551	13
610x229x125	7	568	2	568	2	100	10	500	FA7	3.8	3.0	444	231	643	13
{1160 kN}	6	471	2	471	2	100	10	430	FA6	3.7	2.9	536	206*	551	13
610x229x113	7	568	2	568	2	100	10	500	FA7	3.8	3.0	406	204	643	13
{1070 kN}	6	471	2	471	2	100	10	430	FA6	3.7	2.9	489	193*	551	13
610x229x101	7	568	2	568	2	100	10	500	FA7	3.8	3.0	387	195	643	13
{1040 kN}	6	471	2	471	2	100	10	430	FA6	3.7	2.9	467	184*	551	13

For guidance on the use of tables see Explanatory notes in Table H.24

Note: (7) The minimum capacity from Checks 11(i), 11(ii), 12(i), 12(ii) and 13 (Shear capacity of the bolt group)

BEAM: S275
FIN PLATE: S275
BOLTS: M20, 8.8

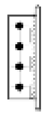
Table H.27 *Continued*



FIN PLATES, ORDINARY BOLTS

Single line of bolts

120x10mm or 100x10mm Fin Plate



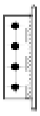
Beam Size { } (1)	Bolt Rows n	Un-notched or Single Notch		Double Notch		Fitting(4) Fin Plate				Min. Support(5) Thickness		Max. Notch Length(6) $c+t_1$		Tying(7)	
		Shear(2) Capacity kN	Critical(3) Design Check	Shear(2) Capacity kN	Critical(3) Design Check	Width mm	t_p mm	Length ℓ mm	Mark	S275 mm	S355 mm	Single mm	Double mm	Capacity kN	Critical Check
533x210x122	6	471	2	471	2	100	10	430	FA6	3.7	2.9	443	220	551	13
{1100 kN}	5	374	2	374	2	100	10	360	FA5	3.5	2.7	555	194*	459	13
533x210x109	6	471	2	471	2	100	10	430	FA6	3.7	2.9	397	201	551	13
{995 kN}	5	374	2	374	2	100	10	360	FA5	3.5	2.7	499	177*	459	13
533x210x101	6	471	2	471	2	100	10	430	FA6	3.7	2.9	366	180	551	13
{922 kN}	5	374	2	374	2	100	10	360	FA5	3.5	2.7	460	165*	459	13
533x210x92	6	471	2	471	2	100	10	430	FA6	3.7	2.9	349	167	551	13
{888 kN}	5	374	2	374	2	100	10	360	FA5	3.5	2.7	439	160*	459	13
533x210x82	6	452	2	452	2	100	10	430	FA6	3.5	2.7	334	162	530	12 (ii)
{837 kN}	5	359	2	359	2	100	10	360	FA5	3.4	2.6	421	155*	442	12 (ii)
457x191x98	5	374	2	374	2	100	10	360	FA5	3.5	2.7	360	174	459	13
{847 kN}	4	279	2	279	2	100	10	290	FA4	3.2	2.5	477	152*	368	13
457x191x89	5	374	2	374	2	100	10	360	FA5	3.5	2.7	327	161	459	13
{774 kN}	4	279	2	279	2	100	10	290	FA4	3.2	2.5	437	140*	368	13
457x191x82	5	371	2	371	2	100	10	360	FA5	3.5	2.7	317	155	455	12 (ii)
{751 kN}	4	277	2	277	2	100	10	290	FA4	3.2	2.5	424	138*	364	12 (ii)
457x191x74	5	337	2	337	2	100	10	360	FA5	3.2	2.4	313	155	414	12 (ii)
{679 kN}	4	251	2	251	2	100	10	290	FA4	2.9	2.3	420	138*	331	12 (ii)
457x191x67	5	318	2	318	2	100	10	360	FA5	3.0	2.3	306	155	391	12 (ii)
{636 kN}	4	237	2	237	2	100	10	290	FA4	2.8	2.1	410	138*	313	12 (ii)

For guidance on the use of tables see Explanatory notes in Table H.24

Note: (7) The minimum capacity from Checks 11(i), 11(ii), 12(i), 12(ii) and 13 (Shear capacity of the bolt group)

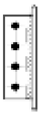
BEAM: S275
FIN PLATE: S275
BOLTS: M20, 8.8

Table H.27 *Continued*



FIN PLATES, ORDINARY BOLTS

Single line of bolts
120x10mm or 100x10mm Fin Plate



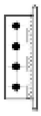
Beam Size { }[1]	Bolt Rows n	Un-notched or Single Notch: Shear[2] Capacity kN	Un-notched or Single Notch: Critical[3] Design Check	Double Notch: Shear[2] Capacity kN	Double Notch: Critical[3] Design Check	Fitting[4] Fin Plate: Width mm	Fitting[4] Fin Plate: t_p mm	Fitting[4] Fin Plate: Length ℓ mm	Fitting[4] Fin Plate: Mark	Min. Support[5] Thickness: S275 mm	Min. Support[5] Thickness: S355 mm	Max. Notch Length[6] $c+t_1$: Single mm	Max. Notch Length[6] $c+t_1$: Double mm	Tying[7]: Capacity kN	Tying[7]: Critical Check
457x152x82	5	374	2	374	2	100	10	360	FA5	3.5	2.7	322	161	459	13
{778 kN}	4	279	2	279	2	100	10	290	FA4	3.2	2.5	431	140*	368	13
457x152x74	5	359	2	359	2	100	10	360	FA5	3.4	2.6	301	142	442	12 (ii)
{705 kN}	4	268	2	268	2	100	10	290	FA4	3.1	2.4	404	133*	353	12 (ii)
457x152x67	5	337	2	337	2	100	10	360	FA5	3.2	2.4	305	155	414	12 (ii)
{680 kN}	4	251	2	251	2	100	10	290	FA4	2.9	2.3	409	138*	331	12 (ii)
457x152x60	5	303	2	303	2	100	10	360	FA5	2.8	2.2	301	155	373	12 (ii)
{608 kN}	4	226	2	226	2	100	10	290	FA4	2.6	2.0	403	138*	298	12 (ii)
457x152x52	5	284	2	284	2	100	10	360	FA5	2.7	2.1	289	155	350	12 (ii)
{564 kN}	4	212	2	212	2	100	10	290	FA4	2.5	1.9	357	138*	280	12 (ii)
406x178x74 {647 kN}	4	265	2	265	2	100	10	290	FA4	3.1	2.4	336	138	350	12 (ii)
406x178x67 {594 kN}	4	246	2	246	2	100	10	290	FA4	2.9	2.2	329	138	324	12 (ii)
406x178x60 {530 kN}	4	221	2	221	2	100	10	290	FA4	2.6	2.0	325	138	291	12 (ii)
406x178x54 {512 kN}	4	215	2	215	2	100	10	290	FA4	2.5	1.9	314	138	283	12 (ii)
406x140x46 {452 kN}	4	190	2	190	2	100	10	290	FA4	2.2	1.7	311	138	250	12 (ii)
406x140x39 {420 kN}	4	179	2	179	2	100	10	290	FA4	2.1	1.6	275	138	236	12 (ii)
356x171x67 {546 kN}	3	171	2	171	2	100	10	220	FA3	2.6	2.2	373	118*	251	12 (ii)
356x171x57 {478 kN}	3	153	2	153	2	100	10	220	FA3	2.3	1.9	364	118*	224	12 (ii)
356x171x51 {433 kN}	3	139	2	139	2	100	10	220	FA3	2.1	1.8	357	118*	204	12 (ii)
356x171x45 {406 kN}	3	132	2	132	2	100	10	220	FA3	2.0	1.7	345	118*	193	12 (ii)
356x127x39 {385 kN}	3	124	2	124	2	100	10	220	FA3	1.9	1.6	340	118*	182	12 (ii)
356x127x33 {346 kN}	3	113	2	113	2	100	10	220	FA3	1.7	1.4	294	118*	166	12 (ii)

For guidance on the use of tables see Explanatory notes in Table H.24

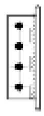
Note: (7) The minimum capacity from Checks 11(i), 11(ii), 12(i), 12(ii) and 13 (Shear capacity of the bolt group)

BEAM: S275
FIN PLATE: S275
BOLTS: M20, 8.8

Table H.27 *Continued*



FIN PLATES, ORDINARY BOLTS



Single line of bolts
120x10mm or 100x10mm Fin Plate

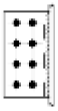
Beam Size { }[1]	Bolt Rows n	Un-notched or Single Notch		Double Notch		Fitting[4] Fin Plate				Min. Support[5] Thickness		Max. Notch Length[6] $c+t_1$		Tying[7]	
		Shear[2] Capacity kN	Critical[3] Design Check	Shear[2] Capacity kN	Critical[3] Design Check	Width mm	t_p mm	Length ℓ mm	Mark	S275 mm	S355 mm	Single mm	Double mm	Capacity kN	Critical Check
305x165x54 {405 kN}	3	149	2	149	2	100	10	220	FA3	2.3	1.9	265	118	218	12 (ii)
305x165x46 {339 kN}	3	126	2	126	2	100	10	220	FA3	1.9	1.6	260	118	185	12 (ii)
305x165x40 {300 kN}	3	113	2	113	2	100	10	220	FA3	1.7	1.4	254	118	166	12 (ii)
305x127x48 {462 kN}	3	169	2	169	2	100	10	220	FA3	2.6	2.1	251	118	248	12 (ii)
305x127x42 {406 kN}	3	151	2	151	2	100	10	220	FA3	2.3	1.9	245	118	221	12 (ii)
305x127x37 {357 kN}	3	134	2	134	2	100	10	220	FA3	2.0	1.7	241	118	196	12 (ii)
305x102x33 {341 kN}	3	124	2	124	2	100	10	220	FA3	1.9	1.6	251	118	182	12 (ii)
305x102x28 {306 kN}	3	113	2	113	2	100	10	220	FA3	1.7	1.4	241	118	166	12 (ii)
305x102x25 {292 kN}	3	109	2	109	2	100	10	220	FA3	1.7	1.4	229	118	160	12 (ii)
254x146x43 {308 kN}	2	76	2	76	2	100	10	150	FA2	2.5	2.1	270	98*	132	12 (ii)
254x146x37 {266 kN}	2	66	2	66	2	100	10	150	FA2	2.2	1.8	266	98*	116	12 (ii)
254x146x31 {249 kN}	2	63	2	63	2	100	10	150	FA2	2.1	1.7	261	98*	110	12 (ii)
254x102x28 {271 kN}	2	66	2	66	2	100	10	150	FA2	2.2	1.8	270	98*	116	12 (ii)
254x102x25 {255 kN}	2	63	2	63	2	100	10	150	FA2	2.1	1.7	267	98*	110	12 (ii)
254x102x22 {239 kN}	2	60	2	60	2	100	10	150	FA2	2.0	1.6	264	98*	105	12 (ii)

For guidance on the use of tables see Explanatory notes in Table H.24

Note: (7) The minimum capacity from Checks 11(i), 11(ii), 12(i), 12(ii) and 13 (Shear capacity of the bolt group)

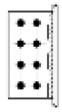
BEAM: S275
FIN PLATE: S275
BOLTS: M20, 8.8

Table H.28



FIN PLATES, ORDINARY BOLTS

Double line of bolts
180x10mm or 150x10mm Fin Plate



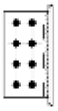
Beam Size { } (1)	Bolt Rows n	Un-notched or Single Notch: Shear(2) Capacity kN	Un-notched or Single Notch: Critical(3) Design Check	Double Notch: Shear(2) Capacity kN	Double Notch: Critical(3) Design Check	Fitting(4) Fin Plate: Width mm	Fitting(4) Fin Plate: t_p mm	Fitting(4) Fin Plate: Length ℓ mm	Fitting(4) Fin Plate: Mark	Min. Support(5) Thickness: S275 mm	Min. Support(5) Thickness: S355 mm	Max. Notch Length(6) $c+t_1$: Single mm	Max. Notch Length(6) $c+t_1$: Double mm	Tying(7): Capacity kN	Tying(7): Critical Check
914x305x253 {2530 kN}	8	762	3 (ii)	762	3 (ii)	180	10	570	FD8	4.5	3.5	928	326*	1300	11 (i)
914x305x224 {2300 kN}	8	762	3 (ii)	762	3 (ii)	180	10	570	FD8	4.5	3.5	786	299*	1300	11 (i)
914x305x201 {2170 kN}	8	762	3 (ii)	762	3 (ii)	180	10	570	FD8	4.5	3.5	686	284*	1300	11 (i)
838x292x226 {2180 kN}	8	762	3 (ii)	762	3 (ii)	180	10	570	FD8	4.5	3.5	861	303*	1300	11 (i)
838x292x194 {1960 kN}	8	762	3 (ii)	762	3 (ii)	180	10	570	FD8	4.5	3.5	729	277*	1300	11 (i)
838x292x176 {1860 kN}	8	762	3 (ii)	762	3 (ii)	180	10	570	FD8	4.5	3.5	640	264*	1300	11 (i)
762x267x197	8	762	3 (ii)	762	3 (ii)	180	10	570	FD8	4.5	3.5	699	294	1300	11 (i)
{1910 kN}	7	665	3 (ii)	665	3 (ii)	180	10	500	FD7	4.5	3.5	780	259*	1140	11 (i)
762x267x173	8	762	3 (ii)	762	3 (ii)	180	10	570	FD8	4.5	3.5	623	269	1300	11 (i)
{1730 kN}	7	665	3 (ii)	665	3 (ii)	180	10	500	FD7	4.5	3.5	714	237*	1140	11 (i)
762x267x147	8	762	3 (ii)	762	3 (ii)	180	10	570	FD8	4.5	3.5	540	220	1300	11 (i)
{1530 kN}	7	665	3 (ii)	665	3 (ii)	180	10	500	FD7	4.5	3.5	600	196*	1140	11 (i)
762x267x134	8	762	3 (ii)	762	3 (ii)	180	10	570	FD8	4.5	3.5	502	203	1300	11 (i)
{1490 kN}	7	665	3 (ii)	665	3 (ii)	180	10	500	FD7	4.5	3.5	502	182*	1140	11 (i)

For guidance on the use of tables see Explanatory notes in Table H.24

Note: (7) The minimum capacity from Checks 11(i), 11(ii), 12(i), 12(ii) and 13 (Shear capacity of the bolt group)

BEAM: S275
FIN PLATE: S275
BOLTS: M20, 8.8

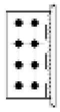
Table H.28 *Continued*



FIN PLATES, ORDINARY BOLTS

Double line of bolts

180x10mm or 150x10mm Fin Plate



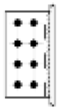
Beam Size { } (1)	Bolt Rows n	Un-notched or Single Notch		Double Notch		Fitting(4) Fin Plate				Min. Support(5) Thickness		Max. Notch Length(6) $c+t_1$		Tying(7)	
		Shear(2) Capacity kN	Critical(3) Design Check	Shear(2) Capacity kN	Critical(3) Design Check	Width mm	t_p mm	Length ℓ mm	Mark	S275 mm	S355 mm	Single mm	Double mm	Capacity kN	Critical Check
686x254x170	8	762	3 (ii)	762	3 (ii)	180	10	570	FD8	4.5	3.5	524	273	1300	11 (i)
{1600 kN}	7	665	3 (ii)	665	3 (ii)	180	10	500	FD7	4.5	3.5	600	241*	1140	11 (i)
	6	568	3 (ii)	568	3 (ii)	180	10	430	FD6	4.4	3.4	703	209*	983	11 (i)
686x254x152	8	762	3 (ii)	762	3 (ii)	180	10	570	FD8	4.5	3.5	469	238	1300	11 (i)
{1440 kN}	7	665	3 (ii)	665	3 (ii)	180	10	500	FD7	4.5	3.5	537	212	1140	11 (i)
	6	568	3 (ii)	568	3 (ii)	180	10	430	FD6	4.4	3.4	629	186*	983	11 (i)
686x254x140	8	762	3 (ii)	762	3 (ii)	180	10	570	FD8	4.5	3.5	434	201	1290	12 (i)
{1350 kN}	7	665	3 (ii)	665	3 (ii)	180	10	500	FD7	4.5	3.5	497	179	1140	12 (i)
	6	568	3 (ii)	568	3 (ii)	180	10	430	FD6	4.4	3.4	582	158*	979	12 (i)
686x254x125	8	762	3 (ii)	762	3 (ii)	180	10	570	FD8	4.5	3.5	397	164	1220	12 (i)
{1260 kN}	7	665	3 (ii)	665	3 (ii)	180	10	500	FD7	4.5	3.5	455	147	1070	12 (i)
	6	568	3 (ii)	568	3 (ii)	180	10	430	FD6	4.4	3.4	534	131*	924	12 (i)
610x305x238	7	671	3 (ii)	671	3 (ii)	150	10	500	FB7	4.5	3.5	646	303	1140	11 (i)
{1860 kN}	6	574	3 (ii)	574	3 (ii)	150	10	430	FB6	4.5	3.5	646	262*	983	11 (i)
610x305x179	7	671	3 (ii)	671	3 (ii)	150	10	500	FB7	4.5	3.5	476	232	1140	11 (i)
{1390 kN}	6	574	3 (ii)	574	3 (ii)	150	10	430	FB6	4.5	3.5	556	201*	983	11 (i)
610x305x149	7	671	3 (ii)	671	3 (ii)	150	10	500	FB7	4.5	3.5	390	147	1080	12 (i)
{1150 kN}	6	574	3 (ii)	574	3 (ii)	150	10	430	FB6	4.5	3.5	456	130*	932	12 (i)
610x229x140	7	671	3 (ii)	671	3 (ii)	150	10	500	FB7	4.5	3.5	421	204	1140	11 (i)
{1290 kN}	6	574	3 (ii)	574	3 (ii)	150	10	430	FB6	4.5	3.5	492	178*	983	11 (i)
610x229x125	7	671	3 (ii)	671	3 (ii)	150	10	500	FB7	4.5	3.5	376	152	1090	12 (i)
{1160 kN}	6	574	3 (ii)	574	3 (ii)	150	10	430	FB6	4.5	3.5	439	134*	940	12 (i)
610x229x113	7	671	3 (ii)	671	3 (ii)	150	10	500	FB7	4.5	3.5	344	109	1020	12 (i)
{1070 kN}	6	574	3 (ii)	572	4 (ii)	150	10	430	FB6	4.5	3.5	401	100*	877	12 (i)
610x229x101	7	671	3 (ii)	667	4 (ii)	150	10	500	FB7	4.5	3.5	328	100	999	12 (i)
{1040 kN}	6	574	3 (ii)	561	4 (ii)	150	10	430	FB6	4.5	3.5	383	100*	860	12 (i)

For guidance on the use of tables see Explanatory notes in Table H.24

Note: (7) The minimum capacity from Checks 11(i), 11(ii), 12(i), 12(ii) and 13 (Shear capacity of the bolt group)

BEAM: S275
FIN PLATE: S275
BOLTS: M20, 8.8

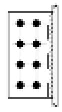
Table H.28 *Continued*



FIN PLATES, ORDINARY BOLTS

Double line of bolts

180x10mm or 150x10mm Fin Plate



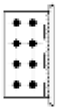
Beam Size	Bolt Rows	Un-notched or Single Notch		Double Notch		Fitting[4] Fin Plate				Min. Support[5] Thickness		Max. Notch Length[6] $c+t_1$		Tying[7]	
{ } [1]	n	Shear[2] Capacity kN	Critical[3] Design Check	Shear[2] Capacity kN	Critical[3] Design Check	Width mm	t_p mm	Length ℓ mm	Mark	S275 mm	S355 mm	Single mm	Double mm	Capacity kN	Critical Check
533x210x122	6	574	3 (ii)	574	3 (ii)	150	10	430	FB6	4.5	3.5	364	164	983	11 (i)
{1100 kN}	5	476	3 (ii)	476	3 (ii)	150	10	360	FB5	4.5	3.5	438	141*	825	11 (i)
533x210x109	6	574	3 (ii)	574	3 (ii)	150	10	430	FB6	4.5	3.5	326	121	916	12 (i)
{995 kN}	5	476	3 (ii)	476	3 (ii)	150	10	360	FB5	4.5	3.5	392	106*	769	12 (i)
533x210x101	6	574	3 (ii)	556	4 (ii)	150	10	430	FB6	4.5	3.5	300	100	853	12 (i)
{922 kN}	5	476	3 (ii)	450	4 (ii)	150	10	360	FB5	4.5	3.5	362	100*	716	12 (i)
533x210x92	6	574	3 (ii)	540	4 (ii)	150	10	430	FB6	4.5	3.5	286	100	828	12 (i)
{888 kN}	5	476	3 (ii)	437	4 (ii)	150	10	360	FB5	4.5	3.5	345	100*	694	12 (i)
533x210x82	6	574	3 (ii)	513	4 (ii)	150	10	430	FB6	4.5	3.5	258	100	787	12 (i)
{837 kN}	5	476	3 (ii)	415	4 (ii)	150	10	360	FB5	4.5	3.5	317	100*	660	12 (i)
457x191x98	5	476	3 (ii)	475	4 (ii)	150	10	360	FB5	4.5	3.5	283	100	755	12 (i)
{847 kN}	4	378	3 (ii)	361	4 (ii)	150	10	290	FB4	4.4	3.4	357	100*	610	12 (i)
457x191x89	5	476	3 (ii)	437	4 (ii)	150	10	360	FB5	4.5	3.5	257	100	696	12 (i)
{774 kN}	4	378	3 (ii)	333	4 (ii)	150	10	290	FB4	4.4	3.4	324	100*	562	12 (i)
457x191x82	5	476	3 (ii)	428	4 (ii)	150	10	360	FB5	4.5	3.5	246	100	681	12 (i)
{751 kN}	4	378	3 (ii)	325	4 (ii)	150	10	290	FB4	4.4	3.4	311	100*	550	12 (i)
457x191x74	5	476	3 (ii)	389	4 (ii)	150	10	360	FB5	4.5	3.5	205	100	619	12 (i)
{679 kN}	4	378	3 (ii)	296	4 (ii)	150	10	290	FB4	4.4	3.4	279	100*	500	12 (i)
457x191x67	5	476	3 (ii)	367	4 (ii)	150	10	360	FB5	4.5	3.5	165	100	584	12 (i)
{636 kN}	4	362	2	279	4 (ii)	150	10	290	FB4	4.2	3.3	269	100*	472	12 (i)

For guidance on the use of tables see Explanatory notes in Table H.24

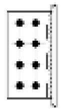
Note: (7) The minimum capacity from Checks 11(i), 11(ii), 12(i), 12(ii) and 13 (Shear capacity of the bolt group)

BEAM: S275
FIN PLATE: S275
BOLTS: M20, 8.8

Table H.28 *Continued*



FIN PLATES, ORDINARY BOLTS
Double line of bolts
180x10mm or 150x10mm Fin Plate



Beam Size	Bolt Rows	Un-notched or Single Notch		Double Notch		Fitting[4] Fin Plate				Min. Support[5] Thickness		Max. Notch Length[6] $c+t_1$		Tying[7]	
{ }[1]	n	Shear[2] Capacity kN	Critical[3] Design Check	Shear[2] Capacity kN	Critical[3] Design Check	Width mm	t_p mm	Length ℓ mm	Mark	S275 mm	S355 mm	Single mm	Double mm	Capacity kN	Critical Check
457x152x82	5	476	3 (ii)	437	4 (ii)	150	10	360	FB5	4.5	3.5	253	100	696	12 (i)
{778 kN}	4	378	3 (ii)	333	4 (ii)	150	10	290	FB4	4.4	3.4	319	100*	562	12 (i)
457x152x74	5	476	3 (ii)	400	4 (ii)	150	10	360	FB5	4.5	3.5	223	100	636	12 (i)
{705 kN}	4	378	3 (ii)	304	4 (ii)	150	10	290	FB4	4.4	3.4	287	100*	514	12 (i)
457x152x67	5	476	3 (ii)	389	4 (ii)	150	10	360	FB5	4.5	3.5	200	100	619	12 (i)
{680 kN}	4	378	3 (ii)	296	4 (ii)	150	10	290	FB4	4.4	3.4	272	100*	500	12 (i)
457x152x60	5	456	4 (ii)	350	4 (ii)	150	10	360	FB5	4.3	3.3	161	100	557	12 (i)
{608 kN}	4	345	2	266	4 (ii)	150	10	290	FB4	4.0	3.1	265	100*	450	12 (i)
457x152x52	5	426	4 (ii)	329	4 (ii)	150	10	360	FB5	4.0	3.1	153	100	523	12 (i)
{564 kN}	4	323	2	250	4 (ii)	150	10	290	FB4	3.8	2.9	255	100*	422	12 (i)
406x178x74 {647 kN}	4	378	3 (ii)	312	4 (ii)	150	10	290	FB4	4.4	3.4	236	100	528	12 (i)
406x178x67 {594 kN}	4	374	2	289	4 (ii)	150	10	290	FB4	4.3	3.4	216	100	489	12 (i)
406x178x60 {530 kN}	4	336	2	260	4 (ii)	150	10	290	FB4	3.9	3.0	214	100	439	12 (i)
406x178x54 {512 kN}	4	328	2	253	4 (ii)	150	10	290	FB4	3.8	2.9	206	100	428	12 (i)
406x140x46 {452 kN}	4	289	2	224	4 (ii)	150	10	290	FB4	3.4	2.6	204	100	378	12 (i)
406x140x39 {420 kN}	4	272	2	210	4 (ii)	150	10	290	FB4	3.2	2.4	194	100	356	12 (i)
356x171x67 {546 kN}	3	251	2	202	4 (ii)	150	10	220	FB3	3.8	3.2	258	100*	385	12 (i)
356x171x57 {478 kN}	3	223	2	180	4 (ii)	150	10	220	FB3	3.4	2.8	249	100*	343	12 (i)
356x171x51 {433 kN}	3	204	2	164	4 (ii)	150	10	220	FB3	3.1	2.6	244	100*	313	12 (i)
356x171x45 {406 kN}	3	193	2	155	4 (ii)	150	10	220	FB3	3.0	2.4	236	100*	296	12 (i)
356x127x39 {385 kN}	3	182	2	146	4 (ii)	150	10	220	FB3	2.8	2.3	232	100*	280	12 (i)
356x127x33 {346 kN}	3	165	2	133	4 (ii)	150	10	220	FB3	2.5	2.1	223	100*	254	12 (i)

For guidance on the use of tables see Explanatory notes in Table H.24

Note: (7) The minimum capacity from Checks 11(i), 11(ii), 12(i), 12(ii) and 13 (Shear capacity of the bolt group)

Incorporating corrigendum 1, 2002 and corrigendum 2, 2005 (see page 491)

BEAM: S275
FIN PLATE: S275
BOLTS: M20, 8.8

Table H.28 *Continued*

FIN PLATES, ORDINARY BOLTS
Double line of bolts
180x10mm or 150x10mm Fin Plate

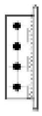
Beam Size { } (1)	Bolt Rows n	Un-notched or Single Notch: Shear(2) Capacity kN	Un-notched or Single Notch: Critical(3) Design Check	Double Notch: Shear(2) Capacity kN	Double Notch: Critical(3) Design Check	Fitting(4) Fin Plate: Width mm	Fitting(4) Fin Plate: t_p mm	Fitting(4) Fin Plate: Length ℓ mm	Fitting(4) Fin Plate: Mark	Min. Support(5) Thickness: S275 mm	Min. Support(5) Thickness: S355 mm	Max. Notch Length(6) $c+t_1$: Single mm	Max. Notch Length(6) $c+t_1$: Double mm	Tying(7): Capacity kN	Tying(7): Critical Check
305x165x54 {405 kN}	3	218	2	175	4 (ii)	150	10	220	FB3	3.3	2.8	181	100	335	12 (i)
305x165x46 {339 kN}	3	185	2	149	4 (ii)	150	10	220	FB3	2.8	2.3	178	100	284	12 (i)
305x165x40 {300 kN}	3	165	2	133	4 (ii)	150	10	220	FB3	2.5	2.1	173	100	254	12 (i)
305x127x48 {462 kN}	3	248	2	200	4 (ii)	150	10	220	FB3	3.8	3.1	172	100	381	12 (i)
305x127x42 {406 kN}	3	220	2	177	4 (ii)	150	10	220	FB3	3.4	2.8	167	100	339	12 (i)
305x127x37 {357 kN}	3	196	2	158	4 (ii)	150	10	220	FB3	3.0	2.5	164	100	301	12 (i)
305x102x33 {341 kN}	3	182	2	146	4 (ii)	150	10	220	FB3	2.8	2.3	172	100	280	12 (i)
305x102x28 {306 kN}	3	165	2	133	4 (ii)	150	10	220	FB3	2.5	2.1	165	100	254	12 (i)
305x102x25 {292 kN}	3	160	2	129	4 (ii)	150	10	220	FB3	2.4	2.0	157	100	246	12 (i)
254x146x43 {308 kN}	2	108	2	74	4 (ii)	150	10	150	FB2	3.5	2.9	218	100*	210	12 (i)
254x146x37 {266 kN}	2	94	2	65	4 (ii)	150	10	150	FB2	3.1	2.6	212	100*	184	12 (i)
254x146x31 {249 kN}	2	90	2	62	4 (ii)	150	10	150	FB2	2.9	2.4	200	100*	175	12 (i)
254x102x28 {271 kN}	2	94	2	65	4 (ii)	150	10	150	FB2	3.1	2.6	208	100*	184	12 (i)
254x102x25 {255 kN}	2	90	2	62	4 (ii)	150	10	150	FB2	2.9	2.4	200	100*	175	12 (i)
254x102x22 {239 kN}	2	85	2	59	4 (ii)	150	10	150	FB2	2.8	2.3	190	100*	166	12 (i)

For guidance on the use of tables see Explanatory notes in Table H.24

Note: (7) The minimum capacity from Checks 11(i), 11(ii), 12(i), 12(ii) and 13 (Shear capacity of the bolt group)

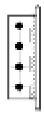
BEAM: S355
FIN PLATE: S275
BOLTS: M20, 8.8

Table H.29



FIN PLATES, ORDINARY BOLTS

Single line of bolts
120x10mm or 100x10mm Fin Plate



Beam Size	Bolt Rows	Un-notched or Single Notch		Double Notch		Fitting[4] Fin Plate				Min. Support[5] Thickness		Max. Notch Length[6] $c+t_1$		Tying[7]	
{ } [1]	n	Shear[2] Capacity kN	Critical[3] Design Check	Shear[2] Capacity kN	Critical[3] Design Check	Width mm	t_p mm	Length ℓ mm	Mark	S275 mm	S355 mm	Single mm	Double mm	Capacity kN	Critical Check
914x305x253 {3290 kN}	8	639	2	639	2	120	10	570	FC8	3.8	2.9	685	506*	735	13
914x305x224 {3000 kN}	8	639	2	639	2	120	10	570	FC8	3.8	2.9	543	465*	735	13
914x305x201 {2820 kN}	8	639	2	639	2	120	10	570	FC8	3.8	2.9	474	441*	735	13
838x292x226 **{2840 kN}**	8	639	2	639	2	120	10	570	FC8	3.8	2.9	644	471*	735	13
838x292x194 **{2560 kN}**	8	639	2	639	2	120	10	570	FC8	3.8	2.9	504	430*	735	13
838x292x176 {2420 kN}	8	639	2	639	2	120	10	570	FC8	3.8	2.9	443	409*	735	13
762x267x197	8	639	2	639	2	120	10	570	FC8	3.8	2.9	715	456	735	13
{2490 kN}	7	542	2	542	2	120	10	500	FC7	3.6	2.8	715	414*	643	13
762x267x173	8	639	2	639	2	120	10	570	FC8	3.8	2.9	564	418	735	13
{2260 kN}	7	542	2	542	2	120	10	500	FC7	3.6	2.8	564	379*	643	13
762x267x147	8	639	2	639	2	120	10	570	FC8	3.8	2.9	416	374	735	13
{2000 kN}	7	542	2	542	2	120	10	500	FC7	3.6	2.8	416	340*	643	13
762x267x134	8	639	2	639	2	120	10	570	FC8	3.8	2.9	348	348	735	13
{1920 kN}	7	542	2	542	2	120	10	500	FC7	3.6	2.8	348	328*	643	13

For guidance on the use of tables see Explanatory notes in Table H.24

Note: (7) The minimum capacity from Checks 11(i), 11(ii), 12(i), 12(ii) and 13 (Shear capacity of the bolt group)

Incorporating corrigendum 1, 2002 and corrigendum 2, 2005 (see page 491)

BEAM: S355
FIN PLATE: S275
BOLTS: M20, 8.8

Table H.29 *Continued*

FIN PLATES, ORDINARY BOLTS

Single line of bolts
120x10mm or 100x10mm Fin Plate

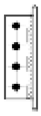
Beam Size { } (1)	Bolt Rows n	Un-notched or Single Notch: Shear(2) Capacity kN	Un-notched or Single Notch: Critical(3) Design Check	Double Notch: Shear(2) Capacity kN	Double Notch: Critical(3) Design Check	Fitting(4) Fin Plate: Width mm	Fitting(4) Fin Plate: t_p mm	Fitting(4) Fin Plate: Length ℓ mm	Fitting(4) Fin Plate: Mark	Min. Support(5) Thickness: S275 mm	Min. Support(5) Thickness: S355 mm	Max. Notch Length(6) $c+t_1$: Single mm	Max. Notch Length(6) $c+t_1$: Double mm	Tying(7): Capacity kN	Tying(7): Critical Check
686x254x170	8	639	2	639	2	120	10	570	FC8	3.8	2.9	703	424	735	13
{2080 kN}	7	542	2	542	2	120	10	500	FC7	3.6	2.8	703	385*	643	13
	6	445	2	445	2	120	10	430	FC6	3.5	2.7	703	347*	551	13
686x254x152	8	639	2	639	2	120	10	570	FC8	3.8	2.9	545	386	735	13
{1880 kN}	7	542	2	542	2	120	10	500	FC7	3.6	2.8	545	350	643	13
	6	445	2	445	2	120	10	430	FC6	3.5	2.7	545	315*	551	13
686x254x140	8	639	2	639	2	120	10	570	FC8	3.8	2.9	459	363	735	13
{1750 kN}	7	542	2	542	2	120	10	500	FC7	3.6	2.8	459	329	643	13
	6	445	2	445	2	120	10	430	FC6	3.5	2.7	459	296*	551	13
686x254x125	8	639	2	639	2	120	10	570	FC8	3.8	2.9	393	342	735	13
{1640 kN}	7	542	2	542	2	120	10	500	FC7	3.6	2.8	393	310	643	13
	6	445	2	445	2	120	10	430	FC6	3.5	2.7	393	280*	551	13
610x305x238	7	568	2	568	2	100	10	500	FA7	3.8	3.0	646	466	643	13
{2420 kN}	6	471	2	471	2	100	10	430	FA6	3.7	2.9	646	416*	551	13
610x305x179	7	568	2	568	2	100	10	500	FA7	3.8	3.0	630	357	643	13
{1810 kN}	6	471	2	471	2	100	10	430	FA6	3.7	2.9	630	318*	551	13
610x305x149	7	568	2	568	2	100	10	500	FA7	3.8	3.0	492	299	643	13
{1500 kN}	6	471	2	471	2	100	10	430	FA6	3.7	2.9	492	266*	551	13
610x229x140	7	568	2	568	2	100	10	500	FA7	3.8	3.0	627	332	643	13
{1670 kN}	6	471	2	471	2	100	10	430	FA6	3.7	2.9	627	296*	551	13
610x229x125	7	568	2	568	2	100	10	500	FA7	3.8	3.0	505	301	643	13
{1510 kN}	6	471	2	471	2	100	10	430	FA6	3.7	2.9	505	269*	551	13
610x229x113	7	568	2	568	2	100	10	500	FA7	3.8	3.0	417	281	643	13
{1400 kN}	6	471	2	471	2	100	10	430	FA6	3.7	2.9	417	251*	551	13
610x229x101	7	568	2	568	2	100	10	500	FA7	3.8	3.0	361	274	643	13
{1350 kN}	6	471	2	471	2	100	10	430	FA6	3.7	2.9	361	244*	551	13

For guidance on the use of tables see Explanatory notes in Table H.24

Note: (7) The minimum capacity from Checks 11(i), 11(ii), 12(i), 12(ii) and 13 (Shear capacity of the bolt group)

BEAM: S355
FIN PLATE: S275
BOLTS: M20, 8.8

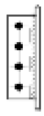
Table H.29 *Continued*



FIN PLATES, ORDINARY BOLTS

Single line of bolts

120x10mm or 100x10mm Fin Plate



Beam Size	Bolt Rows	Un-notched or Single Notch		Double Notch		Fitting[(4)] Fin Plate				Min. Support[(5)] Thickness		Max. Notch Length[(6)] $c+t_1$		Tying[(7)]	
Beam Size { } [(1)]	n	Shear[(2)] Capacity kN	Critical[(3)] Design Check	Shear[(2)] Capacity kN	Critical[(3)] Design Check	Width mm	t_p mm	Length ℓ mm	Mark	S275 mm	S355 mm	Single mm	Double mm	Capacity kN	Critical Check
533x210x122	6	471	2	471	2	100	10	430	FA6	3.7	2.9	555	287	551	13
{1430 kN}	5	374	2	374	2	100	10	360	FA5	3.5	2.7	555	253*	459	13
533x210x109	6	471	2	471	2	100	10	430	FA6	3.7	2.9	517	262	551	13
{1300 kN}	5	374	2	374	2	100	10	360	FA5	3.5	2.7	550	231*	459	13
533x210x101	6	471	2	471	2	100	10	430	FA6	3.7	2.9	476	244	551	13
{1200 kN}	5	374	2	374	2	100	10	360	FA5	3.5	2.7	491	215*	459	13
533x210x92	6	471	2	471	2	100	10	430	FA6	3.7	2.9	409	235	551	13
{1150 kN}	5	374	2	374	2	100	10	360	FA5	3.5	2.7	409	207*	459	13
533x210x82	6	471	2	471	2	100	10	430	FA6	3.7	2.9	359	223	551	13
{1080 kN}	5	374	2	374	2	100	10	360	FA5	3.5	2.7	359	197*	459	13
457x191x98	5	374	2	374	2	100	10	360	FA5	3.5	2.7	469	227	459	13
{1100 kN}	4	279	2	279	2	100	10	290	FA4	3.2	2.5	477	197*	368	13
457x191x89	5	374	2	374	2	100	10	360	FA5	3.5	2.7	425	209	459	13
{1010 kN}	4	279	2	279	2	100	10	290	FA4	3.2	2.5	473	182*	368	13
457x191x82	5	374	2	374	2	100	10	360	FA5	3.5	2.7	405	203	459	13
{970 kN}	4	279	2	279	2	100	10	290	FA4	3.2	2.5	470	176*	368	13
457x191x74	5	374	2	374	2	100	10	360	FA5	3.5	2.7	364	184	459	13
{876 kN}	4	279	2	279	2	100	10	290	FA4	3.2	2.5	394	160*	368	13
457x191x67	5	374	2	374	2	100	10	360	FA5	3.5	2.7	335	172	459	13
{821 kN}	4	279	2	279	2	100	10	290	FA4	3.2	2.5	339	151*	368	13

For guidance on the use of tables see Explanatory notes in Table H.24

Note: (7) The minimum capacity from Checks 11(i), 11(ii), 12(i), 12(ii) and 13 (Shear capacity of the bolt group)

BEAM: S355
FIN PLATE: S275
BOLTS: M20, 8.8

Table H.29 *Continued*

FIN PLATES, ORDINARY BOLTS

Single line of bolts
120x10mm or 100x10mm Fin Plate

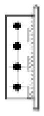
Beam Size	Bolt Rows n	Un-notched or Single Notch		Double Notch		Fitting[4] Fin Plate				Min. Support[5] Thickness		Max. Notch Length[6] $c+t_1$		Tying[7]	
		Shear[2] Capacity	Critical[3] Design Check	Shear[2] Capacity	Critical[3] Design Check	Width	t_p	Length ℓ	Mark	S275	S355	Single	Double	Capacity	Critical Check
{ } [1]		kN		kN		mm	mm	mm		mm	mm	mm	mm	kN	
457x152x82	5	374	2	374	2	100	10	360	FA5	3.5	2.7	419	209	459	13
{1010 kN}	4	279	2	279	2	100	10	290	FA4	3.2	2.5	476	182*	368	13
457x152x74	5	374	2	374	2	100	10	360	FA5	3.5	2.7	377	191	459	13
{918 kN}	4	279	2	279	2	100	10	290	FA4	3.2	2.5	466	166*	368	13
457x152x67	5	374	2	374	2	100	10	360	FA5	3.5	2.7	355	184	459	13
{878 kN}	4	279	2	279	2	100	10	290	FA4	3.2	2.5	392	160*	368	13
457x152x60	5	363	2	363	2	100	10	360	FA5	3.4	2.6	293	165	446	12 (ii)
{784 kN}	4	271	2	271	2	100	10	290	FA4	3.1	2.4	293	149*	356	12 (ii)
457x152x52	5	340	2	340	2	100	10	360	FA5	3.2	2.5	249	165	418	12 (ii)
{728 kN}	4	254	2	254	2	100	10	290	FA4	2.9	2.3	249	149*	334	12 (ii)
406x178x74	4	279	2	279	2	100	10	290	FA4	3.2	2.5	412	169	368	13
{835 kN}															
406x178x67	4	279	2	279	2	100	10	290	FA4	3.2	2.5	374	157	368	13
{767 kN}															
406x178x60	4	264	2	264	2	100	10	290	FA4	3.1	2.4	338	149	348	12 (ii)
{684 kN}															
406x178x54	4	257	2	257	2	100	10	290	FA4	3.0	2.3	320	149	339	12 (ii)
{660 kN}															
406x140x46	4	227	2	227	2	100	10	290	FA4	2.6	2.0	223	149	299	12 (ii)
{584 kN}															
406x140x39	4	214	2	214	2	100	10	290	FA4	2.5	1.9	192	149	282	12 (ii)
{543 kN}															
356x171x67	3	188	2	188	2	100	10	220	FA3	2.9	2.4	373	138*	276	13
{704 kN}															
356x171x57	3	182	2	182	2	100	10	220	FA3	2.8	2.3	368	127*	267	12 (ii)
{618 kN}															
356x171x51	3	167	2	167	2	100	10	220	FA3	2.6	2.1	365	127*	244	12 (ii)
{560 kN}															
356x171x45	3	158	2	158	2	100	10	220	FA3	2.4	2.0	316	127*	231	12 (ii)
{524 kN}															
356x127x39	3	149	2	149	2	100	10	220	FA3	2.3	1.9	263	127*	218	12 (ii)
{497 kN}															
356x127x33	3	135	2	135	2	100	10	220	FA3	2.1	1.7	205	127*	198	12 (ii)
{446 kN}															

For guidance on the use of tables see Explanatory notes in Table H.24

Note: (7) The minimum capacity from Checks 11(i), 11(ii), 12(i), 12(ii) and 13 (Shear capacity of the bolt group)

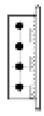
BEAM: S355
FIN PLATE: S275
BOLTS: M20, 8.8

Table H.29 *Continued*



FIN PLATES, ORDINARY BOLTS

Single line of bolts
120x10mm or 100x10mm Fin Plate



Beam Size	Bolt Rows	Un-notched or Single Notch		Double Notch		Fitting(4) Fin Plate				Min. Support(5) Thickness		Max. Notch Length(6) $c+t_1$		Tying(7)	
{ } (1)	n	Shear(2) Capacity kN	Critical(3) Design Check	Shear(2) Capacity kN	Critical(3) Design Check	Width mm	t_p mm	Length ℓ mm	Mark	S275 mm	S355 mm	Single mm	Double mm	Capacity kN	Critical Check
305x165x54 {522 kN}	3	178	2	178	2	100	10	220	FA3	2.7	2.3	286	127	261	12 (ii)
305x165x46 {438 kN}	3	151	2	151	2	100	10	220	FA3	2.3	1.9	280	127	221	12 (ii)
305x165x40 {388 kN}	3	135	2	135	2	100	10	220	FA3	2.1	1.7	268	127	198	12 (ii)
305x127x48 {596 kN}	3	188	2	188	2	100	10	220	FA3	2.9	2.4	292	137	276	13
305x127x42 {523 kN}	3	180	2	180	2	100	10	220	FA3	2.8	2.3	264	127	264	12 (ii)
305x127x37 {460 kN}	3	160	2	160	2	100	10	220	FA3	2.4	2.0	260	127	234	12 (ii)
305x102x33 {440 kN}	3	149	2	149	2	100	10	220	FA3	2.3	1.9	271	127	218	12 (ii)
305x102x28 {395 kN}	3	135	2	135	2	100	10	220	FA3	2.1	1.7	259	127	198	12 (ii)
305x102x25 {377 kN}	3	131	2	131	2	100	10	220	FA3	2.0	1.7	241	127	191	12 (ii)
254x146x43 {398 kN}	2	91	2	91	2	100	10	150	FA2	3.0	2.5	270	106*	158	12 (ii)
254x146x37 {344 kN}	2	79	2	79	2	100	10	150	FA2	2.6	2.2	266	106*	139	12 (ii)
254x146x31 {321 kN}	2	76	2	76	2	100	10	150	FA2	2.5	2.1	261	106*	132	12 (ii)
254x102x28 {349 kN}	2	79	2	79	2	100	10	150	FA2	2.6	2.2	270	106*	139	12 (ii)
254x102x25 {329 kN}	2	76	2	76	2	100	10	150	FA2	2.5	2.1	267	106*	132	12 (ii)
254x102x22 {308 kN}	2	72	2	72	2	100	10	150	FA2	2.3	2.0	264	106*	125	12 (ii)

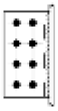
For guidance on the use of tables see Explanatory notes in Table H.24

Note: (7) The minimum capacity from Checks 11(i), 11(ii), 12(i), 12(ii) and 13 (Shear capacity of the bolt group)

Incorporating corrigendum 1, 2002 and corrigendum 2, 2005 (see page 491)

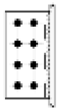
BEAM: S355
FIN PLATE: S275
BOLTS: M20, 8.8

Table H.30



FIN PLATES, ORDINARY BOLTS

Double line of bolts
180x10mm or 150x10mm Fin Plate



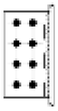
Beam Size	Bolt Rows	Un-notched or Single Notch		Double Notch		Fitting[4] Fin Plate				Min. Support[5] Thickness		Max. Notch Length[6] $c+t_1$		Tying[7]	
{ } [1]	n	Shear[2] Capacity kN	Critical[3] Design Check	Shear[2] Capacity kN	Critical[3] Design Check	Width mm	t_p mm	Length ℓ mm	Mark	S275 mm	S355 mm	Single mm	Double mm	Capacity kN	Critical Check
914x305x253	8	762	3 (ii)	762	3 (ii)	180	10	570	FD8	4.5	3.5	685	424*	1300	11 (i)
{3290 kN}															
914x305x224	8	762	3 (ii)	762	3 (ii)	180	10	570	FD8	4.5	3.5	543	390*	1300	11 (i)
{3000 kN}															
914x305x201	8	762	3 (ii)	762	3 (ii)	180	10	570	FD8	4.5	3.5	474	370*	1300	11 (i)
{2820 kN}															
838x292x226	8	762	3 (ii)	762	3 (ii)	180	10	570	FD8	4.5	3.5	644	395*	1300	11 (i)
{2840 kN}															
838x292x194	8	762	3 (ii)	762	3 (ii)	180	10	570	FD8	4.5	3.5	504	360*	1300	11 (i)
{2560 kN}															
838x292x176	8	762	3 (ii)	762	3 (ii)	180	10	570	FD8	4.5	3.5	443	343*	1300	11 (i)
{2420 kN}															
762x267x197	8	762	3 (ii)	762	3 (ii)	180	10	570	FD8	4.5	3.5	715	382	1300	11 (i)
{2490 kN}	7	665	3 (ii)	665	3 (ii)	180	10	500	FD7	4.5	3.5	715	337*	1140	11 (i)
762x267x173	8	762	3 (ii)	762	3 (ii)	180	10	570	FD8	4.5	3.5	564	351	1300	11 (i)
{2260 kN}	7	665	3 (ii)	665	3 (ii)	180	10	500	FD7	4.5	3.5	564	309*	1140	11 (i)
762x267x147	8	762	3 (ii)	762	3 (ii)	180	10	570	FD8	4.5	3.5	416	314	1300	11 (i)
{2000 kN}	7	665	3 (ii)	665	3 (ii)	180	10	500	FD7	4.5	3.5	416	277*	1140	11 (i)
762x267x134	8	762	3 (ii)	762	3 (ii)	180	10	570	FD8	4.5	3.5	348	303	1300	11 (i)
{1920 kN}	7	665	3 (ii)	665	3 (ii)	180	10	500	FD7	4.5	3.5	348	267*	1140	11 (i)

For guidance on the use of tables see Explanatory notes in Table H.24

Note: (7) The minimum capacity from Checks 11(i), 11(ii), 12(i), 12(ii) and 13 (Shear capacity of the bolt group)

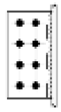
BEAM: S355
FIN PLATE: S275
BOLTS: M20, 8.8

Table H.30 *Continued*



FIN PLATES, ORDINARY BOLTS

Double line of bolts
180x10mm or 150x10mm Fin Plate



Beam Size { } (1)	Bolt Rows n	Un-notched or Single Notch: Shear(2) Capacity kN	Un-notched or Single Notch: Critical(3) Design Check	Double Notch: Shear(2) Capacity kN	Double Notch: Critical(3) Design Check	Fitting(4) Fin Plate: Width mm	Fitting(4) Fin Plate: t_p mm	Fitting(4) Fin Plate: Length ℓ mm	Fitting(4) Fin Plate: Mark	Min. Support(5) Thickness: S275 mm	Min. Support(5) Thickness: S355 mm	Max. Notch Length(6) $c+t_1$: Single mm	Max. Notch Length(6) $c+t_1$: Double mm	Tying(7): Capacity kN	Tying(7): Critical Check
686x254x170	8	762	3 (ii)	762	3 (ii)	180	10	570	FD8	4.5	3.5	682	355	1300	11 (i)
{2080 kN}	7	665	3 (ii)	665	3 (ii)	180	10	500	FD7	4.5	3.5	703	313*	1140	11 (i)
	6	568	3 (ii)	568	3 (ii)	180	10	430	FD6	4.4	3.4	703	272*	983	11 (i)
686x254x152	8	762	3 (ii)	762	3 (ii)	180	10	570	FD8	4.5	3.5	545	324	1300	11 (i)
{1880 kN}	7	665	3 (ii)	665	3 (ii)	180	10	500	FD7	4.5	3.5	545	285	1140	11 (i)
	6	568	3 (ii)	568	3 (ii)	180	10	430	FD6	4.4	3.4	545	247*	983	11 (i)
686x254x140	8	762	3 (ii)	762	3 (ii)	180	10	570	FD8	4.5	3.5	459	304	1300	11 (i)
{1750 kN}	7	665	3 (ii)	665	3 (ii)	180	10	500	FD7	4.5	3.5	459	268	1140	11 (i)
	6	568	3 (ii)	568	3 (ii)	180	10	430	FD6	4.4	3.4	459	232*	983	11 (i)
686x254x125	8	762	3 (ii)	762	3 (ii)	180	10	570	FD8	4.5	3.5	393	287	1300	11 (i)
{1640 kN}	7	665	3 (ii)	665	3 (ii)	180	10	500	FD7	4.5	3.5	393	253	1140	11 (i)
	6	568	3 (ii)	568	3 (ii)	180	10	430	FD6	4.4	3.4	393	219*	983	11 (i)
610x305x238	7	671	3 (ii)	671	3 (ii)	150	10	500	FB7	4.5	3.5	646	394	1140	11 (i)
{2420 kN}	6	574	3 (ii)	574	3 (ii)	150	10	430	FB6	4.5	3.5	646	341*	983	11 (i)
610x305x179	7	671	3 (ii)	671	3 (ii)	150	10	500	FB7	4.5	3.5	620	302	1140	11 (i)
{1810 kN}	6	574	3 (ii)	574	3 (ii)	150	10	430	FB6	4.5	3.5	630	261*	983	11 (i)
610x305x149	7	671	3 (ii)	671	3 (ii)	150	10	500	FB7	4.5	3.5	492	253	1140	11 (i)
{1500 kN}	6	574	3 (ii)	574	3 (ii)	150	10	430	FB6	4.5	3.5	492	219*	983	11 (i)
610x229x140	7	671	3 (ii)	671	3 (ii)	150	10	500	FB7	4.5	3.5	548	281	1140	11 (i)
{1670 kN}	6	574	3 (ii)	574	3 (ii)	150	10	430	FB6	4.5	3.5	627	243*	983	11 (i)
610x229x125	7	671	3 (ii)	671	3 (ii)	150	10	500	FB7	4.5	3.5	489	255	1140	11 (i)
{1510 kN}	6	574	3 (ii)	574	3 (ii)	150	10	430	FB6	4.5	3.5	505	220*	983	11 (i)
610x229x113	7	671	3 (ii)	671	3 (ii)	150	10	500	FB7	4.5	3.5	417	228	1140	11 (i)
{1400 kN}	6	574	3 (ii)	574	3 (ii)	150	10	430	FB6	4.5	3.5	417	199*	983	11 (i)
610x229x101	7	671	3 (ii)	671	3 (ii)	150	10	500	FB7	4.5	3.5	361	213	1140	11 (i)
{1350 kN}	6	574	3 (ii)	574	3 (ii)	150	10	430	FB6	4.5	3.5	361	186*	983	11 (i)

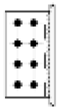
For guidance on the use of tables see Explanatory notes in Table H.24

Note: (7) The minimum capacity from Checks 11(i), 11(ii), 12(i), 12(ii) and 13 (Shear capacity of the bolt group)

Incorporating corrigendum 1, 2002 and corrigendum 2, 2005 (see page 491)

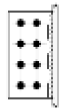
BEAM: S355
FIN PLATE: S275
BOLTS: M20, 8.8

Table H.30 *Continued*



FIN PLATES, ORDINARY BOLTS

Double line of bolts
180x10mm or 150x10mm Fin Plate



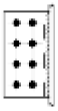
Beam Size { } (1)	Bolt Rows n	Un-notched or Single Notch: Shear(2) Capacity kN	Un-notched or Single Notch: Critical(3) Design Check	Double Notch: Shear(2) Capacity kN	Double Notch: Critical(3) Design Check	Fitting(4) Fin Plate: Width mm	Fitting(4) Fin Plate: t_p mm	Fitting(4) Fin Plate: Length ℓ mm	Fitting(4) Fin Plate: Mark	Min. Support(5) Thickness: S275 mm	Min. Support(5) Thickness: S355 mm	Max. Notch Length(6) $c+t_1$: Single mm	Max. Notch Length(6) $c+t_1$: Double mm	Tying(7): Capacity kN	Tying(7): Critical Check
533x210x122	6	574	3 (ii)	574	3 (ii)	150	10	430	FB6	4.5	3.5	474	235	983	11 (i)
{1430 kN}	5	476	3 (ii)	476	3 (ii)	150	10	360	FB5	4.5	3.5	555	199*	825	11 (i)
533x210x109	6	574	3 (ii)	574	3 (ii)	150	10	430	FB6	4.5	3.5	424	215	983	11 (i)
{1300 kN}	5	476	3 (ii)	476	3 (ii)	150	10	360	FB5	4.5	3.5	511	181*	825	11 (i)
533x210x101	6	574	3 (ii)	574	3 (ii)	150	10	430	FB6	4.5	3.5	391	186	983	11 (i)
{1200 kN}	5	476	3 (ii)	476	3 (ii)	150	10	360	FB5	4.5	3.5	471	160*	825	11 (i)
533x210x92	6	574	3 (ii)	574	3 (ii)	150	10	430	FB6	4.5	3.5	370	167	983	11 (i)
{1150 kN}	5	476	3 (ii)	476	3 (ii)	150	10	360	FB5	4.5	3.5	409	144*	825	11 (i)
533x210x82	6	574	3 (ii)	574	3 (ii)	150	10	430	FB6	4.5	3.5	340	141	983	11 (i)
{1080 kN}	5	476	3 (ii)	476	3 (ii)	150	10	360	FB5	4.5	3.5	359	123*	825	11 (i)
457x191x98	5	476	3 (ii)	476	3 (ii)	150	10	360	FB5	4.5	3.5	369	178	825	11 (i)
{1100 kN}	4	378	3 (ii)	378	3 (ii)	150	10	290	FB4	4.4	3.4	465	146*	667	11 (i)
457x191x89	5	476	3 (ii)	476	3 (ii)	150	10	360	FB5	4.5	3.5	334	148	825	11 (i)
{1010 kN}	4	378	3 (ii)	378	3 (ii)	150	10	290	FB4	4.4	3.4	421	125*	667	11 (i)
457x191x82	5	476	3 (ii)	476	3 (ii)	150	10	360	FB5	4.5	3.5	318	136	825	11 (i)
{970 kN}	4	378	3 (ii)	378	3 (ii)	150	10	290	FB4	4.4	3.4	401	116*	667	11 (i)
457x191x74	5	476	3 (ii)	469	4 (ii)	150	10	360	FB5	4.5	3.5	286	100	799	12 (i)
{876 kN}	4	378	3 (ii)	360	4 (ii)	150	10	290	FB4	4.4	3.4	361	100*	645	12 (i)
457x191x67	5	476	3 (ii)	443	4 (ii)	150	10	360	FB5	4.5	3.5	264	100	754	12 (i)
{821 kN}	4	378	3 (ii)	340	4 (ii)	150	10	290	FB4	4.4	3.4	332	100*	610	12 (i)

For guidance on the use of tables see Explanatory notes in Table H.24

Note: (7) The minimum capacity from Checks 11(i), 11(ii), 12(i), 12(ii) and 13 (Shear capacity of the bolt group)

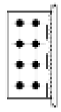
BEAM: S355
FIN PLATE: S275
BOLTS: M20, 8.8

Table H.30 *Continued*



FIN PLATES, ORDINARY BOLTS

Double line of bolts
180x10mm or 150x10mm Fin Plate



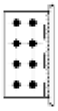
Beam Size { } (1)	Bolt Rows n	Un-notched or Single Notch: Shear(2) Capacity kN	Un-notched or Single Notch: Critical(3) Design Check	Double Notch: Shear(2) Capacity kN	Double Notch: Critical(3) Design Check	Fitting(4) Fin Plate: Width mm	Fitting(4) Fin Plate: t_p mm	Fitting(4) Fin Plate: Length ℓ mm	Fitting(4) Fin Plate: Mark	Min. Support(5) Thickness: S275 mm	Min. Support(5) Thickness: S355 mm	Max. Notch Length(6) $c+t_1$: Single mm	Max. Notch Length(6) $c+t_1$: Double mm	Tying(7): Capacity kN	Tying(7): Critical Check
457x152x82	5	476	3 (ii)	476	3 (ii)	150	10	360	FB5	4.5	3.5	329	148	825	11 (i)
{1010 kN}	4	378	3 (ii)	378	3 (ii)	150	10	290	FB4	4.4	3.4	415	125*	667	11 (i)
457x152x74	5	476	3 (ii)	476	3 (ii)	150	10	360	FB5	4.5	3.5	296	110	825	11 (i)
{918 kN}	4	378	3 (ii)	373	4 (ii)	150	10	290	FB4	4.4	3.4	373	100*	667	11 (i)
457x152x67	5	476	3 (ii)	469	4 (ii)	150	10	360	FB5	4.5	3.5	279	100	799	12 (i)
{878 kN}	4	378	3 (ii)	360	4 (ii)	150	10	290	FB4	4.4	3.4	352	100*	645	12 (i)
457x152x60	5	476	3 (ii)	422	4 (ii)	150	10	360	FB5	4.5	3.5	247	100	719	12 (i)
{784 kN}	4	378	3 (ii)	324	4 (ii)	150	10	290	FB4	4.4	3.4	293	100*	581	12 (i)
457x152x52	5	476	3 (ii)	396	4 (ii)	150	10	360	FB5	4.5	3.5	223	100	675	12 (i)
{728 kN}	4	378	3 (ii)	304	4 (ii)	150	10	290	FB4	4.4	3.4	249	100*	545	12 (i)
406x178x74	4	378	3 (ii)	378	3 (ii)	150	10	290	FB4	4.4	3.4	305	102	667	11 (i)
{835 kN}															
406x178x67	4	378	3 (ii)	352	4 (ii)	150	10	290	FB4	4.4	3.4	277	100	631	12 (i)
{767 kN}															
406x178x60	4	378	3 (ii)	316	4 (ii)	150	10	290	FB4	4.4	3.4	245	100	567	12 (i)
{684 kN}															
406x178x54	4	378	3 (ii)	308	4 (ii)	150	10	290	FB4	4.4	3.4	231	100	552	12 (i)
{660 kN}															
406x140x46	4	346	2	272	4 (ii)	150	10	290	FB4	4.0	3.1	221	100	488	12 (i)
{584 kN}															
406x140x39	4	325	2	256	4 (ii)	150	10	290	FB4	3.8	2.9	192	100	459	12 (i)
{543 kN}															
356x171x67	3	275	2	251	4 (ii)	150	10	220	FB3	4.2	3.5	303	100*	497	12 (i)
{704 kN}															
356x171x57	3	267	2	224	4 (ii)	150	10	220	FB3	4.1	3.4	268	100*	443	12 (i)
{618 kN}															
356x171x51	3	244	2	204	4 (ii)	150	10	220	FB3	3.7	3.1	263	100*	405	12 (i)
{560 kN}															
356x171x45	3	231	2	193	4 (ii)	150	10	220	FB3	3.5	2.9	255	100*	383	12 (i)
{524 kN}															
356x127x39	3	217	2	182	4 (ii)	150	10	220	FB3	3.3	2.7	251	100*	361	12 (i)
{497 kN}															
356x127x33	3	198	2	166	4 (ii)	150	10	220	FB3	3.0	2.5	205	100*	328	12 (i)
{446 kN}															

For guidance on the use of tables see Explanatory notes in Table H.24

Note: (7) The minimum capacity from Checks 11(i), 11(ii), 12(i), 12(ii) and 13 (Shear capacity of the bolt group)

BEAM: S355
FIN PLATE: S275
BOLTS: M20, 8.8

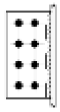
Table H.30 *Continued*



FIN PLATES, ORDINARY BOLTS

Double line of bolts

180x10mm or 150x10mm Fin Plate



Beam Size	Bolt Rows n	Un-notched or Single Notch		Double Notch		Fitting(4) Fin Plate				Min. Support(5) Thickness		Max. Notch Length(6) $c+t_1$		Tying(7)	
{ } (1)		Shear(2) Capacity kN	Critical(3) Design Check	Shear(2) Capacity kN	Critical(3) Design Check	Width mm	t_p mm	Length ℓ mm	Mark	S275 mm	S355 mm	Single mm	Double mm	Capacity kN	Critical Check
305x165x54 {522 kN}	3	260	2	218	4 (ii)	150	10	220	FB3	4.0	3.3	196	100	432	12 (i)
305x165x46 {438 kN}	3	221	2	185	4 (ii)	150	10	220	FB3	3.4	2.8	192	100	366	12 (i)
305x165x40 {388 kN}	3	198	2	166	4 (ii)	150	10	220	FB3	3.0	2.5	187	100	328	12 (i)
305x127x48 {596 kN}	3	275	2	249	4 (ii)	150	10	220	FB3	4.2	3.5	200	100	492	12 (i)
305x127x42 {523 kN}	3	263	2	221	4 (ii)	150	10	220	FB3	4.0	3.3	181	100	437	12 (i)
305x127x37 {460 kN}	3	234	2	196	4 (ii)	150	10	220	FB3	3.6	3.0	178	100	388	12 (i)
305x102x33 {440 kN}	3	217	2	182	4 (ii)	150	10	220	FB3	3.3	2.7	185	100	361	12 (i)
305x102x28 {395 kN}	3	198	2	166	4 (ii)	150	10	220	FB3	3.0	2.5	178	100	328	12 (i)
305x102x25 {377 kN}	3	191	2	160	4 (ii)	150	10	220	FB3	2.9	2.4	169	100	317	12 (i)
254x146x43 {398 kN}	2	129	2	96	4 (ii)	150	10	150	FB2	4.2	3.5	235	100*	271	12 (i)
254x146x37 {344 kN}	2	113	2	84	4 (ii)	150	10	150	FB2	3.7	3.1	229	100*	237	12 (i)
254x146x31 {321 kN}	2	107	2	80	4 (ii)	150	10	150	FB2	3.5	2.9	216	100*	226	12 (i)
254x102x28 {349 kN}	2	113	2	84	4 (ii)	150	10	150	FB2	3.7	3.1	225	100*	237	12 (i)
254x102x25 {329 kN}	2	107	2	80	4 (ii)	150	10	150	FB2	3.5	2.9	215	100*	226	12 (i)
254x102x22 {308 kN}	2	102	2	76	4 (ii)	150	10	150	FB2	3.3	2.8	205	100*	214	12 (i)

For guidance on the use of tables see Explanatory notes in Table H.24

Note: (7) The minimum capacity from Checks 11(i), 11(ii), 12(i), 12(ii) and 13 (Shear capacity of the bolt group)

Table H.31

Explanatory notes - UNIVERSAL COLUMN SPLICES (Bearing Type)
Use of Capacity Tables

The following notes apply to Tables H.32 and H.33 which are for bearing type U.C. splice connections. The design check numbers refer to those listed in Section 7.5 Design procedures.

(1) The ends of the column shaft must be cut to the tolerances required for end bearing as given in BS 5950-2: 2001 [1] or the National Structural Steelwork Specification[8].

(2) Design CHECK 2 for bearing splices must be carried out to see whether the worst combination of axial load and bending leads to net tension.

(3) If net tension is present then it must not exceed the tensile capacities given in the tables.

(4) The tensile capacities given are the minimum from CHECKS 3 and 4. When the flange is thinner than the cover plate, the bearing capacity of the column flange must be checked separately.

(5) In buildings where it is necessary to comply with structural integrity requirements, the tensile capacity of the column splice is twice the tension value given in each table.

(6) The tables are based on the use of M20 or M24 grade 8.8 bolts in clearance holes.

Preloaded HSFG bolts are required when significant net tension exists (i.e. when the net tensile stress of upper member exceeds 10% p_y of the upper column) or if joint slip is unacceptable. Bolt spacings and plate thicknesses shown may not be adequate for HSFG bolts and separate checks must be carried out.

When tension is present from integrity loading only, Grade 8.8 bolts may be used.

(7) The horizontal gauge for the bolts in the column flanges has been selected so that the bolts would be positioned centrally down an internal flange plate if these were used.

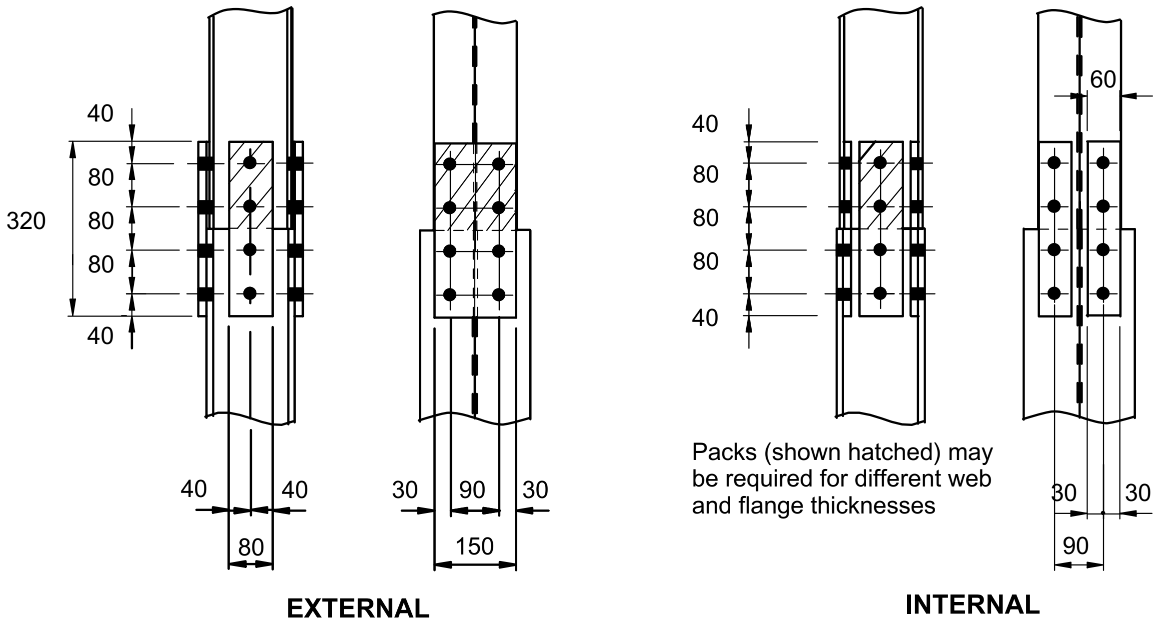
(8) Flange and web cover plates are in S275 steel. The sizes have been rationalised as much as practicable to produce a limited range of widths and thicknesses.

(9) Where hole edge or end distances are less than 1.4 times the hole diameter, cover plates used must have rolled, machine flame cut, sawn or planed edges.

(10) Where tensile capacity values are marked ‡ the thickness of the flange pack exceeds the limit $^4/_3$ d (where 'd' is the bolt diameter). In these cases, the pack must be welded to either the column or the cover plate in order to develop the tensile capacity.

Table H.32

Standard Geometry and Tensile Capacities Upper and Lower Columns 152 UC Series	**UC COLUMN: S275 or S355** **COVER PLATES: S275** **BOLTS: M20, 8.8**



Upper Column	External Cover Plates (mm)	Internal Cover Plates (mm)	Web Cover Plates (mm)
All 152 x 152 UC	150 x 10 x 320	60 x 10 x 320	80 x 6 x 320

	Upper Column All 152 UCs
Lower column All 152 UCs	350kN

Tensile Capacity for one 150 x 10 External Flange Cover Plate

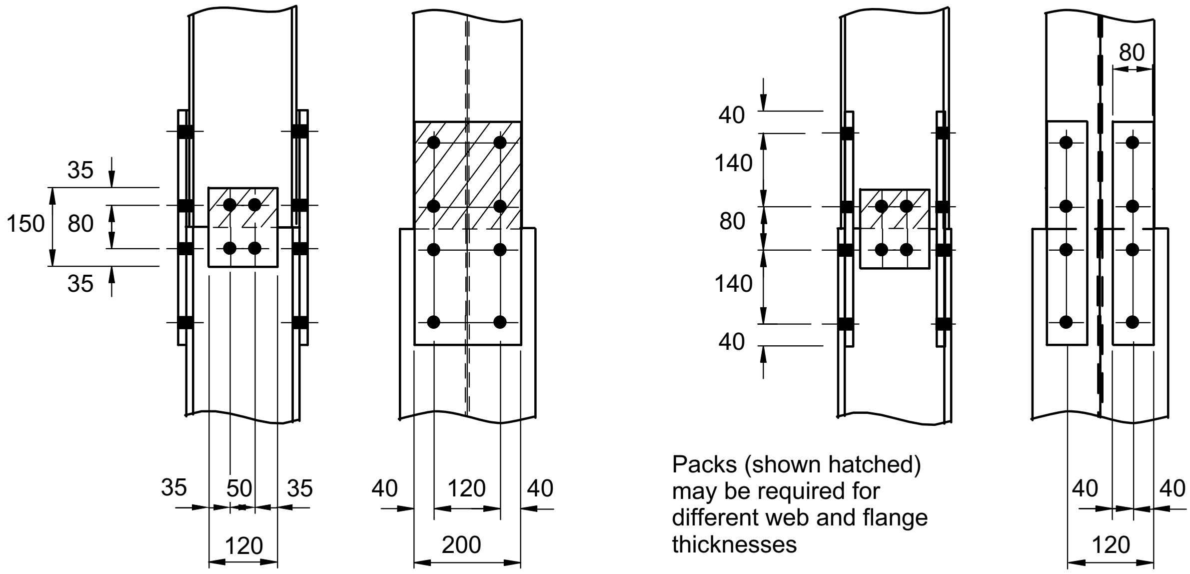
	Upper Column All 152 UCs
Lower column All 152 UCs	251kN

Tensile Capacity for two 60 x 10 Internal Flange Cover Plates

For guidance on the use of tables see Explanatory notes - Table H.31

Table H.32 *Continued*

Standard Geometry and Tensile Capacities Upper and Lower Columns 203 UC Series	**UC COLUMN: S275 or S355** **COVER PLATES: S275** **BOLTS: M20, 8.8**



EXTERNAL **INTERNAL**

Upper Column	External Cover Plates (mm)	Internal Cover Plates (mm)	Web Cover Plates (mm)
All 203 x 203 UC	200 x 10 x 440	80 x 10 x 440	120 x 8 x 150

		Upper Column (kg/m)				
		46	52	60	71	86
Lower column (kg/m)	46	368	-	-	-	-
	52	368	368	-	-	-
	60	368	368	368	-	-
	71	368	368	368	368	-
	86	351	360	368	368	368

Tensile Capacity (kN) for one 200 x 10 External Flange Cover Plate

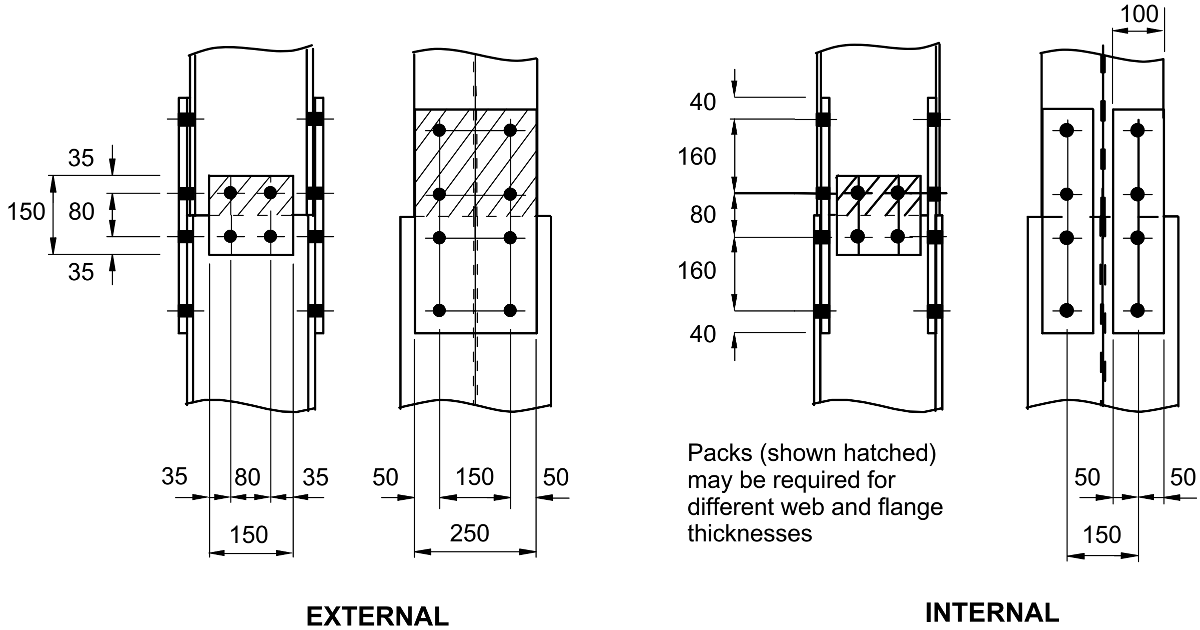
	Upper Column All 203 UCs
Lower column All 203 UCs	368kN

Tensile Capacity (kN) for two 80 x 10 Internal Flange Cover Plates

For guidance on the use of tables see Explanatory notes - Table H.31

Table H.32 *Continued*

Standard Geometry and Tensile Capacities Upper and Lower Columns 254 UC Series	UC COLUMN: S275 or S355 COVER PLATES: S275 BOLTS: M20, 8.8



Upper Column	External Cover Plates (mm)	Internal Cover Plate s (mm)	Web Cover Plates (mm)
254 x 254 x 167UC	250 x 15 x 480	100 x 15 x 480	150 x 8 x 150
254 x 254 x 132UC to 73UC	250 x 12 x 480	100 x 12 x 480	150 x 8 x 150

		Upper Column (kg/m)				
		73	89	107	132	167
Lower column (kg/m)	73	368	-	-	-	-
	89	368	368	-	-	-
	107	368	368	368	-	-
	132	342	360	368	368	-
	167	311	326	342	368	368

Tensile Capacity (kN) for one Standard External Flange Cover Plate

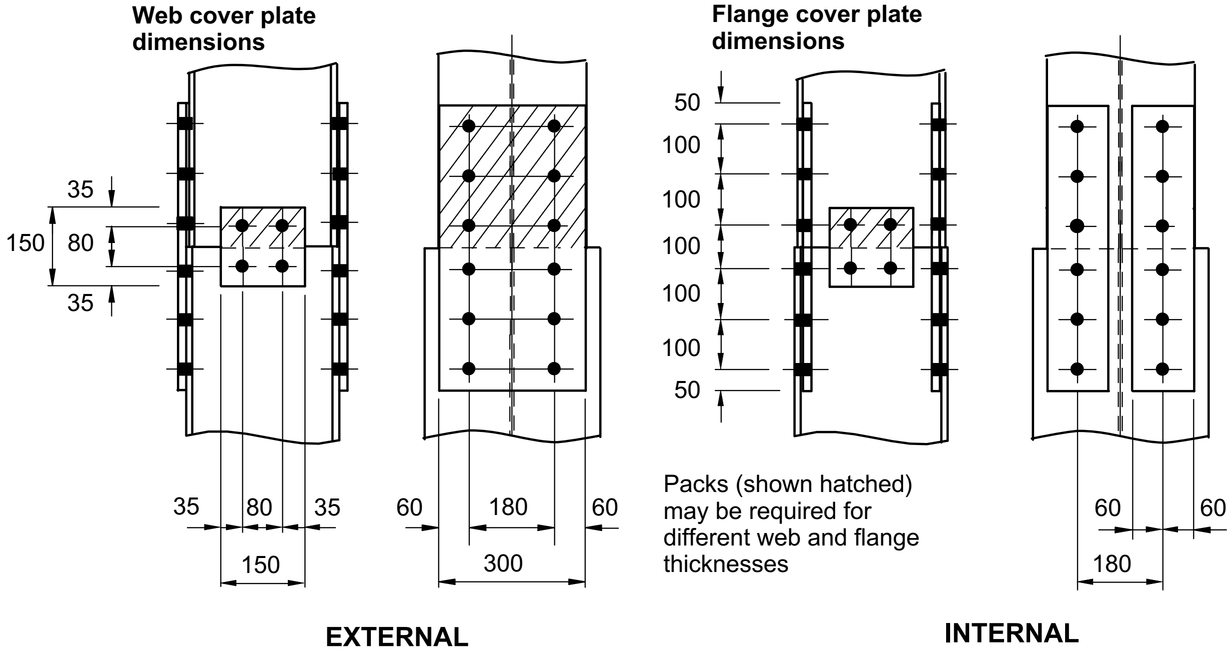
	Upper Column All 254 UCs
Lower column All 254 UCs	368kN

Tensile Capacity for two Standard Internal Flange Cover Plates

For guidance on the use of tables see Explanatory notes - Table H.31

Table H.32 *Continued*

Standard Geometry and Tensile Capacities Upper and Lower Columns 305 UC Series	**UC COLUMN: S275 or S355** **COVER PLATES: S275** **BOLTS: M24, 8.8**



Upper Column	External Cover Plates (mm)	Internal Cover Plates (mm)	Web Cover Plates (mm)
305 x 305UC ≥ 240kg/m	300 x 20 x 600	120 x 20 x 600	150 x 10 x 150
305 x 305UC < 240kg/m	300 x 15 x 600	120 x 15 x 600	150 x 10 x 150

Lower column (kg/m)	Upper Column (kg/m) 97	118	137	158	198	240	283
97	794	-	-	-	-	-	-
118	794	794	-	-	-	-	-
137	794	794	794	-	-	-	-
158	777	794	794	794	-	-	-
198	715	746	776	794	794	-	-
240	663	689	715	746	794	794	-
283	617	640	662	688	746	794	794

Tensile Capacity (kN) for one Standard External Flange Cover Plate

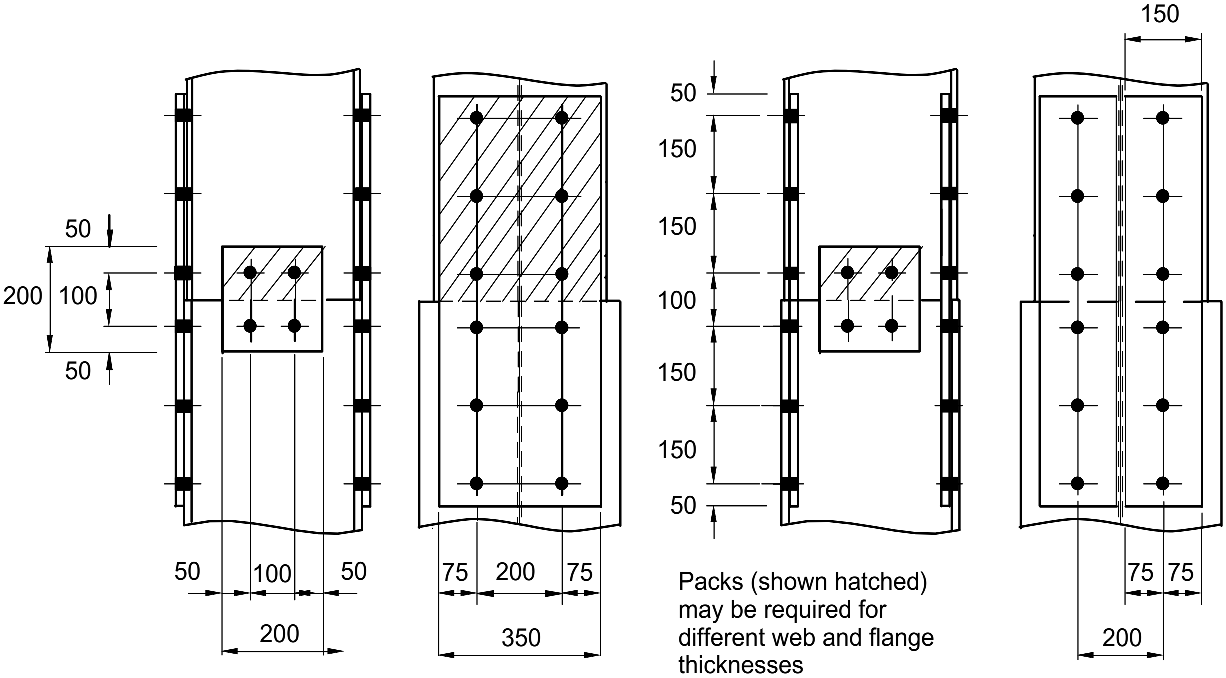
	Upper Column All 305 UCs
Lower column All 305 UCs	794kN

Tensile Capacity for two Standard Internal Flange Cover Plates

For guidance on the use of tables see Explanatory notes - Table H.31

Table H.32 *Continued*

Standard Geometry and Tensile Capacities Upper and Lower Columns 356 x 368 UC Series	UC COLUMN: S275 or S355 COVER PLATES: S275 BOLTS: M24, 8.8



EXTERNAL **INTERNAL**

Upper Column	External Cover Plates (mm)	Internal Cover Plates (mm)	Web Cover Plates (mm)
356 x 368 x 202UC	350 x 15 x 800	150 x 15 x 800	200 x 10 x 200
356 x 368UC < 202kg/m	350 x 12 x 800	150 x 12 x 800	200 x 10 x 200

		Upper Column (kg/m)			
		129	153	177	202
Lower column (kg/m)	129	794	-	-	-
	153	794	794	-	-
	177	794	794	794	-
	202	778	794	794	794

Tensile Capacity (kN) for one Standard External Flange Cover Plate

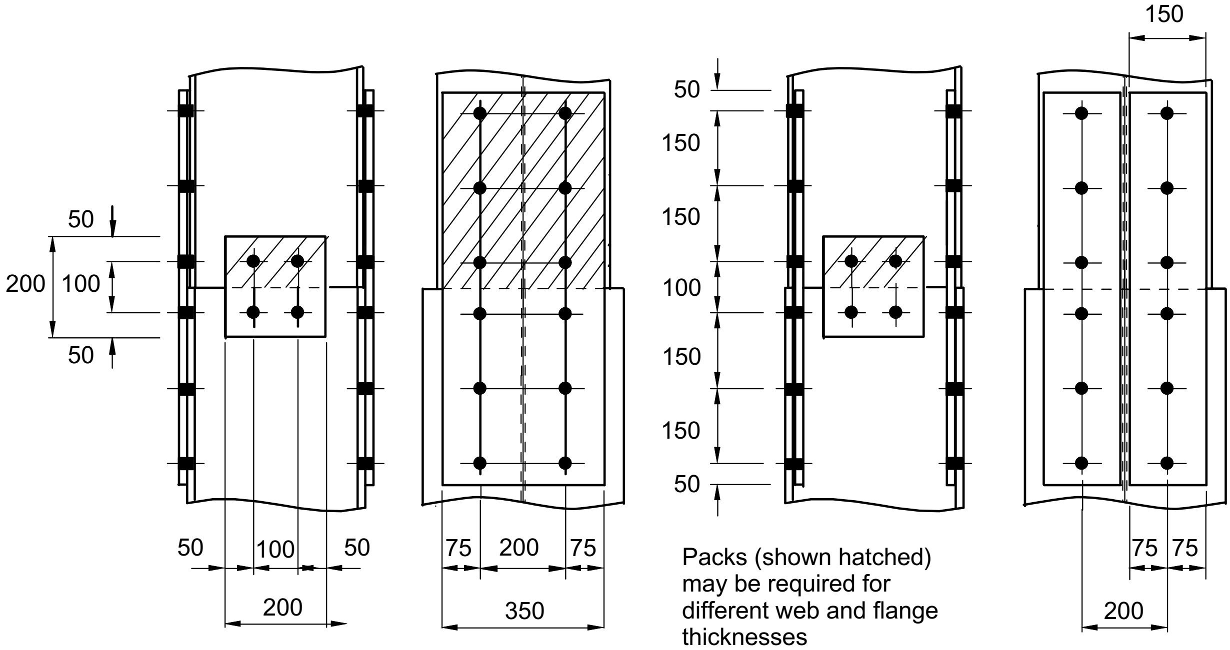
	Upper Column All 356 x 368 UCs
Lower column All 356 x 368 UCs	794kN

Tensile Capacity for two Standard Internal Flange Cover Plates

For guidance on the use of tables see Explanatory notes - Table H.31

Table H.32 *Continued*

Standard Geometry and Tensile Capacities Upper and Lower Columns 356 x 406 UC Series	UC COLUMN: S275 or S355 COVER PLATES: S275 BOLTS: M24, 8.8



Packs (shown hatched) may be required for different web and flange thicknesses

EXTERNAL **INTERNAL**

Upper Column	External Cover Plates (mm)	Internal Cover Plates (mm)	Web Cover Plates (mm))
*356 x 406 x 634UC	350 x 40 x 800	150 x 40 x 800	200 x 15 x 200
*356 x 406 x 551UC	350 x 35 x 800	150 x 35 x 800	200 x 15 x 200
*356 x 406 x 467UC	350 x 30 x 800	150 x 30 x 800	200 x 12 x 200
*356 x 406 x 393UC	350 x 25 x 800	150 x 25 x 800	200 x 12 x 200
*356 x 406 x 340, 287UC	350 x 20 x 800	150 x 20 x 800	200 x 10 x 200
356 x 406 x 235UC	350 x 15 x 800	150 x 15 x 800	200 x 10 x 200

* If standard details are adhered to with these sections, the internal cover plates must be chamfered to clear the root radius, alternatively slightly modified details may be used.

		Upper Column (kg/m)						
		235	287	340	393	467	551	634
Lower column (kg/m)	235	794	-	-	-	-	-	-
	287	794	794	-	-	-	-	-
	340	746	794	794	-	-	-	-
	393	689	746	794	794	-	-	-
	467	623	669	723	786	794	-	-
	551	565	602	645	695	778	794	-
	634	516	547	583	623	689	778	794

Tensile Capacity (kN) for one Standard External Flange Cover Plate

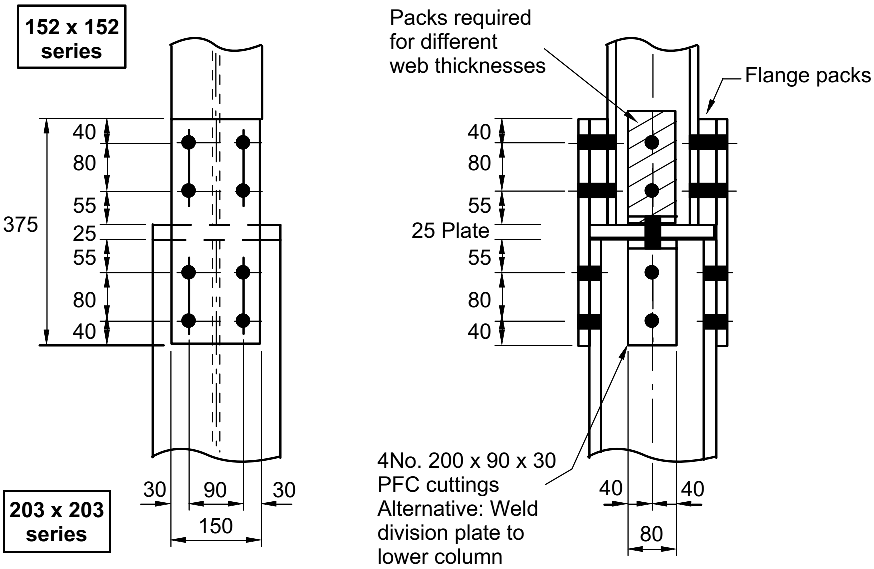
	Upper Column All 356 x 368 UCs
Lower column All 356 x 368 UCs	794kN

Tensile Capacity for two Standard Internal Flange Cover Plates

For guidance on the use of tables see Explanatory notes - Table H.31

Table H.33

Standard Geometry and Tensile Capacities Upper Column 152 UC, Lower Column 203 UC	UC COLUMN: S275 or S355 COVER PLATES: S275 BOLTS: M20, 8.8



Upper Column	Flange Cover Plates (mm)
All 152 x 152 UC	150 x 10 x 375

		Upper Column 152 x 152 UC (kg/m)		
		23	30	37
Lower Column 203 x 203 UC (kg/m)	46	280	290	298
	52	275‡	284	292
	60	269‡	278	285
	71	259‡	267‡	274‡
	86	250‡	257‡	264‡

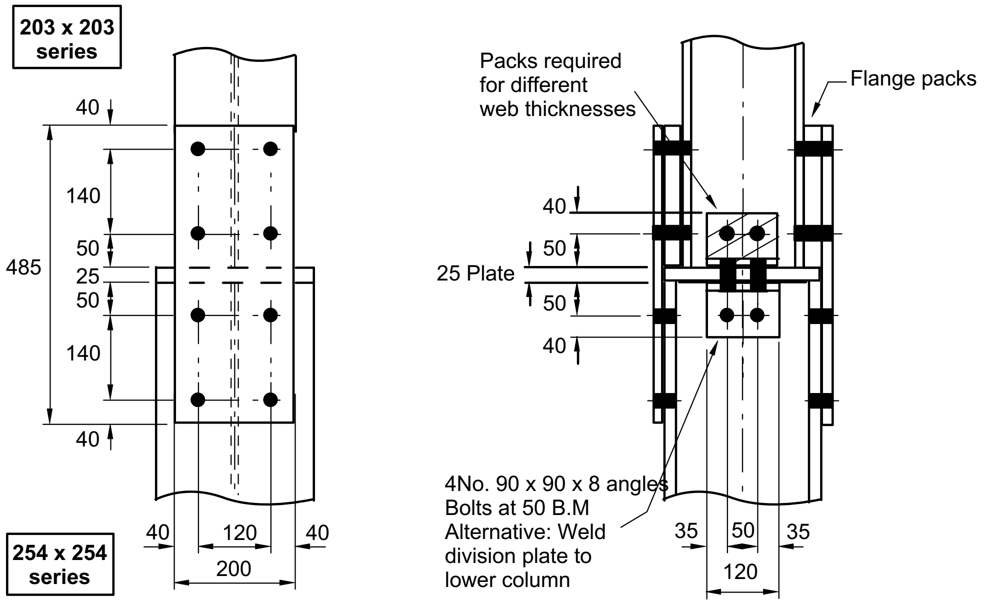
‡ See Note (10), Table H.31

Tensile Capacities (kN) for one Standard Flange Cover Plate

For guidance on the use of tables see Explanatory notes - Table H.31

Table H.33 *Continued*

Standard Geometry and Tensile Capacities Upper Column 203 UC, Lower Column 254 UC	UC COLUMN: S275 or S355 COVER PLATES: S275 BOLTS: M20, 8.8



Upper Column	Flange Cover Plates (mm)
All 203 x 203 UC	200 x 10 x 485

		Upper Column 203 x 203UC (kg/m)				
		46	52	60	71	86
Lower Column 254 x 254 UC (kg/m)	73	280	285	292	304	-
	89	269‡	274‡	280	292	305
	107	259‡	264‡	269‡	280	292
	132	245‡	249‡	254‡	264‡	274‡
	167	229‡	233‡	237‡	245‡	254‡

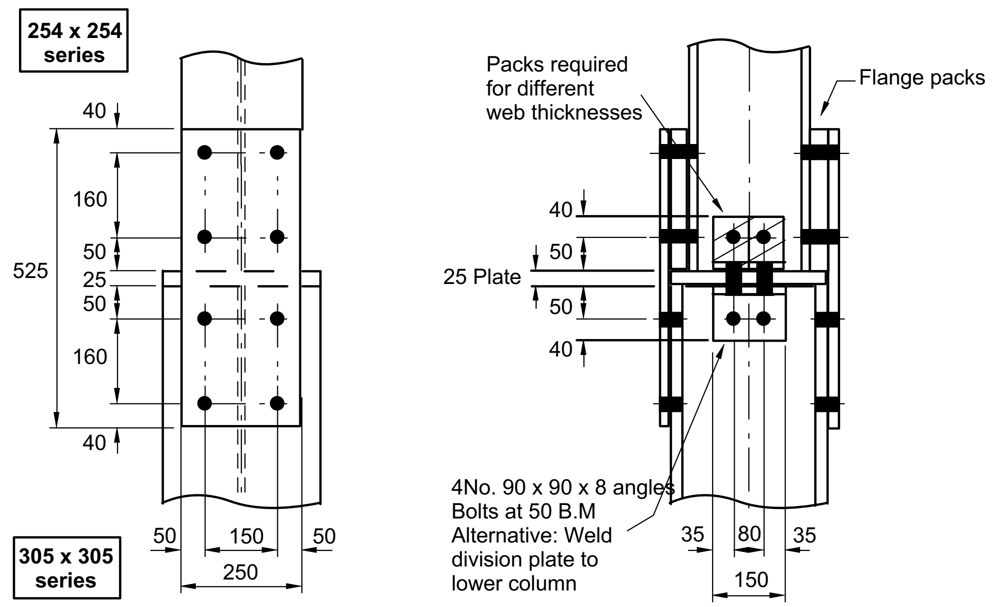
‡ See Note (10), Table H.31

Tensile Capacities (kN) for one Standard Flange Cover Plate

For guidance on the use of tables see Explanatory notes - Table H.31

Table H.33 *Continued*

Standard Geometry and Tensile Capacities **Upper Column 254 UC, Lower Column 305 UC**	**UC COLUMN: S275 or S355** **COVER PLATES: S275** **BOLTS: M20, 8.8**



Upper Column	Flange Cover Plates (mm)
254 x 254UC 167kg/m	250 x 15 x 525
254 x 254UC < 167kg/m	250 x 12 x 525

		Upper Column 254 x 254UC (kg/m)				
		73	89	107	132	167
Lower Column 305 x 305 UC (kg/m)	97	275‡	286	-	-	-
	118	264‡	274‡	285	-	-
	137	255‡	264‡	275‡	292	-
	158	245‡	254‡	264‡	280	-
	198	229‡	237‡	245‡	259‡	280
	240	215‡	222‡	229‡	241‡	259‡
	283	202‡	208‡	215‡	225‡	241‡

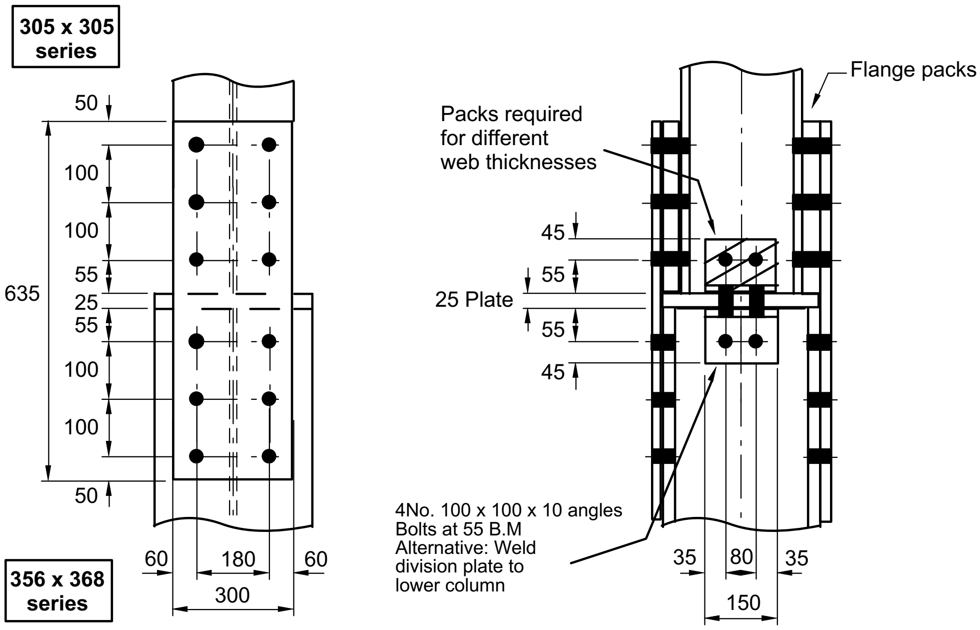
‡ See Note (10), Table H.31

Tensile Capacities (kN) for one Standard Flange Cover Plate

For guidance on the use of tables see Explanatory notes - Table H.31

Table H.33 *Continued*

Standard Geometry and Tensile Capacities **Upper Column 305 UC, Lower Column 356 UC**	**UC COLUMN: S275 or S355** **COVER PLATES: S275** **BOLTS: M24, 8.8**



Upper Column	Flange Cover Plates (mm)
305 x 305UC ≥ 240kg/m	300 x 20 x 635
305 x 305UC < 240kg/m	300 x 15 x 635

		Upper Column 305 x 305UC (kg/m)						
		97	118	137	158	198	240	283
Lower Column 356 x 368 UC (kg/m)	129	651	676	-	-	-	-	-
	153	628	655	675	-	-	-	-
	177	607	629	651	676	-	-	-
	202	587‡	608	628	652	703	-	-

‡ See Note (10), Table H.31

Tensile Capacities (kN) for one Standard Flange Cover Plate

For guidance on the use of tables see Explanatory notes - Table H.31

Table H.34

Explanatory notes - HOLLOW SECTION TENSION SPLICES
Use of capacity tables.

The following notes refer to the **suffix** numbers given at the top of the column descriptions for Tables H.36 to H.38 for hollow section tension splices. The check numbers refer to those listed in Section 7.7 and 7.8 Design procedures.

The connection details are suitable for members in tension.

(1) DETAILING

- All detailing requirements of CHECK 1 from Sections 7.7 and 7.8 are adhered to.
- The capacity tables are based on the standard connection details that are given in Table H.35 for circular, square and rectangular hollow sections.
- Dimension e_1 has been set ≥ 51 mm to allow sufficient tightening access when using a torque wrench. (See Table H.62 where clearance c = 51 mm for M24 bolts

(2) TENSION CAPACITY OF THE CONNECTION

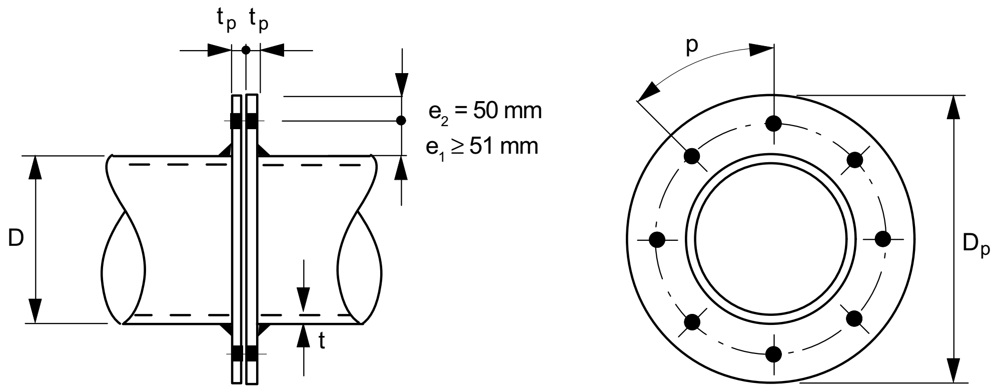
- The tables are based on M24 grade 8.8 bolts and S275 plates.
- Tension capacities tabulated are valid for both S275 and S355 members.
- The tension capacities are conservatively based on the dimensions of the heaviest section available for the given outside dimensions. Lighter sections will have higher connection capacities than those tabulated.
- The tension capacities are based on the minimum of CHECKS 2, 3 and 4 from Section 7.7 for square and rectangular hollow sections and from Section 7.8 for circular hollow sections.

(3) CRITICAL DESIGN CHECK

- The critical design check column indicates whether it is CHECK 2 (Complete end plate yielding) or CHECK 3 (Bolt failure with end plate yielding) which controls the connection capacity.

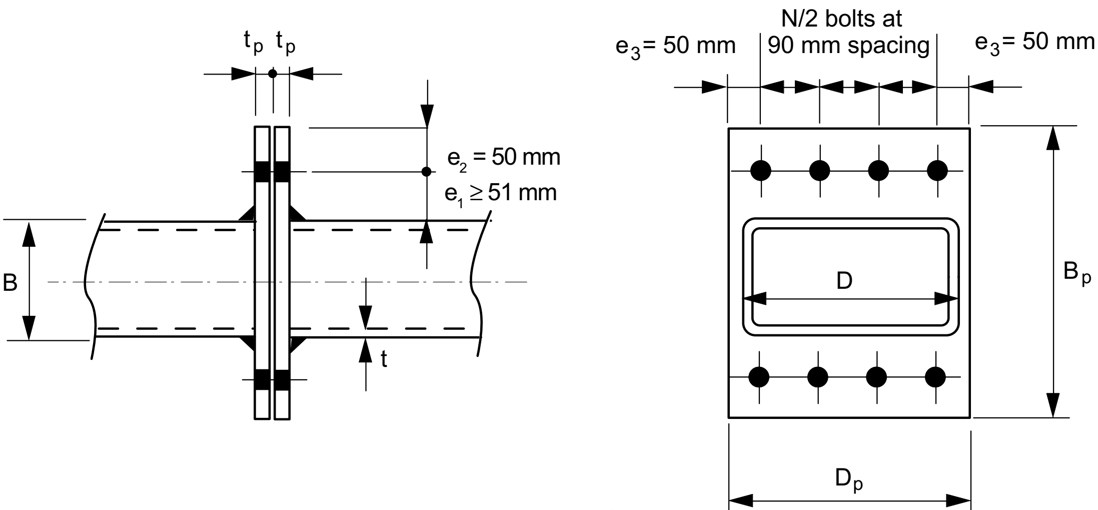
Table H.35

HOLLOW SECTION TENSION SPLICES
Standard details used in Capacity Tables



N = total number of bolts
Bolts: M24 grade 8.8
Plate: S275

CIRCULAR HOLLOW SECTION SPLICE DETAILS

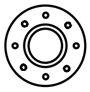


N = total number of bolts
Bolts: M24 grade 8.8
Plate: S275

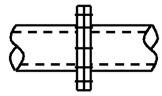
SQUARE AND RECTANGULAR HOLLOW SECTION SPLICE DETAILS

CHS: S275 or S355
PLATES: S275
BOLTS: M24, 8.8

Table H.36



TENSION SPLICES
CIRCULAR HOLLOW SECTIONS

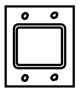


Section Diameter D mm	Total Number of bolts[(1)] N	Bolt spacing[(1)] p mm	Standard Plate[(1)] Diameter D_p mm	Thickness t_p mm	Standard[(1)] Edge distance e_2 mm	Dimension[(1)] e_1 mm	Tension Capacity[(2)] kN	Critical Design Check[(3)]
114.3	4	173	320	12	50	53	197	2
	4	173	320	15	50	53	309 §	2
	4	173	320	20	50	53	531 ‡	3
139.7	4	196	350	15	50	55	330	2
	4	196	350	20	50	55	513	3
	6	131	350	20	50	55	586 §	2
	6	131	350	25	50	55	770 ‡	3
168.3	6	141	370	15	50	51	396	2
	6	141	370	20	50	51	704	2
	6	141	370	25	50	51	790 §	3
	8	106	370	25	50	51	1050 ‡	3
193.7	6	157	400	15	50	53	433	2
	6	157	400	20	50	53	769	2
	8	118	400	25	50	53	1030 §	3
219.1	6	173	430	15	50	55	454	2
	6	173	430	20	50	55	754	3
	8	130	430	25	50	55	1010 §	3
	10	104	430	25	50	55	1260 ‡	3
244.5	6	183	450	15	50	53	511	2
	8	137	450	20	50	53	909	2
	10	110	450	25	50	53	1280	3
	12	92	450	25	50	53	1420	2
273.0	6	199	480	15	50	54	532	2
	8	149	480	20	50	54	946	2
	10	119	480	25	50	54	1250 §	3
	12	99	480	25	50	54	1480 §	2
323.9	8	169	530	20	50	53	1000	3
	10	135	530	25	50	53	1250	3
	12	113	530	25	50	53	1500	3
	14	96	530	25	50	53	1710	2
406.4	8	200	610	20	50	52	1010	3
	10	160	610	25	50	52	1260	3
	14	114	610	25	50	52	1760	3
	16	100	610	25	50	52	2020	3
457.0	10	176	660	20	50	52	1260	3
	14	126	660	25	50	52	1760	3
	16	110	660	25	50	52	2020	3
	20	88	660	25	50	52	2330	2
508.0	12	160	710	20	50	51	1500	3
	16	120	710	25	50	51	1990	3
	18	106	710	25	50	51	2240	3
	22	87	710	25	50	51	2480	2

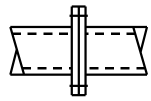
For further information on standard connection details see Table H.35
For guidance on the use of tables see Explanatory notes in Table H.34
§ Check 6 (member capacity) should be checked for S275 sections
‡ Check 6 (member capacity) should be checked for S275 and S355 sections

SHS: S275 or S355
PLATES: S275
BOLTS: M24, 8.8

Table H.37



TENSION SPLICES
SQUARE HOLLOW SECTIONS

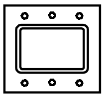


Section Size D x B mm	Total Number of bolts[1] N	Standard Plate[1] Size D_p mm	B_p mm	Thickness t_p mm	Standard[1] Edge distance e_2 mm	Dimension[1] e_1 mm	Tension Capacity[2] kN	Critical Design Check[3]
100 x 100	4	190	310	15	50	55	474 §	3
				20	50	55	510 §	3
				25	50	55	557 ‡	3
120 x 120	4	190	330	15	50	55	464	3
				20	50	55	499	3
				25	50	55	545	3
140 x 140	4	190	350	15	50	55	464	3
				20	50	55	499	3
				25	50	55	545	3
150 x 150	4	190	360	15	50	55	450	3
				20	50	55	485	3
				25	50	55	529	3
160 x 160	4	190	370	15	50	55	464	3
				20	50	55	499	3
				25	50	55	545	3
180 x 180	6	280	390	15	50	55	675	3
				20	50	55	727	3
				25	50	55	794	3
200 x 200	6	280	410	15	50	55	675	3
				20	50	55	727	3
				25	50	55	794	3
250 x 250	6	280	460	15	50	55	675	3
				20	50	55	727	3
				25	50	55	794	3
300 x 300	8	370	510	15	50	55	900	3
				20	50	55	969	3
				25	50	55	1060	3
350 x 350	8	370	560	15	50	55	900	3
				20	50	55	969	3
				25	50	55	1060	3
400 x 400	10	460	610	15	50	55	1090	3
				20	50	55	1170	3
				25	50	55	1280	3

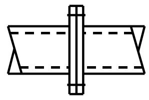
For further information on standard connection details see Table H.35
For guidance on the use of tables see Explanatory notes in Table H.34
§ Check 6 (member capacity) should be checked for S275 sections
‡ Check 6 (member capacity) should be checked for S275 and S355 sections

RHS: S275 or S355
PLATES: S275
BOLTS: M24, 8.8

Table H.38



TENSION SPLICES
RECTANGULAR HOLLOW SECTIONS



Section Size	Total Number of bolts[1]	Standard Plate[1]			Standard[1]		Tension Capacity[2]	Critical Design Check[3]
		Size		Thickness	Edge distance	Dimension[1]		
D x B	N	D_p	B_p	t_p	e_2	e_1		
mm		mm	mm	mm	mm	mm	kN	
200 x 100	6	280	310	15	50	55	695	3
				20	50	55	749	3
				25	50	55	817 §	3
200 x 120	6	280	330	15	50	55	710	3
				20	50	55	765	3
				25	50	55	835	3
200 x 150	6	280	360	15	50	55	710	3
				20	50	55	765	3
				25	50	55	835	3
250 x 100	6	280	310	15	50	55	695	3
				20	50	55	749	3
				25	50	55	817	3
250 x 150	6	280	360	15	50	55	675	3
				20	50	55	727	3
				25	50	55	794	3
300 x 100	8	370	310	15	50	55	947	3
				20	50	55	1020	3
				25	50	55	1110	3
300 x 200	8	370	410	15	50	55	900	3
				20	50	55	969	3
				25	50	55	1060	3
400 x 200	10	460	410	15	50	55	1130	3
				20	50	55	1210	3
				25	50	55	1320	3
450 x 250	12	550	460	15	50	55	1350	3
				20	50	55	1450	3
				25	50	55	1590	3
500 x 300	12	550	510	15	50	55	1310	3
				20	50	55	1410	3
				25	50	55	1540	3

For further information on standard connection details see Table H.35
For guidance on the use of tables see Explanatory notes in Table H.34
§ Check 6 (member capacity) should be checked for S275 sections

Table H.39

Explanatory notes - COLUMN BASES
Use of capacity tables.

The following notes refer to the **suffix** numbers given at the top of the column descriptions for Table H.41 for universal column bases and Tables H.42 to H.44 for hollow section column bases. The check numbers refer to those listed in Section 8.5, Design procedures.

(1) BASE PLATE SIZE

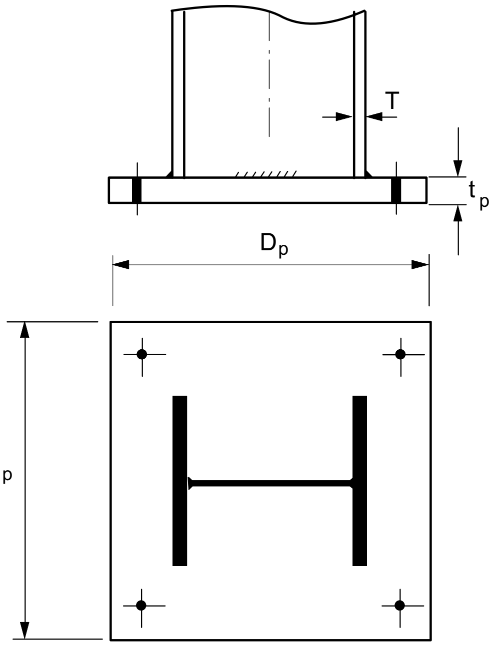
See Table H.40 for standard base plate details.

(2) AXIAL CAPACITY OF BASE PLATE

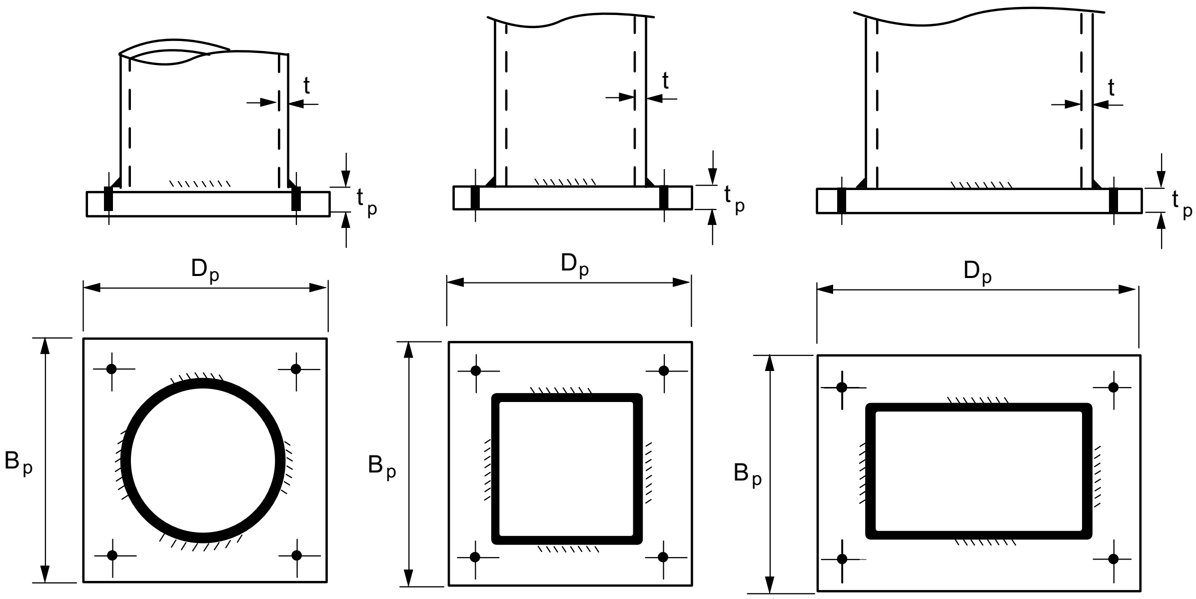
- The capacities tabulated are axial compression capacities based on the minimum capacity from CHECKS 1 and 2 in Section 8.5 (Effective area method).
- The capacities are conservatively based on the dimensions of the lightest section available for the outside dimensions of the section. Heavier sections will have higher base plate capacities than those tabulated.
- Capacities are tabulated for a range of cube strengths of bedding materials/foundation concrete. The capacity based on the cube strength of the weaker material should be used.
- The capacities calculated are for S275 base plates.
- Base plate capacities tabulated are valid for both S275 and S355 columns.

Table H.40

COLUMN BASES
Standard details used in Capacity Tables



UNIVERSAL COLUMN BASE PLATE DETAILS



HOLLOW SECTION COLUMN BASE PLATE DETAILS

COLUMN: S275 or S355

Table H.41

PLATE: S275

COLUMN BASES UNIVERSAL COLUMNS

Column Size D x B mm	Base Plate Size[(1)] $D_p \times B_p \times t_p$ mm	Base Plate Mass kg	Axial Capacity in kN [(2)] for cube strengths f_{cu} 20 N/mm^2	25 N/mm^2	30 N/mm^2	35 N/mm^2	40 N/mm^2
152 x 152 UC	300 x 300 x 15	10.6	566	628	685	737	787
	20	14.1	764	842	913	979	1040
	25	17.7	990	1090	1170	1250	1330
	30	21.2	1080	1330	1450	1540	1630
	350 x 350 x 20	19.2	764	842	913	979	1040
	25	24.0	990	1090	1170	1250	1330
	30	28.9	1190	1330	1450	1540	1630
	35	33.7	1410	1560	1700	1840	1960
203 x 203 UC	400 x 400 x 20	25.1	995	1110	1210	1300	1390
	25	31.4	1270	1400	1530	1640	1750
	30	37.7	1560	1720	1860	2000	2120
	35	44.0	1850	2050	2220	2370	2510
	40	50.2	1920	2370	2590	2760	2920
	450 x 450 x 25	39.7	1270	1400	1530	1640	1750
	30	47.7	1560	1720	1860	2000	2120
	35	55.6	1850	2050	2220	2370	2510
	40	63.6	2120	2370	2590	2760	2920
	45	71.5	2350	2610	2860	3100	3280
	500 x 500 x 25	49.1	1270	1400	1530	1640	1750
	30	58.9	1560	1720	1860	2000	2120
	40	78.5	2120	2370	2590	2760	2920
	50	98.1	2640	2920	3180	3430	3680
	60	118	3000	3580	3870	4150	4420
254 x 254 UC	450 x 450 x 25	39.7	1560	1730	1890	2040	2170
	30	47.7	1900	2100	2290	2460	2620
	35	55.6	2260	2490	2700	2900	3080
	40	63.6	2410	2900	3130	3350	3560
	45	71.5	2430	3000	3510	3750	3970
	500 x 500 x 25	49.1	1560	1730	1890	2040	2170
	30	58.9	1900	2100	2290	2460	2620
	40	78.5	2640	2900	3130	3350	3560
	50	98.1	3000	3630	3960	4230	4470
	60	118	3000	3750	4500	5150	5530
	600 x 600 x 30	84.8	1900	2100	2290	2460	2620
	40	113	2640	2900	3130	3350	3560
	50	141	3250	3630	3960	4230	4470
	60	170	3950	4370	4770	5150	5530
	70	198	4320	5060	5490	5900	6300
	700 x 700 x 40	154	2640	2900	3130	3350	3560
	50	192	3250	3630	3960	4230	4470
	60	231	3950	4370	4770	5150	5530
	70	269	4600	5060	5490	5900	6300
	80	308	5410	5900	6370	6810	7240

For further information on standard connection details see Table H.40

For guidance on the use of tables see Explanatory notes in Table H.39

COLUMN: S275 or S355
PLATE: S275

Table H.41 *Continued*

COLUMN BASES
UNIVERSAL COLUMNS

Column Size D x B mm	Base Plate Size[(1)] $D_p \times B_p \times t_p$ mm	Base Plate Mass kg	Axial Capacity in kN [(2)] for cube strengths f_{cu} 20 N/mm^2	25 N/mm^2	30 N/mm^2	35 N/mm^2	40 N/mm^2
305 x 305 UC	500 x 500 x 25	49.1	1840	2050	2250	2430	2600
	30	58.9	2240	2480	2710	2920	3120
	40	78.5	2800	3380	3680	3950	4200
	50	98.1	2970	3600	4210	4790	5240
	60	118	3000	3750	4430	5060	5680
	600 x 600 x 30	84.8	2240	2480	2710	2920	3120
	40	113	3070	3390	3680	3950	4200
	50	141	3890	4270	4620	4940	5240
	60	170	4320	5260	5700	6080	6430
	70	198	4320	5400	6480	7120	7520
	700 x 700 x 40	154	3070	3390	3680	3950	4200
	50	192	3890	4270	4620	4940	5240
	60	231	4700	5260	5700	6080	6430
	70	269	5410	6010	6580	7120	7520
	80	308	5880	6930	7540	8120	8680
	800 x 800 x 40	201	3070	3390	3680	3950	4200
	50	251	3890	4270	4620	4940	5240
	60	301	4700	5260	5700	6080	6430
	70	352	5410	6010	6580	7120	7520
	80	402	6280	6930	7540	8120	8680
	90	452	7040	7720	8370	8980	9570
356 x 368 UC	600 x 600 x 30	84.8	2630	2920	3200	3460	3700
	40	113	3580	3960	4310	4630	4940
	50	141	4110	4910	5370	5760	6130
	60	170	4320	5240	6130	6940	7480
	70	198	4320	5400	6360	7270	8150
	700 x 700 x 40	154	3580	3960	4310	4630	4940
	50	192	4500	4960	5370	5760	6130
	60	231	5560	6100	6590	7050	7480
	70	269	5880	7110	7700	8210	8690
	80	308	5880	7350	8770	9600	10100
	800 x 800 x 40	201	3580	3960	4310	4630	4940
	50	251	4500	4960	5370	5760	6130
	60	301	5560	6100	6590	7050	7480
	70	352	6340	7110	7700	8210	8690
	80	402	7290	8110	8890	9600	10100
	90	452	7680	8970	9790	10600	11300

For further information on standard connection details see Table H.40
For guidance on the use of tables see Explanatory notes in Table H.39

COLUMN: S275 or S355

Table H.41 *Continued*

PLATE: S275

COLUMN BASES
UNIVERSAL COLUMNS

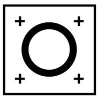
Column Size D x B mm	Base Plate Size(1) D_p x B_p x t_p mm	Base Plate Mass kg	Axial Capacity in kN (2) for cube strengths f_{cu} 20 N/mm^2	25 N/mm^2	30 N/mm^2	35 N/mm^2	40 N/mm^2
356 x 406 UC	800 x 800 x 40	201	3920	4370	4780	5160	5530
(≤ 393 kg/m)	50	251	4890	5410	5890	6340	6770
	60	301	6000	6610	7170	7690	8180
	70	352	6810	7660	8320	8910	9460
	80	402	7660	8690	9560	10400	11000
	90	452	7680	9510	10500	11400	12200
	900 x 900 x 50	318	4890	5410	5890	6340	6770
	60	382	6000	6610	7170	7690	8180
	70	445	6810	7660	8320	8910	9460
	80	509	7780	8690	9560	10400	11000
	90	572	8630	9580	10500	11400	12200
	100	636	9630	10700	11700	12600	13500
	1000 x 1000 x 50	393	4890	5410	5890	6340	6770
	60	471	6000	6610	7170	7690	8180
	70	550	6810	7660	8320	8910	9460
	80	628	7780	8690	9560	10400	11000
	90	707	8630	9580	10500	11400	12200
	100	785	9700	10700	11700	12600	13500
	110	864	10600	11600	12600	13600	14500
	120	942	11700	12800	13900	14900	15800
356 x 406 UC	900 x 900 x 50	318	5290	5890	6450	6980	7480
(>393 kg/m)	60	382	6430	7120	7760	8370	8940
	70	445	7270	8200	8950	9620	10200
	80	509	8270	9270	10200	11100	11800
	90	572	9140	10200	11200	12100	13100
	100	636	9720	11400	12400	13400	14400
	1000 x 1000 x 50	393	5290	5890	6450	6980	7480
	60	471	6430	7120	7760	8370	8940
	70	550	7270	8200	8950	9620	10200
	80	628	8270	9270	10200	11100	11800
	90	707	9140	10200	11200	12100	13100
	100	785	10200	11400	12400	13400	14400
	110	864	11100	12300	13400	14400	15400
	120	942	12000	13500	14700	15800	16800
	1100 x 1100 x 70	665	7270	8200	8950	9620	10200
	80	760	8270	9270	10200	11100	11800
	90	855	9140	10200	11200	12100	13100
	100	950	10200	11400	12400	13400	14400
	110	1040	11100	12300	13400	14400	15400
	120	1140	12300	13500	14700	15800	16800
	1250 x 1250 x 80	981	8270	9270	10200	11100	11800
	90	1100	9140	10200	11200	12100	13100
	100	1230	10200	11400	12400	13400	14400
	120	1470	12300	13500	14700	15800	16800
	140	1720	14900	16200	17400	18600	19800
	150	1840	16200	17600	18900	20200	21400

For further information on standard connection details see Table H.40
For guidance on the use of tables see Explanatory notes in Table H.39

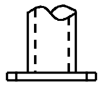
COLUMN: S275 or S355

Table H.42

PLATE: S275



COLUMN BASES
CIRCULAR HOLLOW SECTIONS



Column Size D mm	Base Plate Size[1] D_p x B_p x t_p mm	Base Plate Mass kg	Axial Capacity in kN [2] for cube strengths f_{cu} 20 N/mm^2	25 N/mm^2	30 N/mm^2	35 N/mm^2	40 N/mm^2
139.7	200 x 200 x 15	4.71	356	435	511	586	646
	20	6.28	373	459	542	622	701
	250 x 250 x 15	7.36	446	502	554	601	646
	20	9.81	576	648	713	773	830
	25	12.3	589	735	879	956	1020
	300 x 300 x 20	14.1	576	648	713	773	830
	25	17.7	715	802	882	956	1020
	30	21.2	848	959	1050	1140	1220
	350 x 350 x 20	19.2	576	648	713	773	830
	25	24.0	715	802	882	956	1020
	30	28.8	862	959	1050	1140	1220
168.3	250 x 250 x 15	7.36	535	609	671	729	783
	20	9.81	566	692	815	935	1010
	25	12.3	584	720	852	980	1100
	300 x 300 x 15	10.6	541	609	671	729	783
	20	14.1	699	785	864	938	1010
	25	17.7	843	972	1070	1160	1240
	30	21.2	848	1060	1260	1380	1480
	350 x 350 x 20	19.2	699	785	864	938	1010
	25	24.0	866	972	1070	1160	1240
	30	28.8	1030	1160	1270	1380	1480
	35	33.7	1150	1350	1480	1600	1720
	400 x 400 x 20	25.1	699	785	864	938	1010
	25	31.4	866	972	1070	1160	1240
	30	37.7	1030	1160	1270	1380	1480
	35	44.0	1210	1350	1480	1600	1720
	40	50.2	1400	1550	1690	1820	1950
193.7	300 x 300 x 15	10.6	625	704	776	843	905
	20	14.1	795	908	999	1080	1160
	25	17.7	826	1010	1200	1340	1440
	30	21.2	844	1040	1240	1420	1610
	350 x 350 x 20	19.2	808	908	999	1080	1160
	25	24.0	1000	1120	1240	1340	1440
	30	28.8	1150	1340	1470	1590	1710
	400 x 400 x 20	25.1	808	908	999	1080	1160
	25	31.4	1000	1120	1240	1340	1440
	30	37.7	1190	1340	1470	1590	1710
	35	44.0	1390	1550	1710	1850	1980
	40	50.2	1510	1770	1940	2100	2250
	450 x 450 x 25	39.7	1000	1120	1240	1340	1440
	30	47.7	1190	1340	1470	1590	1710
	35	55.6	1390	1550	1710	1850	1980
	40	63.6	1590	1770	1940	2100	2250
	45	71.5	1770	1960	2140	2320	2480

For further information on standard connection details see Table H.40
For guidance on the use of tables see Explanatory notes in Table H.39

COLUMN: S275 or S355
PLATE: S275

Table H.42 *Continued*

COLUMN BASES
CIRCULAR HOLLOW SECTIONS

Column Size	Base Plate Size[(1)]	Base Plate Mass	Axial Capacity in kN [(2)] for cube strengths f_{cu}				
D mm	D_p x B_p x t_p mm	kg	20 N/mm^2	25 N/mm^2	30 N/mm^2	35 N/mm^2	40 N/mm^2
219.1	350 x 350 x 20	19.2	916	1030	1130	1230	1320
	25	24.0	1100	1270	1400	1520	1630
	30	28.8	1130	1400	1650	1810	1940
	35	33.7	1150	1430	1690	1950	2200
	400 x 400 x 20	25.1	916	1030	1130	1230	1320
	25	31.4	1140	1270	1400	1520	1630
	30	37.7	1350	1520	1670	1810	1940
	35	44.0	1500	1760	1940	2100	2250
	40	50.2	1510	1880	2210	2390	2560
	450 x 450 x 25	39.7	1140	1270	1400	1520	1630
	30	47.7	1350	1520	1670	1810	1940
	35	55.6	1570	1760	1940	2100	2250
	40	63.6	1790	2010	2210	2390	2560
	45	71.5	1910	2210	2430	2630	2810
	500 x 500 x 25	49.1	1140	1270	1400	1520	1630
	30	58.9	1350	1520	1670	1810	1940
	40	78.5	1790	2010	2210	2390	2560
	50	98.1	2220	2460	2690	2910	3120
	60	118	2360	2950	3260	3500	3730
244.5	400 x 400 x 20	25.1	1030	1150	1270	1380	1480
	25	31.4	1270	1430	1570	1700	1820
	30	37.7	1460	1700	1870	2020	2170
	35	44.0	1490	1840	2170	2350	2520
	40	50.2	1510	1870	2220	2560	2860
	450 x 450 x 25	39.7	1270	1430	1570	1700	1820
	30	47.7	1510	1700	1870	2020	2170
	35	55.6	1760	1970	2170	2350	2520
	40	63.6	1910	2250	2470	2670	2860
	45	71.5	1910	2380	2720	2940	3150
	500 x 500 x 25	49.1	1270	1430	1570	1700	1820
	30	58.9	1510	1700	1870	2020	2170
	40	78.5	2000	2250	2470	2670	2860
	50	98.1	2360	2740	3010	3260	3490
	60	118	2360	2950	3530	3890	4170
	600 x 600 x 30	84.8	1510	1700	1870	2020	2170
	40	113	2000	2250	2470	2670	2860
	50	141	2460	2740	3010	3260	3490

For further information on standard connection details see Table H.40
For guidance on the use of tables see Explanatory notes in Table H.39

COLUMN: S275 or S355

PLATE: S275

Table H.42 *Continued*

COLUMN BASES
CIRCULAR HOLLOW SECTIONS

Column Size D mm	Base Plate Size[1] $D_p \times B_p \times t_p$ mm	Base Plate Mass kg	Axial Capacity in kN [2] for cube strengths f_{cu} 20 N/mm^2	25 N/mm^2	30 N/mm^2	35 N/mm^2	40 N/mm^2
273.0	400 x 400 x 20	25.1	1150	1290	1420	1540	1650
	25	31.4	1360	1600	1750	1900	2040
	30	37.7	1410	1720	2020	2260	2430
	35	44.0	1460	1780	2100	2400	2700
	40	50.2	1490	1830	2160	2480	2790
	450 x 450 x 25	39.7	1420	1600	1750	1900	2040
	30	47.7	1700	1900	2090	2260	2430
	35	55.6	1860	2210	2430	2630	2810
	40	63.6	1890	2330	2760	2990	3200
	45	71.5	1900	2360	2800	3230	3520
	500 x 500 x 25	49.1	1420	1600	1750	1900	2040
	30	58.9	1700	1900	2090	2260	2430
	40	78.5	2240	2510	2760	2990	3200
	50	98.1	2360	2940	3370	3650	3900
	60	118	2360	2950	3530	4120	4660
	600 x 600 x 30	84.8	1700	1900	2090	2260	2430
	40	113	2240	2510	2760	2990	3200
	50	141	2740	3070	3370	3650	3900
	60	170	3310	3680	4030	4360	4660
	70	198	3390	4240	4610	4970	5320
	700 x 700 x 40	154	2240	2510	2760	2990	3200
	50	192	2740	3070	3370	3650	3900
	60	231	3310	3680	4030	4360	4660
	70	269	3840	4240	4610	4970	5320
323.9	450 x 450 x 25	39.7	1620	1910	2100	2280	2450
	30	47.7	1700	2060	2410	2710	2910
	35	55.6	1770	2150	2520	2880	3230
	40	63.6	1820	2220	2610	2980	3350
	45	71.5	1860	2270	2670	3060	3440
	500 x 500 x 25	49.1	1700	1910	2100	2280	2450
	30	58.9	2020	2270	2500	2710	2910
	40	78.5	2270	2780	3280	3570	3830
	50	98.1	2340	2880	3410	3920	4430
	60	118	2360	2940	3500	4040	4570
	600 x 600 x 30	84.8	2020	2270	2500	2710	2910
	40	113	2670	3000	3300	3570	3830
	50	141	3260	3660	4020	4350	4660
	60	170	3390	4230	4800	5190	5560
	70	198	3390	4240	5090	5920	6340
	700 x 700 x 40	154	2670	3000	3300	3570	3830
	50	192	3260	3660	4020	4350	4660
	60	231	3900	4370	4800	5190	5560
	70	269	4480	4990	5470	5920	6340
	80	308	4620	5730	6250	6740	7220

For further information on standard connection details see Table H.40
For guidance on the use of tables see Explanatory notes in Table H.39

COLUMN: S275 or S355
PLATE: S275

Table H.42 *Continued*

COLUMN BASES
CIRCULAR HOLLOW SECTIONS

Column Size	Base Plate Size[(1)]	Base Plate Mass	Axial Capacity in kN [(2)] for cube strengths f_{cu}				
D	D_p x B_p x t_p		20	25	30	35	40
mm	mm	kg	N/mm^2	N/mm^2	N/mm^2	N/mm^2	N/mm^2
406.4	500 x 500 x 25	49.1	1730	2070	2400	2720	3040
	30	58.9	1850	2220	2570	2910	3250
	40	78.5	2060	2480	2870	3250	3620
	50	98.1	2200	2660	3090	3510	3920
	60	118	2300	2810	3280	3740	4180
	600 x 600 x 30	84.8	2550	2860	3150	3410	3660
	40	113	3100	3770	4150	4500	4820
	50	141	3240	3960	4650	5330	5870
	60	170	3340	4100	4840	5550	6250
	70	198	3390	4190	4960	5710	6440
	700 x 700 x 40	154	3370	3780	4150	4500	4820
	50	192	4110	4610	5060	5480	5870
	60	231	4570	5500	6040	6540	7000
	70	269	4610	5720	6800	7450	7980
	80	308	4620	5770	6890	7980	9050
	800 x 800 x 40	201	3370	3780	4150	4500	4820
	50	251	4110	4610	5060	5480	5870
	60	301	4910	5500	6040	6540	7000
	70	352	5600	6280	6890	7450	7980
	80	402	6030	7160	7850	8490	9090
457.0	600 x 600 x 30	84.8	2660	3210	3580	3880	4170
	40	113	2920	3520	4110	4670	5230
	50	141	3100	3760	4380	4990	5580
	60	170	3250	3960	4630	5280	5910
	70	198	3340	4090	4800	5490	6160
	700 x 700 x 40	154	3810	4280	4700	5100	5470
	50	192	4330	5210	5720	6200	6640
	60	231	4480	5490	6470	7390	7920
	70	269	4560	5620	6640	7630	8610
	80	308	4610	5720	6790	7820	8840
	800 x 800 x 40	201	3810	4280	4700	5100	5470
	50	251	4640	5210	5720	6200	6640
	60	301	5540	6210	6820	7390	7920
	70	352	5980	7080	7770	8420	9010
	80	402	6030	7490	8860	9580	10300
	900 x 900 x 50	318	4640	5210	5720	6200	6640
	60	382	5540	6210	6820	7390	7920
	70	445	6320	7080	7770	8420	9010

For further information on standard connection details see Table H.40
For guidance on the use of tables see Explanatory notes in Table H.39

COLUMN: S275 or S355
PLATE: S275

Table H.42 *Continued*

COLUMN BASES
CIRCULAR HOLLOW SECTIONS

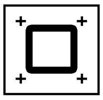
Column Size	Base Plate Size[1]	Base Plate Mass	Axial Capacity in kN [2] for cube strengths f_{cu}				
D mm	D_p x B_p x t_p mm	kg	20 N/mm^2	25 N/mm^2	30 N/mm^2	35 N/mm^2	40 N/mm^2
508.0	700 x 700 x 40	154	3910	4730	5240	5680	6090
	50	192	4140	5010	5860	6690	7400
	60	231	4340	5270	6170	7050	7900
	70	269	4470	5450	6400	7310	8210
	80	308	4570	5610	6600	7570	8510
	800 x 800 x 40	201	4240	4760	5240	5680	6090
	50	251	5170	5800	6370	6900	7400
	60	301	5750	6920	7600	8230	8820
	70	352	5880	7220	8520	9370	10000
	80	402	5980	7370	8720	10000	11300
	90	452	6020	7460	8860	10200	11500
	900 x 900 x 60	382	6170	6920	7600	8230	8820
	70	445	7040	7890	8660	9370	10000
	80	509	7580	8980	9860	10700	11400
	90	572	7620	9470	10800	11700	12600
	100	636	7630	9530	11400	13000	13900

For further information on standard connection details see Table H.40
For guidance on the use of tables see Explanatory notes in Table H.39

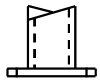
COLUMN: S275 or S355

Table H.43

PLATE: S275



COLUMN BASES
SQUARE HOLLOW SECTIONS

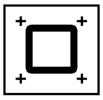


Column Size D x B mm	Base Plate Size[1] $D_p \times B_p \times t_p$ mm	Mass kg	Axial Capacity in kN [2] for cube strengths f_{cu} 20 N/mm²	25 N/mm²	30 N/mm²	35 N/mm²	40 N/mm²
150 x 150	300 x 300 x 15	10.6	612	689	759	824	886
	20	14.1	790	888	977	1060	1140
	25	17.7	979	1100	1210	1310	1400
	30	21.2	1080	1310	1440	1560	1670
	350 x 350 x 20	19.2	790	888	977	1060	1140
	25	24.0	979	1100	1210	1310	1400
	30	28.8	1170	1310	1440	1560	1670
	35	33.7	1390	1530	1680	1810	1940
	400 x 400 x 20	25.1	790	888	977	1060	1140
	25	31.4	979	1100	1210	1310	1400
	30	37.7	1170	1310	1440	1560	1670
	35	44.0	1390	1530	1680	1810	1940
	40	50.2	1620	1780	1930	2070	2210
	450 x 450 x 25	39.7	979	1100	1210	1310	1400
	30	47.7	1170	1310	1440	1560	1670
	35	55.6	1390	1530	1680	1810	1940
	40	63.6	1620	1780	1930	2070	2210
	45	71.5	1820	1990	2150	2300	2450
160 x 160	300 x 300 x 15	10.6	654	736	811	881	947
	20	14.1	845	949	1040	1130	1220
	25	17.7	1050	1170	1290	1400	1500
	30	21.2	1080	1350	1540	1670	1790
	350 x 350 x 20	19.2	845	949	1040	1130	1220
	25	24.0	1050	1170	1290	1400	1500
	30	28.8	1250	1400	1540	1670	1790
	35	33.7	1470	1630	1790	1930	2070
	400 x 400 x 20	25.1	845	949	1040	1130	1220
	25	31.4	1050	1170	1290	1400	1500
	30	37.7	1250	1400	1540	1670	1790
	35	44.0	1470	1630	1790	1930	2070
	40	50.2	1710	1880	2050	2210	2360
	450 x 450 x 25	39.7	1050	1170	1290	1400	1500
	30	47.7	1250	1400	1540	1670	1790
	35	55.6	1470	1630	1790	1930	2070
	40	63.6	1710	1880	2050	2210	2360
	45	71.5	1920	2100	2280	2440	2600

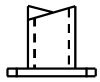
For further information on standard connection details see Table H.40
For guidance on the use of tables see Explanatory notes in Table H.39

COLUMN: S275 or S355
PLATE: S275

Table H.43 *Continued*



COLUMN BASES
SQUARE HOLLOW SECTIONS



Column Size D x B mm	Base Plate Size[1] $D_p \times B_p \times t_p$ mm	Mass kg	Axial Capacity in kN [2] for cube strengths f_{cu} 20 N/mm^2	25 N/mm^2	30 N/mm^2	35 N/mm^2	40 N/mm^2
180 x 180	350 x 350 x 20	19.2	957	1080	1190	1290	1380
	25	24.0	1180	1330	1460	1590	1700
	30	28.8	1410	1580	1740	1890	2020
	35	33.7	1470	1840	2020	2190	2340
	400 x 400 x 20	25.1	957	1080	1190	1290	1380
	25	31.4	1180	1330	1460	1590	1700
	30	37.7	1410	1580	1740	1890	2020
	35	44.0	1640	1840	2020	2190	2340
	40	50.2	1890	2100	2300	2490	2660
	450 x 450 x 25	39.7	1180	1330	1460	1590	1700
	30	47.7	1410	1580	1740	1890	2020
	35	55.6	1640	1840	2020	2190	2340
	40	63.6	1890	2100	2300	2490	2660
	45	71.5	2110	2330	2540	2740	2930
	500 x 500 x 25	49.1	1180	1330	1460	1590	1700
	30	58.9	1410	1580	1740	1890	2020
	40	78.5	1890	2100	2300	2490	2660
	50	98.1	2390	2620	2840	3050	3250
	60	118	2990	3250	3500	3730	3950
200 x 200	400 x 400 x 20	25.1	1060	1190	1310	1430	1530
	25	31.4	1320	1480	1630	1760	1890
	30	37.7	1570	1760	1940	2100	2250
	35	44.0	1820	2050	2250	2430	2610
	40	50.2	1920	2330	2560	2770	2970
	450 x 450 x 25	39.7	1320	1480	1630	1760	1890
	30	47.7	1570	1760	1940	2100	2250
	35	55.6	1820	2050	2250	2430	2610
	40	63.6	2090	2330	2560	2770	2970
	45	71.5	2320	2570	2820	3050	3260
	500 x 500 x 25	49.1	1320	1480	1630	1760	1890
	30	58.9	1570	1760	1940	2100	2250
	40	78.5	2090	2330	2560	2770	2970
	50	98.1	2610	2880	3130	3380	3620
	60	118	3000	3540	3820	4090	4350

For further information on standard connection details see Table H.40
For guidance on the use of tables see Explanatory notes in Table H.39

COLUMN: S275 or S355
PLATE: S275

Table H.43 *Continued*

COLUMN BASES
SQUARE HOLLOW SECTIONS

Column Size D x B mm	Base Plate Size[1] D_p x B_p x t_p mm	Mass kg	Axial Capacity in kN [2] for cube strengths f_{cu} 20 N/mm^2	25 N/mm^2	30 N/mm^2	35 N/mm^2	40 N/mm^2
250 x 250	450 x 450 x 25	39.7	1660	1870	2050	2230	2390
	30	47.7	1980	2220	2440	2650	2840
	35	55.6	2300	2580	2830	3070	3290
	40	63.6	2430	2930	3220	3490	3740
	45	71.5	2430	3030	3540	3840	4110
	500 x 500 x 25	49.1	1660	1870	2050	2230	2390
	30	58.9	1980	2220	2440	2650	2840
	40	78.5	2610	2930	3220	3490	3740
	50	98.1	3000	3570	3920	4250	4550
	60	118	3000	3750	4500	5070	5430
	600 x 600 x 30	84.8	1980	2220	2440	2650	2840
	40	113	2610	2930	3220	3490	3740
	50	141	3200	3570	3920	4250	4550
	60	170	3890	4300	4700	5070	5430
	70	198	4320	4990	5410	5810	6200
300 x 300	500 x 500 x 25	49.1	2000	2250	2480	2690	2880
	30	58.9	2380	2680	2940	3190	3420
	40	78.5	2940	3530	3880	4200	4500
	50	98.1	2990	3710	4410	5090	5480
	60	118	3000	3750	4490	5210	5910
	600 x 600 x 30	84.8	2380	2680	2940	3190	3420
	40	113	3150	3530	3880	4200	4500
	50	141	3840	4310	4730	5120	5480
	60	170	4320	5140	5650	6110	6550
	700 x 700 x 40	154	3150	3530	3880	4200	4500
	50	192	3840	4310	4730	5120	5480
	60	231	4600	5140	5650	6110	6550
	70	269	5310	5890	6440	6970	7460
	80	308	5880	6800	7390	7960	8500
350 x 350	500 x 500 x 25	49.1	2360	2650	2920	3180	3410
	30	58.9	2650	3150	3470	3760	4040
	40	78.5	2840	3460	4060	4640	5220
	50	98.1	2940	3610	4250	4880	5490
	600 x 600 x 30	84.8	2800	3150	3470	3760	4040
	40	113	3690	4150	4560	4940	5300
	50	141	4260	5050	5550	6010	6440
	60	170	4320	5360	6380	7170	7680
	70	198	4320	5400	6460	7490	8500
	700 x 700 x 40	154	3690	4150	4560	4940	5300
	50	192	4500	5050	5550	6010	6440
	60	231	5370	6030	6620	7170	7680
	70	269	5880	6870	7540	8160	8740
	80	308	5880	7350	8590	9290	9950

For further information on standard connection details see Table H.40
For guidance on the use of tables see Explanatory notes in Table H.39

COLUMN: S275 or S355

PLATE: S275

Table H.43 *Continued*

COLUMN BASES
SQUARE HOLLOW SECTIONS

Column Size D x B mm	Base Plate Size[1] D_p x B_p x t_p mm		Mass kg	Axial Capacity in kN [2] for cube strengths f_{cu} 20 N/mm^2	25 N/mm^2	30 N/mm^2	35 N/mm^2	40 N/mm^2
400 x 400	600 x 600 x	30	84.8	3230	3640	4010	4360	4680
		40	113	4000	4780	5260	5700	6120
		50	141	4160	5100	6000	6890	7420
		60	170	4280	5270	6220	7160	8070
		70	198	4320	5360	6360	7330	8280
	700 x 700 x	40	154	4250	4780	5260	5700	6120
		50	192	5170	5800	6380	6920	7420
		60	231	5840	6920	7600	8240	8830
		70	269	5880	7310	8650	9370	10000
		80	308	5880	7350	8790	10200	11400
	800 x 800 x	40	201	4250	4780	5260	5700	6120
		50	251	5170	5800	6380	6920	7420
		60	301	6170	6920	7600	8240	8830
		70	352	7020	7880	8650	9370	10000
		80	402	7680	8970	9850	10700	11400
		90	452	7680	9600	10800	11700	12500
450 x 450	600 x 600 x	30	84.8	3490	4140	4570	4970	5340
		40	113	3800	4590	5370	6120	6860
		50	141	4010	4870	5700	6500	7280
		60	170	4180	5100	5990	6840	7680
		70	198	4280	5250	6190	7090	7960
	700 x 700 x	40	154	4820	5420	5970	6480	6960
		50	192	5570	6570	7230	7840	8420
		60	231	5740	7050	8330	9320	10000
		70	269	5840	7200	8530	9820	11100
		80	308	5880	7310	8690	10000	11300
	800 x 800 x	40	201	4820	5420	5970	6480	6960
		50	251	5850	6570	7230	7840	8420
		60	301	6970	7820	8600	9320	10000
		70	352	7640	8900	9780	10600	11400
		80	402	7680	9560	11100	12100	12900
		90	452	7680	9600	11500	13200	14200
	900 x 900 x	70	445	7930	8900	9780	10600	11400
		80	509	9030	10100	11100	12100	12900
		90	572	9720	11100	12200	13200	14200
		100	636	9720	12200	13500	14700	15700

For further information on standard connection details see Table H.40
For guidance on the use of tables see Explanatory notes in Table H.39

COLUMN: S275 or S355
PLATE: S275

Table H.43 *Continued*

COLUMN BASES
SQUARE HOLLOW SECTIONS

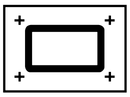
Column Size D x B mm	Base Plate Size[1] $D_p \times B_p \times t_p$ mm	Mass kg	Axial Capacity in kN [2] for cube strengths f_{cu} 20 N/mm²	25 N/mm²	30 N/mm²	35 N/mm²	40 N/mm²
500 x 500	700 x 700 x 40	154	5080	6040	6650	7220	7750
	50	192	5350	6500	7620	8700	9380
	60	231	5590	6810	7990	9130	10300
	70	269	5730	7020	8250	9450	10600
	80	308	5840	7190	8490	9750	11000
	800 x 800 x 40	201	5370	6040	6650	7220	7750
	50	251	6520	7320	8060	8740	9380
	60	301	7390	8720	9580	10400	11100
	70	352	7530	9270	10900	11800	12700
	80	402	7640	9440	11200	12900	14400
	90	452	7680	9540	11300	13100	14800
	900 x 900 x 70	445	8840	9920	10900	11800	12700
	80	509	9680	11300	12400	13400	14400
	90	572	9720	12100	13600	14700	15800
	100	636	9720	12100	14500	16300	17500
	1000 x 1000 x 50	393	6520	7320	8060	8740	9380
	60	471	7760	8720	9580	10400	11100
	70	550	8840	9920	10900	11800	12700
	80	628	10100	11300	12400	13400	14400
	90	707	11100	12400	13600	14700	15800
	100	785	12000	13700	15100	16300	17500
	110	864	12000	14800	16200	17500	18800
	120	942	12000	15000	17600	19100	20400

For further information on standard connection details see Table H.40
For guidance on the use of tables see Explanatory notes in Table H.39

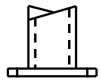
COLUMN: S275 or S355

Table H.44

PLATE: S275



COLUMN BASES
RECTANGULAR HOLLOW SECTIONS



Column Size D x B mm	Base Plate Size[1] D_p x B_p x t_p mm	Base Plate Mass kg	Axial Capacity in kN [2] for cube strengths f_{cu} 20 N/mm^2	25 N/mm^2	30 N/mm^2	35 N/mm^2	40 N/mm^2
200 x 100	400 x 300 x 20	18.8	772	878	977	1060	1140
	25	23.6	949	1070	1180	1290	1390
	30	28.3	1140	1270	1400	1510	1630
	35	33.0	1360	1500	1630	1760	1880
	40	37.7	1440	1740	1880	2020	2150
	450 x 350 x 25	30.9	949	1070	1180	1290	1390
	30	37.1	1140	1270	1400	1510	1630
	35	43.3	1360	1500	1630	1760	1880
	40	49.5	1590	1740	1880	2020	2150
	45	55.6	1790	1950	2100	2250	2390
200 x 120	400 x 300 x 20	18.8	845	949	1040	1130	1220
	25	23.6	1030	1160	1290	1400	1500
	30	28.3	1230	1380	1520	1650	1780
	35	33.0	1400	1610	1760	1900	2040
	40	37.7	1440	1770	2020	2170	2320
	450 x 350 x 25	30.9	1030	1160	1290	1400	1500
	30	37.1	1230	1380	1520	1650	1780
	35	43.3	1450	1610	1760	1900	2040
	40	49.5	1690	1860	2020	2170	2320
	45	55.6	1850	2080	2250	2410	2570
200 x 150	400 x 300 x 20	18.8	934	1050	1160	1260	1360
	25	23.6	1150	1300	1430	1550	1670
	30	28.3	1310	1530	1690	1840	1970
	35	33.0	1400	1660	1920	2120	2280
	40	37.7	1440	1770	2040	2290	2550
	450 x 350 x 35	43.3	1590	1770	1950	2120	2280
	40	49.5	1750	2030	2220	2400	2570
	45	55.6	1850	2170	2460	2650	2830
250 x 100	450 x 300 x 25	26.5	1090	1230	1370	1500	1630
	30	31.8	1300	1460	1610	1750	1880
	35	37.1	1530	1700	1860	2010	2160
	40	42.4	1620	1960	2130	2300	2450
	45	47.7	1620	2030	2370	2540	2710
	500 x 350 x 25	34.3	1090	1230	1370	1500	1630
	30	41.2	1300	1460	1610	1750	1880
	40	55.0	1780	1960	2130	2300	2450
	50	68.7	2100	2470	2670	2850	3030
	60	82.4	2100	2630	3150	3520	3720

For further information on standard connection details see Table H.40
For guidance on the use of tables see Explanatory notes in Table H.39

COLUMN: S275 or S355
PLATE: S275

Table H.44 *Continued*

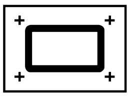
COLUMN BASES RECTANGULAR HOLLOW SECTIONS

Column Size D x B mm	Base Plate Size[1] D_p x B_p x t_p mm	Base Plate Mass kg	Axial Capacity in kN [2] for cube strengths f_{cu} 20 N/mm^2	25 N/mm^2	30 N/mm^2	35 N/mm^2	40 N/mm^2
250 x 150	450 x 350 x 25	30.9	1320	1480	1630	1760	1890
	30	37.1	1550	1750	1940	2100	2250
	35	43.3	1790	2010	2220	2430	2610
	40	49.5	1890	2290	2520	2730	2940
	45	55.6	1890	2360	2770	3000	3210
	500 x 400 x 25	39.3	1320	1480	1630	1760	1890
	30	47.1	1550	1750	1940	2100	2250
	40	62.8	2060	2290	2520	2730	2940
	50	78.5	2400	2840	3090	3330	3560
	60	94.2	2400	3000	3600	4040	4290
	600 x 500 x 30	70.7	1550	1750	1940	2100	2250
	40	94.2	2060	2290	2520	2730	2940
	50	118	2580	2840	3090	3330	3560
	60	141	3210	3500	3780	4040	4290
300 x 100	500 x 300 x 25	29.4	1230	1400	1560	1710	1860
	30	35.3	1460	1640	1810	1980	2140
	40	47.1	1800	2180	2380	2570	2760
	50	58.9	1800	2250	2700	3150	3380
	60	70.7	1800	2250	2700	3150	3600
	600 x 400 x 30	56.5	1460	1640	1810	1980	2140
	40	75.4	1970	2180	2380	2570	2760
	50	94.2	2490	2730	2950	3170	3380
	60	113	2880	3390	3640	3880	4110
	70	132	2880	3600	4290	4550	4800
300 x 200	500 x 400 x 25	39.3	1660	1870	2050	2230	2390
	30	47.1	1980	2220	2440	2650	2840
	40	62.8	2400	2920	3220	3490	3740
	50	78.5	2400	3000	3600	4200	4550
	60	94.2	2400	3000	3600	4200	4800
	600 x 500 x 30	70.7	1980	2220	2440	2650	2840
	40	94.2	2590	2920	3220	3490	3740
	50	118	3170	3540	3890	4220	4550
	60	141	3600	4270	4650	5020	5370
	70	165	3600	4500	5360	5760	6140
	700 x 600 x 40	132	2590	2920	3220	3490	3740
	50	165	3170	3540	3890	4220	4550
	60	198	3860	4270	4650	5020	5370
	70	231	4510	4950	5360	5760	6140

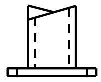
For further information on standard connection details see Table H.40
For guidance on the use of tables see Explanatory notes in Table H.39

COLUMN: S275 or S355
PLATE: S275

Table H.44 *Continued*



COLUMN BASES
RECTANGULAR HOLLOW SECTIONS



Column Size D x B mm	Base Plate Size[1] D_p x B_p x t_p mm	Base Plate Mass kg	Axial Capacity in kN [2] for cube strengths f_{cu} 20 N/mm^2	25 N/mm^2	30 N/mm^2	35 N/mm^2	40 N/mm^2
400 x 200	600 x 400 x 30	56.5	2390	2690	2960	3210	3450
	40	75.4	2880	3510	3890	4220	4530
	50	94.2	2880	3600	4320	5040	5480
	60	113	2880	3600	4320	5040	5760
	70	132	2880	3600	4320	5040	5760
	700 x 500 x 40	110	3090	3510	3890	4220	4530
	50	137	3730	4190	4640	5060	5480
	60	165	4200	4990	5480	5950	6400
	70	192	4200	5250	6260	6760	7240
	80	220	4200	5250	6300	7350	8260
	800 x 600 x 40	151	3090	3510	3890	4220	4530
	50	188	3730	4190	4640	5060	5480
	60	226	4480	4990	5480	5950	6400
	70	264	5190	5740	6260	6760	7240
	80	301	5760	6650	7210	7750	8260
450 x 250	700 x 500 x 40	110	3690	4150	4560	4940	5300
	50	137	4200	5040	5550	6010	6440
	60	165	4200	5250	6300	7140	7680
	70	192	4200	5250	6300	7350	8400
	80	220	4200	5250	6300	7350	8400
	800 x 600 x 40	151	3690	4150	4560	4940	5300
	50	188	4440	5040	5550	6010	6440
	60	226	5260	5910	6530	7140	7680
	70	264	5760	6720	7380	8020	8640
	80	301	5760	7200	8410	9090	9750
	90	339	5760	7200	8640	10000	10700
	900 x 700 x 70	346	6020	6720	7380	8020	8640
	80	396	6950	7700	8410	9090	9750
	90	445	7560	8540	9290	10000	10700
	100	495	7560	9450	10400	11200	11900

For further information on standard connection details see Table H.40
For guidance on the use of tables see Explanatory notes in Table H.39

COLUMN: S275 or S355
PLATE: S275

Table H.44 *Continued*

COLUMN BASES
RECTANGULAR HOLLOW SECTIONS

Column Size	Base Plate Size[1]	Base Plate Mass	Axial Capacity in kN [2] for cube strengths f_{cu}				
D x B	D_p x B_p x t_p		20	25	30	35	40
mm	mm	kg	N/mm^2	N/mm^2	N/mm^2	N/mm^2	N/mm^2
500 x 300	700 x 500 x 40	110	3990	4750	5230	5670	6080
	50	137	4150	5080	5980	6860	7380
	60	165	4200	5250	6210	7130	8040
	70	192	4200	5250	6300	7320	8260
	80	220	4200	5250	6300	7350	8400
	800 x 600 x 40	151	4230	4750	5230	5670	6080
	50	188	5160	5790	6360	6890	7380
	60	226	5760	6900	7590	8210	8800
	70	264	5760	7200	8590	9350	10000
	80	301	5760	7200	8640	10100	11400
	90	339	5760	7200	8640	10100	11500
	900 x 700 x 70	346	6910	7770	8590	9350	10000
	80	396	7560	8820	9700	10500	11400
	90	445	7560	9450	10600	11500	12400
	1000 x 800 x 50	314	5160	5790	6360	6890	7380
	60	377	6090	6900	7590	8210	8800
	70	440	6910	7770	8590	9350	10000
	80	502	7900	8820	9700	10500	11400
	90	565	8750	9720	10600	11500	12400

For further information on standard connection details see Table H.40
For guidance on the use of tables see Explanatory notes in Table H.39

Material Strengths

The following extracts from BS 5950-1:2000[1], BS EN 10210-1[3] and BS EN 10025[21] have been included for the convienience of the connection designer:

Table H.45 Steel Strengths

Strengths for sections, plates and hollow sections			
Steel Grade	**Thickness less than or equal to (mm)**	**Design strength (a) p_y (N/mm^2)**	**Ultimate strength (b) U_s (N/mm^2)**
S275	16 40 63 80 100 150	275 265 255 245 235 225	410
S355	16 40 63 80 100 150	355 345 335 325 315 295	490

(a) Values from BS 5950-1[1] Table 9

(b) Values from BS EN 10025[21] Table 5 for sections and plates and BS EN 10210-1[3] Table A.3 for hollow sections. In BS EN 10025[21] and BS EN 10210-1[3] p_y *and* U_s are designated R_{eH} and R_m respectively

Table H.46 Weld Strengths

Steel Grade	**Fillet weld design strength (a), p_w (N/mm^2) for Electrode Classification (b)**	
	E35	E42
S275	**220**	(220)
S355	(220)	**250**

(a) Values from BS 5950-1[1] Table 37. Values in ( ) are for under- or over-matched electrodes.

(b) Welding consumables should conform to BS EN 440[48], BS EN 499[39], BS EN 756[49], BS EN 758[50] or BS EN 1668[51] as appropriate.

Table H.47 Bolt Strengths

Bolt grade	**Bolt strength in clearance holes (N/mm^2)**		
	Shear strength p_s (a)	**Bearing strength, p_{bb} (b)**	**Tension strength p_t (c)**
4.6	160	460	240
8.8	375	1000	560
10.9	400	1300	700

(a) Values from BS 5950-1[1] Table 30

(b) Values from BS 5950-1[1] Table 31

(c) Values from BS 5950-1[1] Table 34

Table H.48 Bearing Strength of Connected Parts

Steel Grade	**Bearing strength of connected parts p_{bs} (a) (N/mm^2)**
S275	460
S355	550

(a) Values from BS 5950-1[1] Table 32.

BOLT CAPACITIES

BS 5950-1: 2000
BS 4190: 2001

Table H.49 Non-Preloaded Ordinary Bolts in S275

GRADE 4.6 BOLTS IN S275

Diameter of Bolt	Tensile Stress Area	Tension Capacity		Shear Capacity		Bearing Capacity in kN (Minimum of P_{bb} and P_{bs}) End distance equal to 2 x bolt diameter.										
		Nominal $0.8A_tp_t$	Exact A_tp_t	Single Shear	Double Shear	Thickness in mm of ply passed through.										
	A_t	P_{nom}	P_t	P_s	$2P_s$											
mm	mm^2	kN	kN	kN	kN	5	6	7	8	9	10	12	15	20	25	30
12	84.3	16.2	20.2	13.5	27.0	*27.6*	*33.1*	*38.6*	*44.2*	*49.7*	*55.2*	*66.2*	*82.8*	*110*	*138*	*166*
16	157	30.1	37.7	25.1	50.2	36.8	44.2	*51.5*	*58.9*	*66.2*	*73.6*	*88.3*	*110*	*147*	*184*	*221*
20	245	47.0	58.8	39.2	78.4	46.0	55.2	64.4	73.6	*82.8*	*92.0*	*110*	*138*	*184*	*230*	*276*
22	303	58.2	72.7	48.5	97.0	50.6	60.7	70.8	81.0	91.1	*101*	*121*	*152*	*202*	*253*	*304*
24	353	67.8	84.7	56.5	113	**55.2**	66.2	77.3	88.3	99.4	110	*132*	*166*	*221*	*276*	*331*
27	459	88.1	110	73.4	147	**62.1**	74.5	86.9	99.4	112	124	*149*	*186*	*248*	*311*	*373*
30	561	108	135	89.8	180	**69.0**	**82.8**	96.6	110	124	138	166	*207*	*276*	*345*	*414*

See notes below

GRADE 8.8 BOLTS IN S275

Diameter of Bolt	Tensile Stress Area	Tension Capacity		Shear Capacity		Bearing Capacity in kN (Minimum of P_{bb} and P_{bs}) End distance equal to 2 x bolt diameter.										
		Nominal $0.8A_tp_t$	Exact A_tp_t	Single Shear	Double Shear	Thickness in mm of ply passed through.										
	A_t	P_{nom}	P_t	P_s	$2P_s$											
mm	mm^2	kN	kN	kN	kN	5	6	7	8	9	10	12	15	20	25	30
12	84.3	37.8	47.2	31.6	63.2	**27.6**	33.1	38.6	44.2	49.7	55.2	*66.2*	*82.8*	*110*	*138*	*166*
16	157	70.3	87.9	58.9	118	**36.8**	**44.2**	**51.5**	58.9	66.2	73.6	88.3	110	*147*	*184*	*221*
20	245	110	137	91.9	184	**46.0**	**55.2**	**64.4**	**73.6**	**82.8**	92.0	110	138	*184*	*230*	*276*
22	303	136	170	114	227	**50.6**	**60.7**	**70.8**	**81.0**	**91.1**	**101**	121	152	202	*253*	*304*
24	353	158	198	132	265	**55.2**	**66.2**	**77.3**	**88.3**	**99.4**	**110**	132	166	221	*276*	*331*
27	459	206	257	172	344	**62.1**	**74.5**	**86.9**	**99.4**	**112**	**124**	**149**	186	248	311	*373*
30	561	251	314	210	421	**69.0**	**82.8**	**96.6**	**110**	**124**	**138**	**166**	**207**	276	345	414

See notes below

GRADE 10.9 BOLTS IN S275

Diameter of Bolt	Tensile Stress Area	Tension Capacity		Shear Capacity		Bearing Capacity in kN (Minimum of P_{bb} and P_{bs}) End distance equal to 2 x bolt diameter.										
		Nominal $0.8A_tp_t$	Exact A_tp_t	Single Shear	Double Shear	Thickness in mm of ply passed through.										
	A_t	P_{nom}	P_t	P_s	$2P_s$											
mm	mm^2	kN	kN	kN	kN	5	6	7	8	9	10	12	15	20	25	30
12	84.3	47.2	59.0	33.7	67.4	**27.6**	**33.1**	38.6	44.2	49.7	55.2	66.2	*82.8*	*110*	*138*	*166*
16	157	87.9	110	62.8	126	**36.8**	**44.2**	**51.5**	**58.9**	66.2	73.6	88.3	110	*147*	*184*	*221*
20	245	137	172	98.0	196	**46.0**	**55.2**	**64.4**	**73.6**	**82.8**	**92.0**	110	138	184	*230*	*276*
22	303	170	212	121	242	**50.6**	**60.7**	**70.8**	**81.0**	**91.1**	**101**	121	152	202	*253*	*304*
24	353	198	247	141	282	**55.2**	**66.2**	**77.3**	**88.3**	**99.4**	**110**	**132**	166	221	276	*331*
27	459	257	321	184	367	**62.1**	**74.5**	**86.9**	**99.4**	**112**	**124**	**149**	186	248	311	*373*
30	561	314	393	224	449	**69.0**	**82.8**	**96.6**	**110**	**124**	**138**	**166**	**207**	276	345	414

Values in **bold** are less than the single shear capacity of the bolt.
Values in *italic* are greater than the double shear capacity of the bolt.
Bearing values assume standard clearance holes.
If oversize or short slotted holes are used, bearing values should be multiplied by 0.7.
If long slotted or kidney shaped holes are used, bearing values should be multiplied by 0.5.
If appropriate, shear capacity must be reduced for large packings, large grip lengths and long joints.

BOLT CAPACITIES

BS 5950-1: 2000
BS 4395: 1969
BS 4604: 1970

Table H.50 Preloaded HSFG Bolts in S275: Non-Slip in Service

GENERAL GRADE HSFG BOLTS IN S275

Diameter of Bolt	Min. Shank Tension	Tension		Shear Capacity		Slip Resistance for μ = 0.5		Bearing Capacity, P_{bg} in kN. End distance equal to 3 x bolt diameter. Thickness in mm of ply passed through.										
	P_o	$1.1P_o$	A_tp_t	Single Shear	Double Shear	Single Shear	Double Shear											
mm	kN	kN	kN	kN	kN	kN	kN	5	6	7	8	9	10	12	15	20	25	30
12	49.4	54.3	49.7	33.7	67.4	27.2	54.3	41.4	49.7	58.0	66.2	*74.5*	*82.8*	*99.4*	*124*	*166*	*207*	*248*
16	92.1	101	92.6	62.8	126	50.7	101	**55.2**	66.2	77.3	88.3	99.4	110	*132*	*166*	*221*	*276*	*331*
20	144	158	145	98.0	196	79.2	158	**69.0**	**82.8**	**96.6**	110	124	138	166	*207*	*276*	*345*	*414*
22	177	195	179	121	242	97.4	195	**75.9**	**91.1**	**106**	121	137	152	182	228	*304*	*380*	*455*
24	207	228	208	141	282	114	228	**82.8**	**99.4**	**116**	**132**	149	166	199	248	*331*	*414*	*497*
27	234	257	236	161	321	129	257	**93.2**	**112**	**130**	**149**	168	186	224	279	*373*	*466*	*559*
30	286	315	289	196	393	157	315	**104**	**124**	**145**	**166**	**186**	207	248	311	*414*	*518*	*621*

See notes below

HIGHER GRADE HSFG BOLTS IN S275

Diameter of Bolt	Min. Shank Tension	Tension		Shear Capacity		Slip Resistance for μ = 0.5		Bearing Capacity, P_{bg} in kN. End distance equal to 3 x bolt diameter. Thickness in mm of ply passed through.										
	P_o	$1.1P_o$	A_tp_t	Single Shear	Double Shear	Single Shear	Double Shear											
mm	kN	kN	kN	kN	kN	kN	kN	5	6	7	8	9	10	12	15	20	25	30
16	104	114	110	62.8	126	57.1	114	**55.2**	66.2	77.3	88.3	99.4	110	*132*	*166*	*221*	*276*	*331*
20	162	178	172	98.0	196	89.0	178	**69.0**	**82.8**	**96.6**	110	124	138	166	*207*	*276*	*345*	*414*
22	200	220	212	121	242	110	220	**75.9**	**91.1**	**106**	121	137	152	182	228	*304*	*380*	*455*
24	233	257	247	141	282	128	257	**82.8**	**99.4**	**116**	**132**	149	166	199	248	*331*	*414*	*497*
27	303	333	321	184	367	167	333	**93.2**	**112**	**130**	**149**	**168**	186	224	279	*373*	*466*	*559*
30	370	407	393	224	449	204	407	**104**	**124**	**145**	**166**	**186**	**207**	248	311	414	*518*	*621*

Values in **bold** are less than the single shear capacity of the bolt.
Values in *italic* are greater than the double shear capacity of the bolt.
Shading indicates that the ply thickness is not suitable for an outer ply.

BOLT CAPACITIES

BS 5950-1: 2000
BS 4395: 1969
BS 4604: 1970

Table H.51 Preloaded HSFG Bolts in S275: Non-Slip under Factored Loads

GENERAL GRADE HSFG BOLTS IN S275

Diameter of Bolt	Min. Shank Tension	Bolt Tension Capacity	Slip Resistance P_{sL}							
			μ = 0.2		μ = 0.3		μ = 0.4		μ = 0.5	
	P_o	$0.9P_o$	Single Shear	Double Shear	Single Shear	Double Shear	Single Shear	Double Shear	Single Shear	Double Shear
mm	kN	kN	kN	kN	kN	kN	kN	kN	kN	kN
12	49.4	44.5	8.89	17.8	13.3	26.7	17.8	35.6	22.2	44.5
16	92.1	82.9	16.6	33.2	24.9	49.7	33.2	66.3	41.4	82.9
20	144	130	25.9	51.8	38.9	77.8	51.8	104	64.8	130
22	177	159	31.9	63.7	47.8	95.6	63.7	127	79.7	159
24	207	186	37.3	74.5	55.9	112	74.5	149	93.2	186
27	234	211	42.1	84.2	63.2	126	84.2	168	105	211
30	286	257	51.5	103	77.2	154	103	206	129	257

HIGHER GRADE HSFG BOLTS IN S275

Diameter of Bolt	Min. Shank Tension	Bolt Tension Capacity	Slip Resistance P_{sL}							
			μ = 0.2		μ = 0.3		μ = 0.4		μ = 0.5	
	P_o	$0.9P_o$	Single Shear	Double Shear	Single Shear	Double Shear	Single Shear	Double Shear	Single Shear	Double Shear
mm	kN	kN	kN	kN	kN	kN	kN	kN	kN	kN
16	104	93.5	18.7	37.4	28.1	56.1	37.4	74.8	46.8	93.5
20	162	146	29.1	58.2	43.7	87.4	58.2	116	72.8	146
22	200	180	36.0	72.1	54.1	108	72.1	144	90.1	180
24	233	210	42.0	84.0	63.0	126	84.0	168	105	210
27	303	273	54.5	109	81.8	164	109	218	136	273
30	370	333	66.6	133	99.9	200	133	266	167	333

BOLT CAPACITIES

BS 5950-1: 2000
BS 4190: 2001

Table H.52 Non-Preloaded Ordinary Bolts in S355

GRADE 4.6 BOLTS IN S355

Diameter of Bolt	Tensile Stress Area	Tension Capacity		Shear Capacity		Bearing Capacity in kN (Minimum of P_{bb} and P_{bs}) End distance equal to 2 x bolt diameter. Thickness in mm of ply passed through.										
		Nominal $0.8A_tp_t$	Exact A_tp_t	Single Shear	Double Shear											
	A_t	P_{nom}	P_t	P_s	$2P_s$											
mm	mm^2	kN	kN	kN	kN	5	6	7	8	9	10	12	15	20	25	30
12	84.3	16.2	20.2	13.5	27.0	*27.6*	*33.1*	*38.6*	*44.2*	*49.7*	*55.2*	*66.2*	*82.8*	*110*	*138*	*166*
16	157	30.1	37.7	25.1	50.2	36.8	44.2	*51.5*	*58.9*	*66.2*	*73.6*	*88.3*	*110*	*147*	*184*	*221*
20	245	47.0	58.8	39.2	78.4	46.0	55.2	64.4	73.6	*82.8*	*92.0*	*110*	*138*	*184*	*230*	*276*
22	303	58.2	72.7	48.5	97.0	50.6	60.7	70.8	81.0	91.1	*101*	*121*	*152*	*202*	*253*	*304*
24	353	67.8	84.7	56.5	113	**55.2**	66.2	77.3	88.3	99.4	110	*132*	*166*	*221*	*276*	*331*
27	459	88.1	110	73.4	147	**62.1**	74.5	86.9	99.4	112	124	*149*	*186*	*248*	*311*	*373*
30	561	108	135	89.8	180	**69.0**	**82.8**	96.6	110	124	138	166	*207*	*276*	*345*	*414*

See notes below

GRADE 8.8 BOLTS IN S355

Diameter of Bolt	Tensile Stress Area	Tension Capacity		Shear Capacity		Bearing Capacity in kN (Minimum of P_{bb} and P_{bs}) End distance equal to 2 x bolt diameter. Thickness in mm of ply passed through.										
		Nominal $0.8A_tp_t$	Exact A_tp_t	Single Shear	Double Shear											
	A_t	P_{nom}	P_t	P_s	$2P_s$											
mm	mm^2	kN	kN	kN	kN	5	6	7	8	9	10	12	15	20	25	30
12	84.3	37.8	47.2	31.6	63.2	33.0	39.6	46.2	52.8	59.4	*66.0*	*79.2*	*99.0*	*132*	*165*	*198*
16	157	70.3	87.9	58.9	118	**44.0**	**52.8**	61.6	70.4	79.2	88.0	106	*132*	*176*	*220*	*264*
20	245	110	137	91.9	184	**55.0**	**66.0**	**77.0**	**88.0**	99.0	110	132	165	*220*	*275*	*330*
22	303	136	170	114	227	**60.5**	**72.6**	**84.7**	**96.8**	**109**	121	145	182	*242*	*303*	*363*
24	353	158	198	132	265	**66.0**	**79.2**	**92.4**	**106**	**119**	**132**	158	198	264	*330*	*396*
27	459	206	257	172	344	**74.3**	**89.1**	**104**	**119**	**134**	**149**	178	223	297	*371*	*446*
30	561	251	314	210	421	**82.5**	**99.0**	**116**	**132**	**149**	**165**	**198**	248	330	413	*495*

See notes below

GRADE 10.9 BOLTS IN S355

Diameter of Bolt	Tensile Stress Area	Tension Capacity		Shear Capacity		Bearing Capacity in kN (Minimum of P_{bb} and P_{bs}) End distance equal to 2 x bolt diameter. Thickness in mm of ply passed through.										
		Nominal $0.8A_tp_t$	Exact A_tp_t	Single Shear	Double Shear											
	A_t	P_{nom}	P_t	P_s	$2P_s$											
mm	mm^2	kN	kN	kN	kN	5	6	7	8	9	10	12	15	20	25	30
12	84.3	47.2	59.0	33.7	67.4	**33.0**	39.6	46.2	52.8	59.4	66.0	*79.2*	*99.0*	*132*	*165*	*198*
16	157	87.9	110	62.8	126	**44.0**	**52.8**	**61.6**	70.4	79.2	88.0	106	*132*	*176*	*220*	*264*
20	245	137	172	98.0	196	**55.0**	**66.0**	**77.0**	**88.0**	99.0	110	132	165	*220*	*275*	*330*
22	303	170	212	121	242	**60.5**	**72.6**	**84.7**	**96.8**	**109**	**121**	145	182	242	*303*	*363*
24	353	198	247	141	282	**66.0**	**79.2**	**92.4**	**106**	**119**	**132**	158	198	264	*330*	*396*
27	459	257	321	184	367	**74.3**	**89.1**	**104**	**119**	**134**	**149**	**178**	223	297	*371*	*446*
30	561	314	393	224	449	**82.5**	**99.0**	**116**	**132**	**149**	**165**	**198**	248	330	413	*495*

Values in **bold** are less than the single shear capacity of the bolt.
Values in *italic* are greater than the double shear capacity of the bolt.
Bearing values assume standard clearance holes.
If oversize or short slotted holes are used, bearing values should be multiplied by 0.7.
If long slotted or kidney shaped holes are used, bearing values should be multiplied by 0.5.
If appropriate, shear capacity must be reduced for large packings, large grip lengths and long joints.

BOLT CAPACITIES

BS 5950-1: 2000
BS 4395: 1969
BS 4604: 1970

Table H.53 Preloaded HSFG Bolts in S355: Non-Slip in Service

GENERAL GRADE HSFG BOLTS IN S355

Diameter of Bolt	Min. Shank Tension	Tension		Shear Capacity		Slip Resistance for μ = 0.5		Bearing Capacity, P_{bg} in kN End distance equal to 3 x bolt diameter. Thickness in mm of ply passed through.										
	P_o	$1.1P_o$	$A_t p_t$	Single Shear	Double Shear	Single Shear	Double Shear											
mm	kN	kN	kN	kN	kN	kN	kN	5	6	7	8	9	10	12	15	20	25	30
12	49.4	54.3	49.7	33.7	67.4	27.2	54.3	49.5	59.4	*69.3*	*79.2*	*89.1*	*99.0*	*119*	*149*	*198*	*248*	*297*
16	92.1	101	92.6	62.8	126	50.7	101	66.0	79.2	92.4	106	119	*132*	*158*	*198*	*264*	*330*	*396*
20	144	158	145	98.0	196	79.2	158	**82.5**	99.0	116	132	149	165	*198*	*248*	*330*	*413*	*495*
22	177	195	179	121	242	97.4	195	**90.8**	**109**	127	145	163	182	218	*272*	*363*	*454*	*545*
24	207	228	208	141	282	114	228	**99.0**	**119**	**139**	158	178	198	238	*297*	*396*	*495*	*594*
27	234	257	236	161	321	129	257	**111**	**134**	**156**	178	200	223	267	*334*	*446*	*557*	*668*
30	286	315	289	196	393	157	315	**124**	**149**	**173**	198	223	248	297	371	*495*	*619*	*743*

See notes below

HIGHER GRADE HSFG BOLTS IN S355

Diameter of Bolt	Min. Shank Tension	Tension		Shear Capacity		Slip Resistance for μ = 0.5		Bearing Capacity, P_{bg} in kN End distance equal to 3 x bolt diameter. Thickness in mm of ply passed through.										
	P_o	$1.1P_o$	$A_t p_t$	Single Shear	Double Shear	Single Shear	Double Shear											
mm	kN	kN	kN	kN	kN	kN	kN	5	6	7	8	9	10	12	15	20	25	30
16	104	114	110	62.8	126	57.1	114	66.0	79.2	92.4	106	119	*132*	*158*	*198*	*264*	*330*	*396*
20	162	178	172	98.0	196	89.0	178	**82.5**	99.0	116	132	149	165	*198*	*248*	*330*	*413*	*495*
22	200	220	212	121	242	110	220	**90.8**	**109**	127	145	163	182	218	*272*	*363*	*454*	*545*
24	233	257	247	141	282	128	257	**99.0**	**119**	**139**	158	178	198	238	*297*	*396*	*495*	*594*
27	303	333	321	184	367	167	333	**111**	**134**	**156**	**178**	200	223	267	334	*446*	*557*	*668*
30	370	407	393	224	449	204	407	**124**	**149**	**173**	**198**	**223**	248	297	371	*495*	*619*	*743*

Values in **bold** are less than the single shear capacity of the bolt.

Values in *italic* are greater than the double shear capacity of the bolt.

Shading indicates that the ply thickness is not suitable for an outer ply.

BOLT CAPACITIES

BS 5950-1: 2000
BS 4395: 1969
BS 4604: 1970

Table H.54 Preloaded HSFG Bolts in S355: Non-Slip under Factored Loads

GENERAL GRADE HSFG BOLTS IN S355

Diameter of Bolt	Min. Shank Tension	Bolt Tension Capacity	Slip Resistance P_{sL}							
			μ = 0.2		μ = 0.3		μ = 0.4		μ = 0.5	
			Single Shear	Double Shear	Single Shear	Double Shear	Single Shear	Double Shear	Single Shear	Double Shear
	P_o	$0.9P_o$								
mm	kN	kN	kN	kN	kN	kN	kN	kN	kN	kN
12	49.4	44.5	8.89	17.8	13.3	26.7	17.8	35.6	22.2	44.5
16	92.1	82.9	16.6	33.2	24.9	49.7	33.2	66.3	41.4	82.9
20	144	130	25.9	51.8	38.9	77.8	51.8	104	64.8	130
22	177	159	31.9	63.7	47.8	95.6	63.7	127	79.7	159
24	207	186	37.3	74.5	55.9	112	74.5	149	93.2	186
27	234	211	42.1	84.2	63.2	126	84.2	168	105	211
30	286	257	51.5	103	77.2	154	103	206	129	257

HIGHER GRADE HSFG BOLTS IN S355

Diameter of Bolt	Min. Shank Tension	Bolt Tension Capacity	Slip Resistance P_{sL}							
			μ = 0.2		μ = 0.3		μ = 0.4		μ = 0.5	
			Single Shear	Double Shear	Single Shear	Double Shear	Single Shear	Double Shear	Single Shear	Double Shear
	P_o	$0.9P_o$								
mm	kN	kN	kN	kN	kN	kN	kN	kN	kN	kN
16	104	93.5	18.7	37.4	28.1	56.1	37.4	74.8	46.8	93.5
20	162	146	29.1	58.2	43.7	87.4	58.2	116	72.8	146
22	200	180	36.0	72.1	54.1	108	72.1	144	90.1	180
24	233	210	42.0	84.0	63.0	126	84.0	168	105	210
27	303	273	54.5	109	81.8	164	109	218	136	273
30	370	333	66.6	133	99.9	200	133	266	167	333

FLOWDRILL

Tension capacity - normal design

The tension capacity of Grade 8.8 bolts for normal design are shown in Table H.55a and take account of the RHS wall thickness as well as the bolt strength.

Table H.55a

<table>
<tr><th colspan="9">FLOWDRILL CONNECTIONS
Normal tension capacity (P_{nom})kN</th></tr>
<tr><th rowspan="3">Bolt diameter mm</th><th colspan="8">RHS column wall thickness mm</th></tr>
<tr><th colspan="5">S275</th><th colspan="3">S355</th></tr>
<tr><th>5</th><th>6.3</th><th>8</th><th>10</th><th>12.5</th><th>5</th><th>6.3</th><th>8 to 12.5</th></tr>
<tr><td>M16</td><td>46</td><td>60</td><td colspan="3">70.3</td><td>59</td><td colspan="2">70.3</td></tr>
<tr><td>M20</td><td>70</td><td>85</td><td>95</td><td>97</td><td>110</td><td>102</td><td colspan="2">110</td></tr>
<tr><td>M24</td><td>80</td><td>101</td><td>122</td><td>134</td><td>158</td><td>103</td><td>130</td><td>158</td></tr>
</table>

Tension capacity - structural integrity

The pull-out resistances for structural integrity are less than those for normal design because the design method used for structural integrity leads to thinner plates/cleats than for normal design methods, based on BS 5950-1:2000[1], and, as a result, will lead to higher prying forces.

The values in Table H.55b are based on limiting value of p_{tr}= 300N/mm^2 for extreme prying (See CHECK 13).

Table H.55b

<table>
<tr><th colspan="9">FLOWDRILL CONNECTIONS
Structural integrity tension capacity (P_{si})kN</th></tr>
<tr><th rowspan="3">Bolt diameter mm</th><th colspan="8">RHS column wall thickness mm</th></tr>
<tr><th colspan="5">S275</th><th colspan="3">S355</th></tr>
<tr><th>5</th><th>6.3</th><th>8</th><th>10</th><th>12.5</th><th>5</th><th>6.3</th><th>8 to 12.5</th></tr>
<tr><td>M16</td><td>30</td><td>40</td><td colspan="3">46</td><td>39</td><td colspan="2">46</td></tr>
<tr><td>M20</td><td>46</td><td>56</td><td>63</td><td>65</td><td>73</td><td>68</td><td colspan="2">73</td></tr>
<tr><td>M24</td><td>53</td><td>67</td><td>81</td><td>89</td><td>106</td><td>68</td><td>86</td><td>106</td></tr>
</table>

Shear and bearing capacity

For shear and bearing capacities, refer to capacity tables for ordinary bolts (Tables H.49 and H.52)

Note: Additional information on Flowdrill is given in Appendix F.

HOLLO-BOLT CAPACITIES

Shear capacity

Hollo-Bolts have a shear capacity slightly higher than that for ordinary bolting, since the body of the fastener provides resistance as well as the bolt. Values are given in Table H.56.

Tension capacity - normal design

The tension capacity of Grade 8.8 Hollo-Bolts for normal design are shown in Table H.56.

Tension capacity - structural integrity

The pull-out resistances for structural integrity are less than those for normal design because the design method used for structural integrity leads to thinner plates/cleats than for normal design methods, based on BS 5950-1: 2000[1], and, as a result, will lead to higher prying forces. This has been taken into account in the resistances shown in Table H.56.

Table H.56

HOLLO-BOLT DESIGN CAPACITIES			
Bolt diameter mm	**Shear Capacity kN**	**Normal Tension Capacity (P_{nom}) kN**	**Structural integrity Tension Capacity (P_{si}) kN**
M8	12	16	10
M10	25	26	17
M12	38	38	25
M16	75	70	46
M20	100	110	73

Bearing capacity

For bearing capacities refer to capacity tables for ordinary bolts (Tables H.49 and H.52)

Note: Additional information on Hollo-Bolts is given in Appendix G.

BS 5950-1 :2000
BS EN 440
BS EN 499
BS EN 756
BS EN 758
BS EN 1668

Table H.57 Weld Capacities

FILLET WELDS

Leg Length	Throat [(a)] Thickness	WELD CAPACITIES			
		E35 ELECTRODE WITH S275 [(d)]		E42 ELECTRODE WITH S355 [(e)]	
		Longitudinal Capacity	Transverse Capacity	Longitudinal Capacity	Transverse Capacity
s	a	P_L [(b)]	P_T [(c)]	P_L [(b)]	P_T [(c)]
mm	mm	kN/mm	kN/mm	kN/mm	kN/mm
4.0	2.80	0.616	0.770	0.700	0.875
6.0	4.20	0.924	1.16	1.05	1.31
8.0	5.60	1.23	1.54	1.40	1.75
10.0	7.00	1.54	1.93	1.75	2.19
12.0	8.40	1.85	2.31	2.10	2.63
15.0	10.5	2.31	2.89	2.63	3.28
18.0	12.6	2.77	3.47	3.15	3.94
20.0	14.0	3.08	3.85	3.50	4.38
22.0	15.4	3.39	4.24	3.85	4.81
25.0	17.5	3.85	4.81	4.38	5.47

(a) $a = 0.7s$

(b) $P_L = p_w\,a$

(c) $P_T = K\,p_w\,a$. Welds are between two elements at 90^o to each other, therefore K = 1.25.

(d) p_w = 220 N/mm^2 for E35 electrode with S275 steel

(e) p_w = 250 N/mm^2 for E42 electrode with S355 steel

FULL PENETRATION BUTT WELDS

Thickness	WELD CAPACITIES			
	E35 ELECTRODE WITH S275		E42 ELECTRODE WITH S355	
	Shear Capacity [(a)]	Tension or Compression Capacity [(b)]	Shear Capacity [(a)]	Tension or Compression Capacity [(b)]
mm	kN/mm	kN/mm	kN/mm	kN/mm
4.0	0.660	1.10	0.852	1.42
6.0	0.990	1.65	1.28	2.13
8.0	1.32	2.20	1.70	2.84
10.0	1.65	2.75	2.13	3.55
12.0	1.98	3.30	2.56	4.26
15.0	2.48	4.13	3.20	5.33
18.0	2.86	4.77	3.73	6.21
20.0	3.18	5.30	4.14	6.90
22.0	3.50	5.83	4.55	7.59
25.0	3.98	6.63	5.18	8.63

(a) The strength in shear is taken as $0.6p_y$.

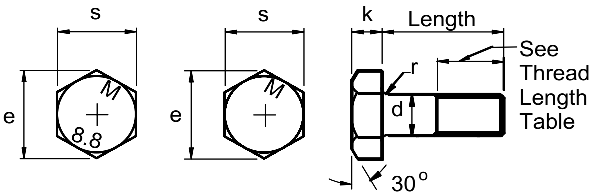
(b) The weld strength in tension or compression is taken as p_y.

Where: p_y is the design strength of the parent material.

Table H.58

DIMENSIONS OF ORDINARY BOLT ASSEMBLIES
(All dimensions in millimetres)

ISO Black Hexagonal Bolts
(BS 4190:2001)[10]

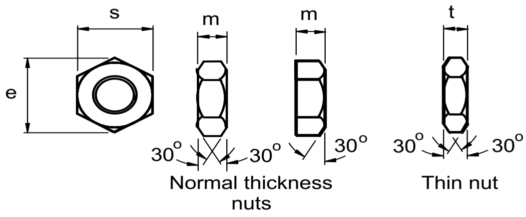


Grade 8.8 Head

Grade 4.6 Head

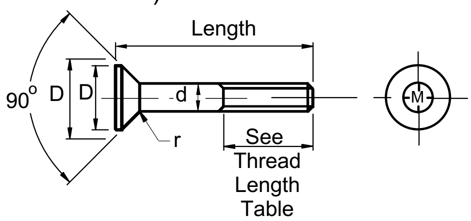
	M12	M16	**M20**	M24
s	19.00	24.00	**30.00**	36.00
d	12.70	16.70	**20.84**	24.84
e	21.90	27.70	**34.60**	41.60
k	8.45	10.45	**13.90**	15.90
r	1.25	1.25	**1.78**	1.78

ISO Metric Black Hexagonal Nuts
(BS 4190:2001)[10]



	M12	M16	**M20**	M24
s	19.00	24.00	**30.00**	36.00
e	21.90	27.70	**34.60**	41.60
m	10.00	13.00	**16.00**	19.00
t	7.00	9.00	**9.00**	10.00

90° Countersunk Round Bolts
(BS 4933:1973)[52]



	M12	M16	**M20**	M24
d	12.70	16.70	**20.84**	24.84
D (sharp)	24.00	32.00	**40.00**	48.00
D (min)	20.40	27.20	**34.00**	40.80
r	1.00	1.00	**1.00**	1.50

Thread Lengths
(BS 4190:2001[10], BS 4933:1973[52])

Nominal Bolt Length	Thread Length
Up to/including 125mm	2*d* + 6mm
Over 125mm up to/including 200	2*d* + 12mm
Over 200mm	2*d* + 25mm
Short Thread Lengths	
Up to/including 125mm	1.5*d*

Black Washers
(BS 4320:1968)[40]

	Bolt size	M12	M16	**M20**	M24
Normal (Form E)	Outside dia	24	30	**37**	44
	Thickness	2.5	3	**3**	4
	Mass of 1000 washers (kg)	6	11	**17**	31
Large (Form F)	Outside dia	28	34	**39**	50
	Thickness	2.5	3	**3**	4
	Mass of 1000 washers (kg)	9	16	**19**	45
Extra Large (Form G)	Outside dia	36	48	**60**	72
	Thickness	3	4	**5**	6
	Mass of 1000 washers (kg)	21	49	**97**	170

NOTE: *Tolerance on nominal thickness (and therefore on mass) may be as much as 30%.*

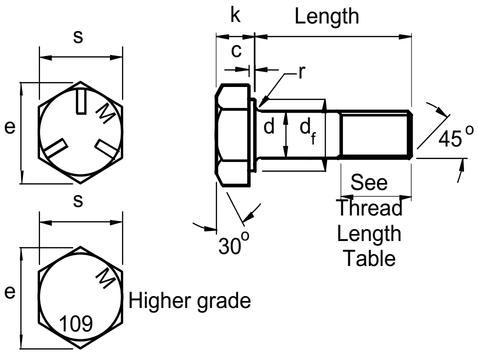
Fully threaded bolts are recommended as the industry standard, see Section 2.2.

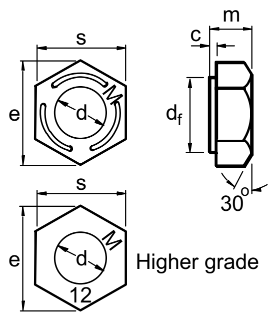
Table H.59

DIMENSIONS OF HIGH STRENGTH FRICTION GRIP BOLT ASSEMBLIES
(All dimensions in millimetres)

High Strength Friction Grip Bolts and Nuts

(BS 4395-1: 1969[11] - General Grade) (BS 4395-2: 1969[11] - Higher Grade)





	M16	**M20**	M24	M30
s	27.00	**32.00**	41.00	50.00
d	16.70	**20.84**	24.84	30.84
e	31.20	**36.90**	47.30	57.70
d_f	27.00	**32.00**	41.00	50.00
k	10.45	**13.90**	15.90	20.05
c	0.40	**0.40**	0.50	0.50
r	1.00	**1.20**	1.20	1.50

	M16	**M20**	M24	M30
s	27.00	**32.00**	41.00	50.00
e	31.20	**36.90**	47.30	57.70
d_f	27.00	**32.00**	41.00	50.00
m	15.55	**18.55**	22.65	26.65
c	0.40	**0.40**	0.50	0.50

Thread Lengths

(a) General Grade (BS 4395-1: 1969[11])

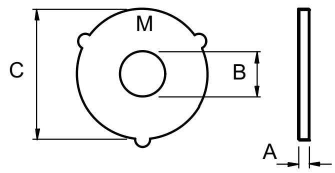
Nominal Bolt Length	**Thread Length**
Up to/including 125mm	2*d* + 6mm
Over 125mm up to/including 200	2*d* + 12mm
Over 200mm	2*d* + 25mm

(b) Higher Grade (BS 4395-2: 1969[11])

Nominal Bolt Length	**Thread Length**
Up to/including 125mm	2*d* + 12mm
Over 125mm up to/including 200	2*d* + 18mm
Over 200mm	2*d* + 30mm

Flat Round Washers

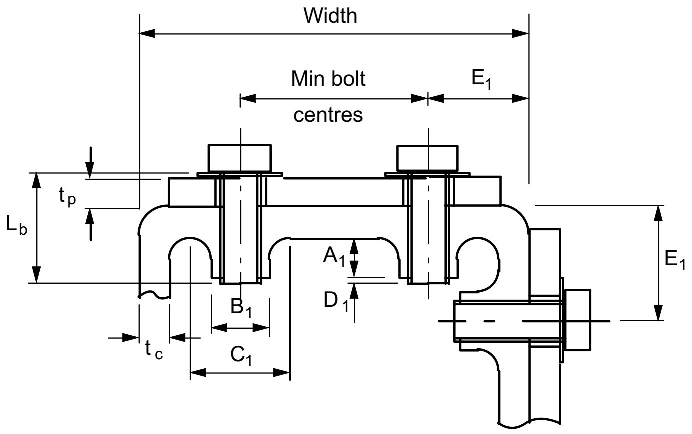
General Grade (BS 4395-1: 1969[11]) and
Higher Grade (BS 4395-2: 1969[11])



	M16	**M20**	M24	M30
A	3.40	**3.70**	4.20	4.20
B	17.80	**21.50**	26.40	32.80
C	37.00	**44.00**	56.00	66.00
Mass of 1000 washers (kg)	22	**34**	64	86

Table H.60

DETAILING OF THERMAL DRILLING BOLT ASSEMBLIES



	Dimensions (mm) for bolt size			
	M12	M16	M20	M24
A_1	7	10	12	15
B_1	13	17	22	25
C_1	18	20	26	29
D_1	Varies with overall bolt length (L_b) specified			
E_1 Minimum	$C_1/2 + t_c$ (for connections made to a single face or opposite faces)			
	$B_1/2 + A_1 + D_1 + t_c$ (for connections made to adjacent faces)			
Min Bolt Centres	30	40	50	60
Hole dia. RHS Hole dia. fitting	12 14	16 18	20 22	24 26

Notes:

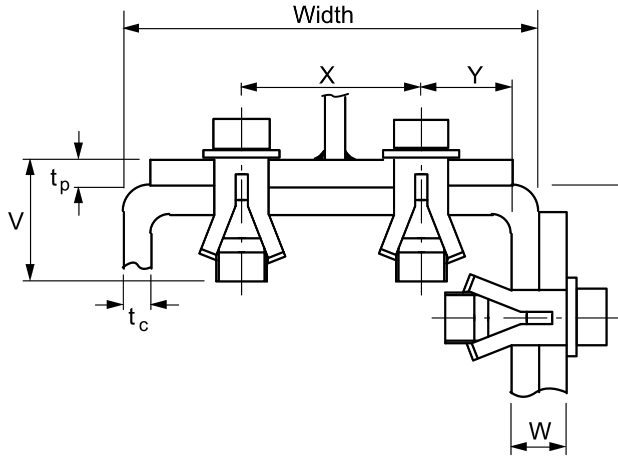
(1) The thermal drilling process is limited to RHS thicknesses up to and including 12.5mm. For thicknesses of 16mm and over, conventional drill and tap methods are recommended, although, due to the RHS material strength being lower than that of the grade 8.8 bolts, pull out strengths may be below the bolt tension capacity.

(2) Detailing must also comply with the requirements given in CHECK 1 of the appropriate design procedures.

(3) Additional information on thermal drilling is given in Appendix F.

Table H.61

DETAILING OF HOLLO-BOLT ASSEMBLIES



Bolt size	Bolt length V mm	Fixing thickness[(1)] W min mm	Fixing thickness[(1)] W max mm	Min. bolt centres X mm	Min. internal edge Y mm	Min. edge distance E_1 mm	Hole dia. D_h mm	Dimension across flats of collar mm	Nominal bolt dia. mm	Tightening torque Nm
M8 (Size 1)	50	3	22	35	13	$50 - t_p$	14	19	8	21
M8 (Size 2)	70	22	41							
M8 (Size 3)	68	41	60							
M10 (Size 1)	55	3	22	40	15	$55 - t_p$	18	24	10	40
M10 (Size 2)	75	22	41							
M10 (Size 3)	90	41	60							
M12 (Size 1)	60	3	25	50	18	$60 - t_p$	20	30	12	78
M12 (Size 2)	90	25	47							
M12 (Size 3)	110	47	69							
M16 (Size 1)	75	8	29	55	20	$65 - t_p$	28	36	16	190
M16 (Size 2)	100	29	50							
M16 (Size 3)	120	50	71							
M20 (Size 1)	90	8	34	70	25	$90 - t_p$	35	46	20	300
M20 (Size 2)	120	34	60							
M20 (Size 3)	150	60	86							

Notes:

(1) To maximise shear capacity, the outer ply thickness must be at least 8mm when M16 or M20 Hollo-Bolts are used. Where the outer ply is less than 8mm, spacer washers should be used to make the thickness up to 8mm.

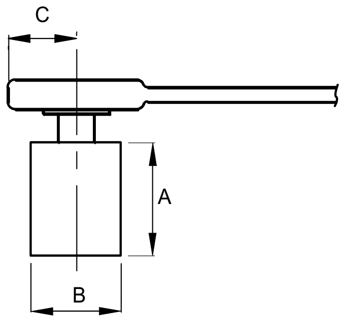
(2) Detailing must also comply with requirements given in CHECK 1 of the appropriate design procedures.

(3) Additional information on Hollo-Bolt is given in Appendix G.

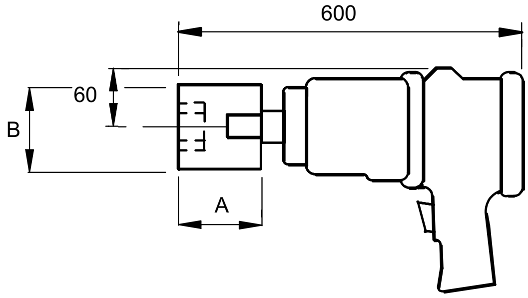
Table H.62

ENTERING AND TIGHTENING DIMENSIONS
(Approximate dimensions in millimetres)

Torque Wrench



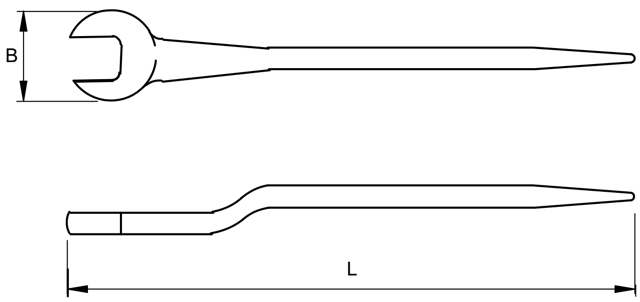
Impact Wrench



Bolt Size	A	B	C	General grade Approximate Torque (Nm)	Higher grade Approximate Torque (Nm)
M16	48	35	32	300 *	400 *
M20	**60**	**44**	**40**	**600 ***	**800 ***
M24	69	55	51	1000 *	1300 *
M30	85	70	60	1800 *	2700 *

* Values are indicative of the torque required to achieve a shank tension equal to the proof load. Site conditions and equipment determine the actual torque required. ($^{+}$/- 20%)
(Refer to BS 4604: 1970)[31]

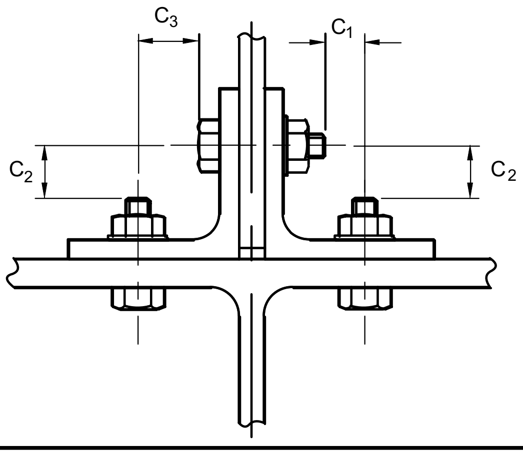
Podger Spanner



Bolt Size	B	L	Approximate Torque (Nm)
M16	60	460	90 *
M20	**70**	**550**	**110 ***
M24	85	640	130 *
M30	110	730	160 *

* Values are indicative of torque achieved when hand tigtened using a force of 250 N.

Tightening Access



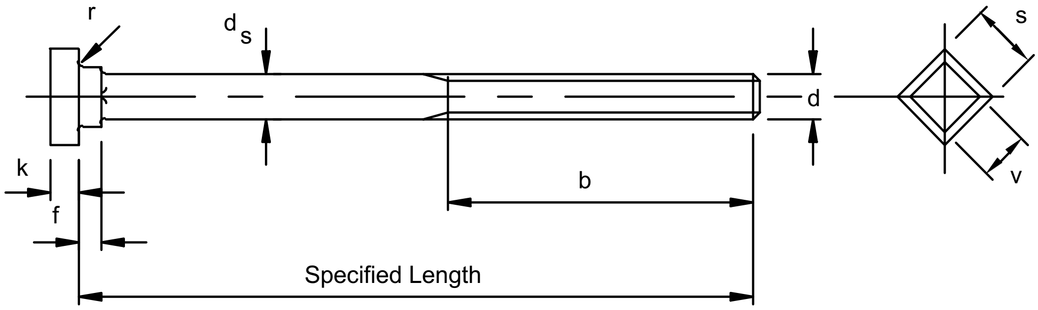
Bolt Size	C_1	C_2	C_3
M16	25	18	15
M20	**32**	**22**	**19**
M24	35	25	22

C_1 = Clearance for tightening
C_2 = Clearance for entering (holding bolt head)
C_3 = Based on standard hardened washer

Table H.63

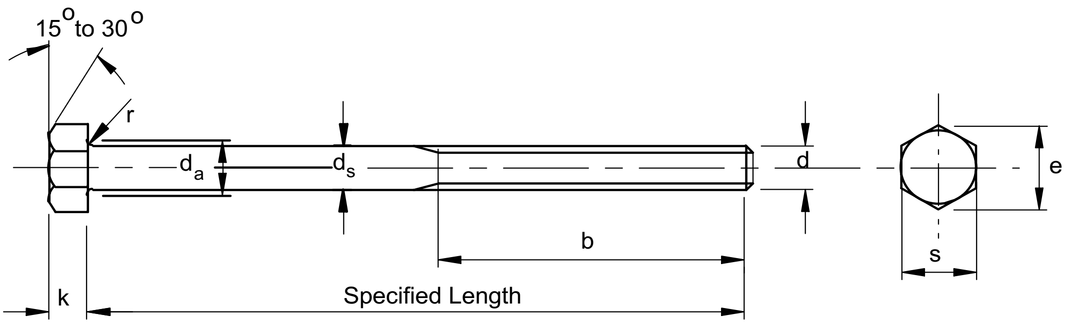
DIMENSIONS FOR HOLDING DOWN BOLTS
(All dimensions in millimetres)

Square Head Bolts
(BS 7419: 1991)[35]



	M20	M24	M30	M36	M42
b	127.50	133.00	140.50	148.00	155.50
d_s	20.84	24.84	30.84	37.00	43.00
k	12.50	15.00	18.70	22.50	26.00
r	0.80	0.80	1.00	1.00	1.20
f	10.00	12.00	15.00	18.00	21.00
s	30.00	36.00	46.00	55.00	65.00
v	20.84	24.84	30.84	37.00	43.00

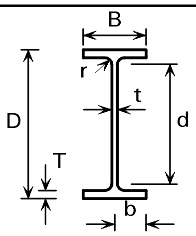
Hexagon Head Bolts
(BS 7419: 1991)[35]



	M20	M24	M30	M36	M42
b	127.50	133.00	140.50	148.00	155.50
d_a	24.40	28.40	35.40	42.40	48.60
d_s	20.84	24.84	30.84	37.00	43.00
k	12.50	15.00	18.70	22.50	26.00
r	0.80	0.80	1.00	1.00	1.20
e	32.95	39.55	50.85	60.79	71.30
s	30.00	36.00	46.00	55.00	65.00

BS 5950-1: 2000
BS 4-1: 1993

Table H.64



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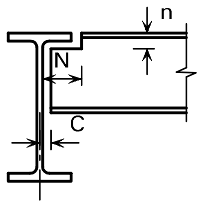
Dimensions and properties

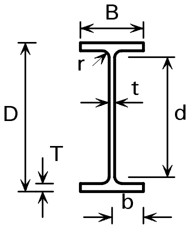


Section Designation	Mass per Metre	Depth of Section	Width of Section	Thickness		Root Radius	Depth between Fillets	Dimensions for detailing			Surface Area		Area of Section
				Web	Flange			End Clearance	Notch		Per Metre	Per Tonne	
		D	B	t	T	r	d	C	N	n			A
	kg/m	mm	mm	mm	mm	mm	mm	mm	mm	mm	m^2	m^2	cm^2
1016x305x487 # +	486.6	1036.1	308.5	30.0	54.1	30.0	867.9	17	150	86	3.19	6.57	620
1016x305x437 # +	436.9	1025.9	305.4	26.9	49.0	30.0	867.9	16	150	80	3.17	7.25	557
1016x305x393 # +	392.7	1016.0	303.0	24.4	43.9	30.0	868.2	14	150	74	3.14	8.01	500
1016x305x349 # +	349.4	1008.1	302.0	21.1	40.0	30.0	868.1	13	150	70	3.13	8.96	445
1016x305x314 # +	314.3	1000.0	300.0	19.1	35.9	30.0	868.2	12	150	66	3.11	9.90	400
1016x305x272 # +	272.3	990.1	300.0	16.5	31.0	30.0	868.1	10	152	63	3.10	11.4	347
1016x305x249 # +	248.7	980.2	300.0	16.5	26.0	30.0	868.2	10	152	56	3.08	12.4	317
1016x305x222 # +	222.0	970.3	300.0	16.0	21.1	30.0	868.1	10	152	52	3.06	13.8	283
914x419x388 #	388.0	921.0	420.5	21.4	36.6	24.1	799.6	13	210	62	3.44	8.87	494
914x419x343 #	343.3	911.8	418.5	19.4	32.0	24.1	799.6	12	210	58	3.42	9.95	437
914x305x289 #	289.1	926.6	307.7	19.5	32.0	19.1	824.4	12	156	52	3.01	10.4	368
914x305x253 #	253.4	918.4	305.5	17.3	27.9	19.1	824.4	11	156	48	2.99	11.8	323
914x305x224 #	224.2	910.4	304.1	15.9	23.9	19.1	824.4	10	156	44	2.97	13.3	286
914x305x201 #	200.9	903.0	303.3	15.1	20.2	19.1	824.4	10	156	40	2.96	14.7	256
838x292x226 #	226.5	850.9	293.8	16.1	26.8	17.8	761.7	10	150	46	2.81	12.4	289
838x292x194 #	193.8	840.7	292.4	14.7	21.7	17.8	761.7	9	150	40	2.79	14.4	247
838x292x176 #	175.9	834.9	291.7	14.0	18.8	17.8	761.7	9	150	38	2.78	15.8	224
762x267x197	196.8	769.8	268.0	15.6	25.4	16.5	686.0	10	138	42	2.55	13.0	251
762x267x173	173.0	762.2	266.7	14.3	21.6	16.5	686.0	9	138	40	2.53	14.6	220
762x267x147	146.9	754.0	265.2	12.8	17.5	16.5	686.0	8	138	34	2.51	17.1	187
762x267x134	133.9	750.0	264.4	12.0	15.5	16.5	686.0	8	138	32	2.51	18.7	171
686x254x170	170.2	692.9	255.8	14.5	23.7	15.2	615.1	9	132	40	2.35	13.8	217
686x254x152	152.4	687.5	254.5	13.2	21.0	15.2	615.1	9	132	38	2.34	15.4	194
686x254x140	140.1	683.5	253.7	12.4	19.0	15.2	615.1	8	132	36	2.33	16.6	178
686x254x125	125.2	677.9	253.0	11.7	16.2	15.2	615.1	8	132	32	2.32	18.5	159
610x305x238	238.1	635.8	311.4	18.4	31.4	16.5	540.0	11	158	48	2.45	10.3	303
610x305x179	179.0	620.2	307.1	14.1	23.6	16.5	540.0	9	158	42	2.41	13.5	228
610x305x149	149.2	612.4	304.8	11.8	19.7	16.5	540.0	8	158	38	2.39	16.0	190
610x229x140	139.9	617.2	230.2	13.1	22.1	12.7	547.6	9	120	36	2.11	15.1	178
610x229x125	125.1	612.2	229.0	11.9	19.6	12.7	547.6	8	120	34	2.09	16.7	159
610x229x113	113.0	607.6	228.2	11.1	17.3	12.7	547.6	8	120	30	2.08	18.4	144
610x229x101	101.2	602.6	227.6	10.5	14.8	12.7	547.6	7	120	28	2.07	20.5	129
533x210x122	122.0	544.5	211.9	12.7	21.3	12.7	476.5	8	110	34	1.89	15.5	155
533x210x109	109.0	539.5	210.8	11.6	18.8	12.7	476.5	8	110	32	1.88	17.2	139
533x210x101	101.0	536.7	210.0	10.8	17.4	12.7	476.5	7	110	32	1.87	18.5	129
533x210x92	92.1	533.1	209.3	10.1	15.6	12.7	476.5	7	110	30	1.86	20.2	117
533x210x82	82.2	528.3	208.8	9.6	13.2	12.7	476.5	7	110	26	1.85	22.5	105

+ Section is not given in BS 4-1: 1993.
# Check availability.

BS 5950-1: 2000
BS 4-1: 1993

Table H.64 *Continued*



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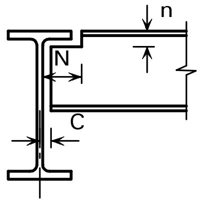
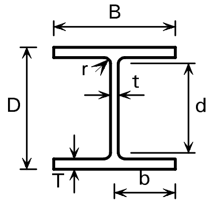
Dimensions and properties



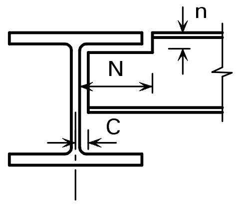
Section Designation	Mass per Metre	Depth of Section D	Width of Section B	Thickness: Web t	Thickness: Flange T	Root Radius r	Depth between Fillets d	Dimensions for detailing: End Clearance C	Notch N	Notch n	Surface Area Per Metre	Surface Area Per Tonne	Area of Section A
	kg/m	mm	mm	mm	mm	mm	mm	mm	mm	mm	m^2	m^2	cm^2
457x191x98	98.3	467.2	192.8	11.4	19.6	10.2	407.6	8	102	30	1.67	16.9	125
457x191x89	89.3	463.4	191.9	10.5	17.7	10.2	407.6	7	102	28	1.66	18.5	114
457x191x82	82.0	460.0	191.3	9.9	16.0	10.2	407.6	7	102	28	1.65	20.1	104
457x191x74	74.3	457.0	190.4	9.0	14.5	10.2	407.6	7	102	26	1.64	22.1	94.6
457x191x67	67.1	453.4	189.9	8.5	12.7	10.2	407.6	6	102	24	1.63	24.3	85.5
457x152x82	82.1	465.8	155.3	10.5	18.9	10.2	407.6	7	84	30	1.51	18.4	105
457x152x74	74.2	462.0	154.4	9.6	17.0	10.2	407.6	7	84	28	1.50	20.3	94.5
457x152x67	67.2	458.0	153.8	9.0	15.0	10.2	407.6	7	84	26	1.50	22.3	85.6
457x152x60	59.8	454.6	152.9	8.1	13.3	10.2	407.6	6	84	24	1.49	24.9	76.2
457x152x52	52.3	449.8	152.4	7.6	10.9	10.2	407.6	6	84	22	1.48	28.2	66.6
406x178x74	74.2	412.8	179.5	9.5	16.0	10.2	360.4	7	96	28	1.51	20.3	94.5
406x178x67	67.1	409.4	178.8	8.8	14.3	10.2	360.4	6	96	26	1.50	22.3	85.5
406x178x60	60.1	406.4	177.9	7.9	12.8	10.2	360.4	6	96	24	1.49	24.8	76.5
406x178x54	54.1	402.6	177.7	7.7	10.9	10.2	360.4	6	96	22	1.48	27.4	69.0
406x140x46	46.0	403.2	142.2	6.8	11.2	10.2	360.4	5	78	22	1.34	29.2	58.6
406x140x39	39.0	398.0	141.8	6.4	8.6	10.2	360.4	5	78	20	1.33	34.2	49.7
356x171x67	67.1	363.4	173.2	9.1	15.7	10.2	311.6	7	94	26	1.38	20.6	85.5
356x171x57	57.0	358.0	172.2	8.1	13.0	10.2	311.6	6	94	24	1.37	24.1	72.6
356x171x51	51.0	355.0	171.5	7.4	11.5	10.2	311.6	6	94	22	1.36	26.7	64.9
356x171x45	45.0	351.4	171.1	7.0	9.7	10.2	311.6	6	94	20	1.36	30.1	57.3
356x127x39	39.1	353.4	126.0	6.6	10.7	10.2	311.6	5	70	22	1.18	30.2	49.8
356x127x33	33.1	349.0	125.4	6.0	8.5	10.2	311.6	5	70	20	1.17	35.4	42.1
305x165x54	54.0	310.4	166.9	7.9	13.7	8.9	265.2	6	90	24	1.26	23.3	68.8
305x165x46	46.1	306.6	165.7	6.7	11.8	8.9	265.2	5	90	22	1.25	27.1	58.7
305x165x40	40.3	303.4	165.0	6.0	10.2	8.9	265.2	5	90	20	1.24	30.8	51.3
305x127x48	48.1	311.0	125.3	9.0	14.0	8.9	265.2	7	70	24	1.09	22.7	61.2
305x127x42	41.9	307.2	124.3	8.0	12.1	8.9	265.2	6	70	22	1.08	25.8	53.4
305x127x37	37.0	304.4	123.4	7.1	10.7	8.9	265.2	6	70	20	1.07	29.0	47.2
305x102x33	32.8	312.7	102.4	6.6	10.8	7.6	275.9	5	58	20	1.01	30.8	41.8
305x102x28	28.2	308.7	101.8	6.0	8.8	7.6	275.9	5	58	18	1.00	35.4	35.9
305x102x25	24.8	305.1	101.6	5.8	7.0	7.6	275.9	5	58	16	0.992	40.0	31.6
254x146x43	43.0	259.6	147.3	7.2	12.7	7.6	219.0	6	82	22	1.08	25.1	54.8
254x146x37	37.0	256.0	146.4	6.3	10.9	7.6	219.0	5	82	20	1.07	29.0	47.2
254x146x31	31.1	251.4	146.1	6.0	8.6	7.6	219.0	5	82	18	1.06	34.2	39.7
254x102x28	28.3	260.4	102.2	6.3	10.0	7.6	225.2	5	58	18	0.904	31.9	36.1
254x102x25	25.2	257.2	101.9	6.0	8.4	7.6	225.2	5	58	16	0.897	35.6	32.0
254x102x22	22.0	254.0	101.6	5.7	6.8	7.6	225.2	5	58	16	0.890	40.5	28.0
203x133x30	30.0	206.8	133.9	6.4	9.6	7.6	172.4	5	74	18	0.923	30.8	38.2
203x133x25	25.1	203.2	133.2	5.7	7.8	7.6	172.4	5	74	16	0.915	36.4	32.0
203x102x23	23.1	203.2	101.8	5.4	9.3	7.6	169.4	5	60	18	0.790	34.2	29.4
178x102x19	19.0	177.8	101.2	4.8	7.9	7.6	146.8	4	60	16	0.738	38.8	24.3
152x89x16	16.0	152.4	88.7	4.5	7.7	7.6	121.8	4	54	16	0.638	39.8	20.3
127x76x13	13.0	127.0	76.0	4.0	7.6	7.6	96.6	4	46	16	0.537	41.3	16.5

BS 5950-1: 2000
BS 4-1: 1993

Table H.65



UNIVERSAL COLUMNS



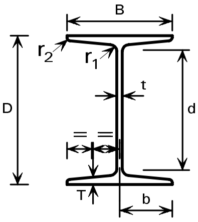
Dimensions and properties

Section Designation	Mass per Metre	Depth of Section D	Width of Section B	Thickness		Root Radius r	Depth between Fillets d	Dimensions for detailing			Surface Area		Area of Section A
				Web t	Flange T			End Clearance C	Notch N	Notch n	Per Metre	Per Tonne	
	kg/m	mm	mm	mm	mm	mm	mm	mm	mm	mm	m^2	m^2	cm^2
356x406x634 #	633.9	474.6	424.0	47.6	77.0	15.2	290.2	26	200	94	2.52	3.98	808
356x406x551 #	551.0	455.6	418.5	42.1	67.5	15.2	290.2	23	200	84	2.47	4.49	702
356x406x467 #	467.0	436.6	412.2	35.8	58.0	15.2	290.2	20	200	74	2.42	5.19	595
356x406x393 #	393.0	419.0	407.0	30.6	49.2	15.2	290.2	17	200	66	2.38	6.05	501
356x406x340 #	339.9	406.4	403.0	26.6	42.9	15.2	290.2	15	200	60	2.35	6.90	433
356x406x287 #	287.1	393.6	399.0	22.6	36.5	15.2	290.2	13	200	52	2.31	8.05	366
356x406x235 #	235.1	381.0	394.8	18.4	30.2	15.2	290.2	11	200	46	2.28	9.69	299
356x368x202 #	201.9	374.6	374.7	16.5	27.0	15.2	290.2	10	190	44	2.19	10.8	257
356x368x177 #	177.0	368.2	372.6	14.4	23.8	15.2	290.2	9	190	40	2.17	12.3	226
356x368x153 #	152.9	362.0	370.5	12.3	20.7	15.2	290.2	8	190	36	2.16	14.1	195
356x368x129 #	129.0	355.6	368.6	10.4	17.5	15.2	290.2	7	190	34	2.14	16.6	164
305x305x283	282.9	365.3	322.2	26.8	44.1	15.2	246.7	15	158	60	1.94	6.86	360
305x305x240	240.0	352.5	318.4	23.0	37.7	15.2	246.7	14	158	54	1.91	7.94	306
305x305x198	198.1	339.9	314.5	19.1	31.4	15.2	246.7	12	158	48	1.87	9.46	252
305x305x158	158.1	327.1	311.2	15.8	25.0	15.2	246.7	10	158	42	1.84	11.6	201
305x305x137	136.9	320.5	309.2	13.8	21.7	15.2	246.7	9	158	38	1.82	13.3	174
305x305x118	117.9	314.5	307.4	12.0	18.7	15.2	246.7	8	158	34	1.81	15.3	150
305x305x97	96.9	307.9	305.3	9.9	15.4	15.2	246.7	7	158	32	1.79	18.5	123
254x254x167	167.1	289.1	265.2	19.2	31.7	12.7	200.3	12	134	46	1.58	9.45	213
254x254x132	132.0	276.3	261.3	15.3	25.3	12.7	200.3	10	134	38	1.55	11.7	168
254x254x107	107.1	266.7	258.8	12.8	20.5	12.7	200.3	8	134	34	1.52	14.2	136
254x254x89	88.9	260.3	256.3	10.3	17.3	12.7	200.3	7	134	30	1.50	16.9	113
254x254x73	73.1	254.1	254.6	8.6	14.2	12.7	200.3	6	134	28	1.49	20.4	93.1
203x203x86	86.1	222.2	209.1	12.7	20.5	10.2	160.8	8	110	32	1.24	14.4	110
203x203x71	71.0	215.8	206.4	10.0	17.3	10.2	160.8	7	110	28	1.22	17.2	90.4
203x203x60	60.0	209.6	205.8	9.4	14.2	10.2	160.8	7	110	26	1.21	20.1	76.4
203x203x52	52.0	206.2	204.3	7.9	12.5	10.2	160.8	6	110	24	1.20	23.0	66.3
203x203x46	46.1	203.2	203.6	7.2	11.0	10.2	160.8	6	110	22	1.19	25.8	58.7
152x152x37	37.0	161.8	154.4	8.0	11.5	7.6	123.6	6	84	20	0.912	24.7	47.1
152x152x30	30.0	157.6	152.9	6.5	9.4	7.6	123.6	5	84	18	0.901	30.0	38.3
152x152x23	23.0	152.4	152.2	5.8	6.8	7.6	123.6	5	84	16	0.889	38.7	29.2

Check availability.

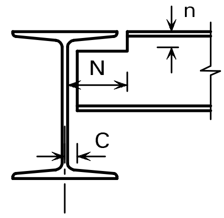
BS 5950-1: 2000
BS 4-1: 1993

Table H.66



JOISTS

Dimensions and properties



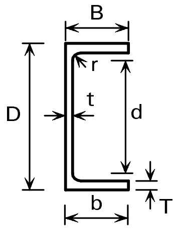
Section Designation	Mass per Metre	Depth of Section D	Width of Section B	Thickness		Radii		Depth between Fillets d	Dimensions for detailing			Surface Area		Area of Section A
				Web t	Flange T	Root r_1	Toe r_2		End Clearance C	Notch		Per Metre	Per Tonne	
										N	n			
	kg/m	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	m^2	m^2	cm^2
254x203x82 #	82.0	254.0	203.2	10.2	19.9	19.6	9.7	166.6	7	104	44	1.21	14.8	105
254x114x37 ‡	37.2	254.0	114.3	7.6	12.8	12.4	6.1	199.3	6	60	28	0.899	24.2	47.3
203x152x52 #	52.3	203.2	152.4	8.9	16.5	15.5	7.6	133.2	6	78	36	0.932	17.8	66.6
152x127x37 #	37.3	152.4	127.0	10.4	13.2	13.5	6.6	94.3	7	66	30	0.737	19.8	47.5
127x114x29 #	29.3	127.0	114.3	10.2	11.5	9.9	4.8	79.5	7	60	24	0.646	22.0	37.4
127x114x27 #	26.9	127.0	114.3	7.4	11.4	9.9	5.0	79.5	6	60	24	0.650	24.2	34.2
127x76x16 ‡	16.5	127.0	76.2	5.6	9.6	9.4	4.6	86.5	5	42	22	0.512	31.0	21.1
114x114x27 ‡	27.1	114.3	114.3	9.5	10.7	14.2	3.2	60.8	7	60	28	0.618	22.8	34.5
102x102x23 #	23.0	101.6	101.6	9.5	10.3	11.1	3.2	55.2	7	54	24	0.549	23.9	29.3
102x44x7 #	7.5	101.6	44.5	4.3	6.1	6.9	3.3	74.6	4	28	14	0.350	46.6	9.50
89x89x19 #	19.5	88.9	88.9	9.5	9.9	11.1	3.2	44.2	7	46	24	0.476	24.4	24.9
76x76x15 ‡	15.0	76.2	80.0	8.9	8.4	9.4	4.6	38.1	6	42	20	0.419	27.9	19.1
76x76x13 #	12.8	76.2	76.2	5.1	8.4	9.4	4.6	38.1	5	42	20	0.411	32.1	16.2

‡ Not available from some leading producers. Check availability.

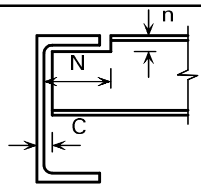
Check availability.

BS 5950-1: 2000
BS 4-1: 1993

Table H.67



PARRALLEL FLANGE CHANNELS



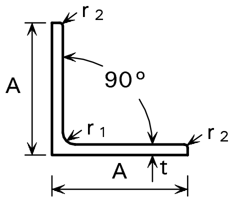
Dimensions and properties

Section Designation	Mass per Metre	Depth of Section D	Width of Section B	Thickness		Root Radius	Depth between Fillets	Dimensions for detailing			Surface Area		Area of Section
				Web t	Flange T	r	d	End Clearance C	Notch N	Notch n	Per Metre	Per Tonne	A
	kg/m	mm	mm	mm	mm	mm	mm	mm	mm	mm	m^2	m^2	cm^2
430x100x64	64.4	430	100	11.0	19.0	15	362	13	96	36	1.23	19.0	82.1
380x100x54	54.0	380	100	9.5	17.5	15	315	12	98	34	1.13	20.9	68.7
300x100x46	45.5	300	100	9.0	16.5	15	237	11	98	32	0.969	21.3	58.0
300x90x41	41.4	300	90	9.0	15.5	12	245	11	88	28	0.932	22.5	52.7
260x90x35	34.8	260	90	8.0	14.0	12	208	10	88	28	0.854	24.5	44.4
260x75x28	27.6	260	75	7.0	12.0	12	212	9	74	26	0.796	28.8	35.1
230x90x32	32.2	230	90	7.5	14.0	12	178	10	90	28	0.795	24.7	41.0
230x75x26	25.7	230	75	6.5	12.5	12	181	9	76	26	0.737	28.7	32.7
200x90x30	29.7	200	90	7.0	14.0	12	148	9	90	28	0.736	24.8	37.9
200x75x23	23.4	200	75	6.0	12.5	12	151	8	76	26	0.678	28.9	29.9
180x90x26	26.1	180	90	6.5	12.5	12	131	9	90	26	0.697	26.7	33.2
180x75x20	20.3	180	75	6.0	10.5	12	135	8	76	24	0.638	31.4	25.9
150x90x24	23.9	150	90	6.5	12.0	12	102	9	90	26	0.637	26.7	30.4
150x75x18	17.9	150	75	5.5	10.0	12	106	8	76	24	0.579	32.4	22.8
125x65x15 #	14.8	125	65	5.5	9.5	12	82.0	8	66	22	0.489	33.1	18.8
100x50x10 #	10.2	100	50	5.0	8.5	9	65.0	7	52	18	0.382	37.5	13.0

Check availability.

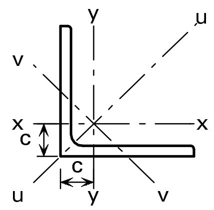
BS 5950-1: 2000
BS EN 10056-1: 1999

Table H.68



EQUAL ANGLES

Dimensions and properties

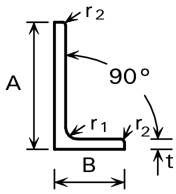


Section Designation		Mass per Metre	Radius		Dimension	Area of Section
Size A x A	Thickness t		Root r_1	Toe r_2	c	
mm	mm	kg/m	mm	mm	cm	cm^2
200x200	24 #	71.1	18.0	9.00	5.84	90.6
	20	59.9	18.0	9.00	5.68	76.3
	18	54.3	18.0	9.00	5.60	69.1
	16	48.5	18.0	9.00	5.52	61.8
150x150	18 #	40.1	16.0	8.00	4.38	51.2
	15	33.8	16.0	8.00	4.25	43.0
	12	27.3	16.0	8.00	4.12	34.8
	10	23.0	16.0	8.00	4.03	29.3
120x120	15 #	26.6	13.0	6.50	3.52	34.0
	12	21.6	13.0	6.50	3.40	27.5
	10	18.2	13.0	6.50	3.31	23.2
	8 #	14.7	13.0	6.50	3.24	18.8
100x100	15 #	21.9	12.0	6.00	3.02	28.0
	12	17.8	12.0	6.00	2.90	22.7
	10	15.0	12.0	6.00	2.82	19.2
	8	12.2	12.0	6.00	2.74	15.5
90x90	12 #	15.9	11.0	5.50	2.66	20.3
	10	13.4	11.0	5.50	2.58	17.1
	8	10.9	11.0	5.50	2.50	13.9
	7 #	9.61	11.0	5.50	2.45	12.2
80x80	10 ‡	11.9	10.0	5.00	2.34	15.1
	8 ‡	9.63	10.0	5.00	2.26	12.3
75x75	8 ‡	8.99	9.00	4.50	2.14	11.4
	6 ‡	6.85	9.00	4.50	2.05	8.73
70x70	7 ‡	7.38	9.00	4.50	1.97	9.40
	6 ‡	6.38	9.00	4.50	1.93	8.13
65x65	7 ‡	6.83	9.00	4.50	2.05	8.73
60x60	8 ‡	7.09	8.00	4.00	1.77	9.03
	6 ‡	5.42	8.00	4.00	1.69	6.91
	5 ‡	4.57	8.00	4.00	1.64	5.82
50x50	6 ‡	4.47	7.00	3.50	1.45	5.69
	5 ‡	3.77	7.00	3.50	1.40	4.80
	4 ‡	3.06	7.00	3.50	1.36	3.89
45x45	4.5 ‡	3.06	7.00	3.50	1.25	3.90
40x40	5 ‡	2.97	6.00	3.00	1.16	3.79
	4 ‡	2.42	6.00	3.00	1.12	3.08
35x35	4 ‡	2.09	5.00	2.50	1.00	2.67
30x30	4 ‡	1.78	5.00	2.50	0.878	2.27
	3 ‡	1.36	5.00	2.50	0.835	1.74
25x25	4 ‡	1.45	3.50	1.75	0.762	1.85
	3 ‡	1.12	3.50	1.75	0.723	1.42
20x20	3 ‡	0.882	3.50	1.75	0.598	1.12

‡ Not available from some leading producers. Check availability.
# Check availability.
c is the distance from the back of the leg to the centre of gravity.

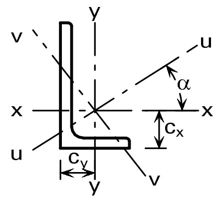
BS 5950-1: 2000
BS EN 10056-1: 1999

Table H.69



UNEQUAL ANGLES

Dimensions and properties



Section Designation		Mass per Metre	Radius		Dimension		Angle Axis x-x to Axis u-u	Area of Section
Size	Thickness		Root	Toe				
A x B	t		r_1	r_2	c_x	c_y	Tan α	
mm	mm	kg/m	mm	mm	cm	cm		cm^2
200x150	18 #	47.1	15.0	7.50	6.34	3.86	0.549	60.1
	15	39.6	15.0	7.50	6.21	3.73	0.551	50.5
	12	32.0	15.0	7.50	6.08	3.61	0.552	40.8
200x100	15	33.8	15.0	7.50	7.16	2.22	0.260	43.0
	12	27.3	15.0	7.50	7.03	2.10	0.262	34.8
	10	23.0	15.0	7.50	6.93	2.01	0.263	29.2
150x90	15	33.9	12.0	6.00	5.21	2.23	0.354	33.9
	12	21.6	12.0	6.00	5.08	2.12	0.358	27.5
	10	18.2	12.0	6.00	5.00	2.04	0.360	23.2
150x75	15	24.8	12.0	6.00	5.52	1.81	0.253	31.7
	12	20.2	12.0	6.00	5.40	1.69	0.258	25.7
	10	17.0	12.0	6.00	5.31	1.61	0.261	21.7
125x75	12	17.8	11.0	5.50	4.31	1.84	0.354	22.7
	10	15.0	11.0	5.50	4.23	1.76	0.357	19.1
	8	12.2	11.0	5.50	4.14	1.68	0.360	15.5
100x75	12	15.4	10.0	5.00	3.27	2.03	0.540	19.7
	10	13.0	10.0	5.00	3.19	1.95	0.544	16.6
	8	10.6	10.0	5.00	3.10	1.87	0.547	13.5
100x65	10 #	12.3	10.0	5.00	3.36	1.63	0.410	15.6
	8 #	9.94	10.0	5.00	3.27	1.55	0.413	12.7
	7 #	8.77	10.0	5.00	3.23	1.51	0.415	11.2
100x50	8 ‡	8.97	8.00	4.00	3.60	1.13	0.258	11.4
	6 ‡	6.84	8.00	4.00	3.51	1.05	0.262	8.71
80x60	7 ‡	7.36	8.00	4.00	2.51	1.52	0.546	9.38
80x40	8 ‡	7.07	7.00	3.50	2.94	0.963	0.253	9.01
	6 ‡	5.41	7.00	3.50	2.85	0.884	0.258	6.89
75x50	8 ‡	7.39	7.00	3.50	2.52	1.29	0.430	9.41
	6 ‡	5.65	7.00	3.50	2.44	1.21	0.435	7.19
70x50	6 ‡	5.41	7.00	3.50	2.23	1.25	0.500	6.89
65x50	5 ‡	4.35	6.00	3.00	1.99	1.25	0.577	5.54
60x40	6 ‡	4.46	6.00	3.00	2.00	1.01	0.431	5.68
	5 ‡	3.76	6.00	3.00	1.96	0.972	0.434	4.79
60x30	5 ‡	3.36	5.00	2.50	2.17	0.684	0.257	4.28
50x30	5 ‡	2.96	5.00	2.50	1.73	0.741	0.352	3.78
45x30	4 ‡	2.25	4.50	2.25	1.48	0.740	0.436	2.87
40x25	4 ‡	1.93	4.00	2.00	1.36	0.623	0.380	2.46
40x20	4 ‡	1.77	4.00	2.00	1.47	0.480	0.252	2.26
30x20	4 ‡	1.46	4.00	2.00	1.03	0.541	0.421	1.86
	3 ‡	1.12	4.00	2.00	0.990	0.502	0.427	1.43

‡ Not available from some leading producers. Check availability.

Check availability.

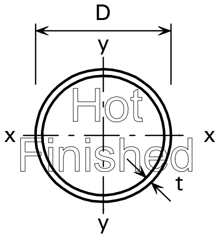
c_x is the distance from the back of the short leg to the centre of gravity.

c_y is the distance from the back of the long leg to the centre of gravity.

BS 5950-1: 2000
BS EN 10210-2: 1997

Table H.70

HOT-FINISHED
CIRCULAR HOLLOW SECTIONS



Dimensions and properties

Section Designation		Mass per Metre	Area of Section	Surface Area	
Outside Diameter D mm	Thickness t mm	kg/m	A cm^2	Per Metre m^2	Per Tonne m^2
26.9	3.2 ~	1.87	2.38	0.0845	45.2
42.4	3.2 ~	3.09	3.94	0.133	43.0
48.3	3.2 ~	3.56	4.53	0.152	42.7
	4.0 ~	4.37	5.57	0.152	34.8
	5.0 ~	5.34	6.80	0.152	28.5
60.3	3.2 ~	4.51	5.74	0.189	41.9
	5.0 ~	6.82	8.69	0.189	27.7
76.1	2.9 ^	5.24	6.67	0.239	45.6
	3.2 ~	5.75	7.33	0.239	41.6
	4.0 ~	7.11	9.06	0.239	33.6
	5.0 ~	8.77	11.2	0.239	27.3
88.9	3.2 ~	6.76	8.62	0.279	41.3
	4.0 ~	8.38	10.7	0.279	33.3
	5.0 ~	10.4	13.2	0.279	27.0
	6.3 ~	12.8	16.3	0.279	21.7
114.3	3.2 ~	8.77	11.2	0.359	40.9
	3.6	9.83	12.5	0.359	36.5
	5.0	13.5	17.2	0.359	26.6
	6.3	16.8	21.4	0.359	21.4
139.7	5.0	16.6	21.2	0.439	26.4
	6.3	20.7	26.4	0.439	21.2
	8.0 ~	26.0	33.1	0.439	16.9
	10.0 ~	32.0	40.7	0.439	13.7
168.3	5.0 ~	20.1	25.7	0.529	26.3
	6.3 ~	25.2	32.1	0.529	21.0
	8.0 ~	31.6	40.3	0.529	16.7
	10.0 ~	39.0	49.7	0.529	13.6
193.7	5.0 ~	23.3	29.6	0.609	26.1
	6.3 ~	29.1	37.1	0.609	20.9
	8.0 ~	36.6	46.7	0.609	16.6
	10.0 ~	45.3	57.7	0.609	13.4

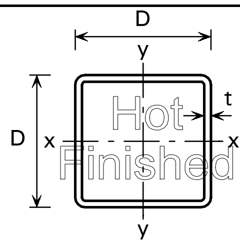
Section Designation		Mass per Metre	Area of Section	Surface Area	
Outside Diameter D mm	Thickness t mm	kg/m	A cm^2	Per Metre m^2	Per Tonne m^2
219.1	5.0 ~	26.4	33.6	0.688	26.1
	6.3 ~	33.1	42.1	0.688	20.8
	8.0 ~	41.6	53.1	0.688	16.5
	10.0 ~	51.6	65.7	0.688	13.3
	12.5 ~	63.7	81.1	0.688	10.8
244.5	12.0 ~	68.8	87.7	0.768	11.2
273.0	5.0 ~	33.0	42.1	0.858	26.0
	6.3	41.4	52.8	0.858	20.7
	8.0 ~	52.3	66.6	0.858	16.4
	10.0	64.9	82.6	0.858	13.2
	12.5	80.3	102	0.858	10.7
	16.0 ~	101	129	0.858	8.46
323.9	6.3 ~	49.3	62.9	1.02	20.7
	8.0 ~	62.3	79.4	1.02	16.4
	10.0 ~	77.4	98.6	1.02	13.2
	12.5 ~	96.0	122	1.02	10.6
	16.0	122	155	1.02	8.40
406.4	6.3 ~	62.2	79.2	1.28	20.6
	8.0 ~	78.6	100	1.28	16.3
	10.0 ~	97.8	125	1.28	13.1
	12.5 ~	121	155	1.28	10.5
	16.0	154	196	1.28	8.31
457.0	8.0 ~	88.6	113	1.44	16.3
	10.0 ~	110	140	1.44	13.1
	12.5 ~	137	175	1.44	10.5
	16.0	174	222	1.44	8.28
508.0	8.0 ~	98.6	126	1.60	16.2
	10.0 ~	123	156	1.60	13.0
	12.5	153	195	1.60	10.5
	16.0 ~	194	247	1.60	8.25
	20.0 ~	241	307	1.60	6.64

~ Check availability in S275.
^ Check availability in S355.

BS 5950-1: 2000
BS EN 10210-2: 1997

Table H.71

HOT-FINISHED SQUARE HOLLOW SECTIONS



Dimensions and properties

Section Designation Size D x D mm	Section Designation Thickness t mm	Mass per Metre kg/m	Area of Section A cm^2	Surface Area Per Metre m^2	Surface Area Per Tonne m^2
50x50	3.0 ~	4.35	5.54	0.192	44.1
	3.2 ~	4.62	5.88	0.192	41.6
	4.0	5.64	7.19	0.190	33.7
	5.0	6.85	8.73	0.187	27.3
	6.3 ~	8.31	10.6	0.184	22.1
60x60	3.0 ~	5.29	6.74	0.232	43.9
	3.2 ~	5.62	7.16	0.232	41.3
	4.0 ~	6.90	8.79	0.230	33.3
	5.0	8.42	10.7	0.227	27.0
	6.3 ~	10.3	13.1	0.224	21.7
	8.0 ~	12.5	16.0	0.219	17.5
70x70	3.6 ~	7.40	9.42	0.271	36.6
	5.0 ~	9.99	12.7	0.267	26.7
	6.3 ~	12.3	15.6	0.264	21.5
	8.0 ~	15.0	19.2	0.259	17.3
80x80	3.6 ~	8.53	10.9	0.311	36.5
	4.0 ~	9.41	12.0	0.310	32.9
	5.0 ~	11.6	14.7	0.307	26.6
	6.3	14.2	18.1	0.304	21.4
	8.0 ~	17.5	22.4	0.299	17.1
90x90	3.6 ~	9.66	12.3	0.351	36.3
	4.0 ~	10.7	13.6	0.350	32.7
	5.0 ~	13.1	16.7	0.347	26.5
	6.3 ~	16.2	20.7	0.344	21.2
	8.0 ~	20.1	25.6	0.339	16.9
100x100	4.0	11.9	15.2	0.390	32.8
	5.0	14.7	18.7	0.387	26.3
	6.3	18.2	23.2	0.384	21.1
	8.0 ~	22.6	28.8	0.379	16.8
	10.0	27.4	34.9	0.374	13.6
120x120	5.0 ~	17.8	22.7	0.467	26.2
	6.3	22.2	28.2	0.464	20.9
	8.0 ~	27.6	35.2	0.459	16.6
	10.0 ~	33.7	42.9	0.454	13.5
	12.5 ~	40.9	52.1	0.448	11.0
140x140	5.0 ~	21.0	26.7	0.547	26.0
	6.3 ~	26.1	33.3	0.544	20.8
	8.0 ~	32.6	41.6	0.539	16.5
	10.0 ~	40.0	50.9	0.534	13.4
	12.5 ~	48.7	62.1	0.528	10.8

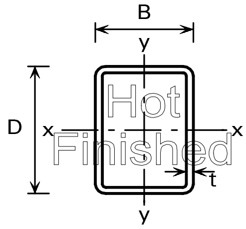
Section Designation Size D x D mm	Section Designation Thickness t mm	Mass per Metre kg/m	Area of Section A cm^2	Surface Area Per Metre m^2	Surface Area Per Tonne m^2
150x150	5.0 ~	22.6	28.7	0.587	26.0
	6.3	28.1	35.8	0.584	20.8
	8.0 ~	35.1	44.8	0.579	16.5
	10.0	43.1	54.9	0.574	13.3
	12.5 ~	52.7	67.1	0.568	10.8
	16.0 ~	65.2	83.0	0.559	8.57
160x160	5.0 ~	24.1	30.7	0.627	26.0
	6.3 ~	30.1	38.3	0.624	20.7
	8.0 ~	37.6	48.0	0.619	16.5
	10.0 ~	46.3	58.9	0.614	13.3
	12.5 ~	56.6	72.1	0.608	10.7
180x180	6.3 ~	34.0	43.3	0.704	20.7
	8.0 ~	42.7	54.4	0.699	16.4
	10.0 ~	52.5	66.9	0.694	13.2
	12.5 ~	64.4	82.1	0.688	10.7
	16.0 ~	80.2	102	0.679	8.47
200x200	5.0 ~	30.4	38.7	0.787	25.9
	6.3 ~	38.0	48.4	0.784	20.6
	8.0 ~	47.7	60.8	0.779	16.3
	10.0	58.8	74.9	0.774	13.2
	12.5 ~	72.3	92.1	0.768	10.6
	16.0 ~	90.3	115	0.759	8.41
250x250	6.3 ~	47.9	61.0	0.984	20.5
	8.0 ~	60.3	76.8	0.979	16.2
	10.0 ~	74.5	94.9	0.974	13.1
	12.5 ~	91.9	117	0.968	10.5
	16.0 ~	115	147	0.959	8.31
300x300	6.3 ~	57.8	73.6	1.18	20.4
	8.0 ~	72.8	92.8	1.18	16.2
	10.0 ~	90.2	115	1.17	13.0
	12.5 ~	112	142	1.17	10.5
	16.0 ~	141	179	1.16	8.26
350x350	8.0 ~	85.4	109	1.38	16.2
	10.0 ~	106	135	1.37	12.9
	12.5 ~	131	167	1.37	10.5
	16.0 ~	166	211	1.36	8.19
400x400	10.0 ~	122	155	1.57	12.9
	12.5	151	192	1.57	10.4
	16.0 ~	191	243	1.56	8.17
	20.0 ~	235	300	1.55	6.60

~ Check availability in S275.

BS 5950-1: 2000
BS EN 10210-2: 1997

Table H.72

HOT-FINISHED
RECTANGULAR HOLLOW SECTIONS



Dimensions and properties

Section Designation		Mass per Metre	Area of Section	Surface Area	
Size D x B mm	Thickness t mm	kg/m	A cm^2	Per Metre m^2	Per Tonne m^2
50x30	3.2 ~	3.61	4.60	0.152	42.1
60x40	3.0 ~	4.35	5.54	0.192	44.1
	4.0 ~	5.64	7.19	0.190	33.7
	5.0 ~	6.85	8.73	0.187	27.3
80x40	3.2 ~	5.62	7.16	0.232	41.3
	4.0 ~	6.90	8.79	0.230	33.3
	5.0 ~	8.42	10.7	0.227	27.0
	6.3 ~	10.3	13.1	0.224	21.7
	8.0 ~	12.5	16.0	0.219	17.5
90x50	3.6 ~	7.40	9.42	0.271	36.6
	5.0 ~	9.99	12.7	0.267	26.7
	6.3 ~	12.3	15.6	0.264	21.5
100x50	3.0 ~	6.71	8.54	0.292	43.5
	3.2 ~	7.13	9.08	0.292	41.0
	5.0 ~	10.8	13.7	0.287	26.6
	6.3 ~	13.3	16.9	0.284	21.4
	8.0 ~	16.3	20.8	0.279	17.1
	10.0 ^	19.6	24.9	0.274	14.0
100x60	3.6 ~	8.53	10.9	0.311	36.5
	5.0 ~	11.6	14.7	0.307	26.5
	6.3 ~	14.2	18.1	0.304	21.4
	8.0 ~	17.5	22.4	0.299	17.1
120x60	3.6 ~	9.70	12.3	0.351	36.2
	5.0 ~	13.1	16.7	0.347	26.5
	6.3 ~	16.2	20.7	0.344	21.2
	8.0 ~	20.1	25.6	0.339	16.9
120x80	5.0 ~	14.7	18.7	0.387	26.3
	6.3	18.2	23.2	0.384	21.1
	8.0	22.6	28.8	0.379	16.8
	10.0 ~	27.4	34.9	0.374	13.6
150x100	5.0 ~	18.6	23.7	0.487	26.2
	6.3	23.1	29.5	0.484	21.0
	8.0 ~	28.9	36.8	0.479	16.6
	10.0 ~	35.3	44.9	0.474	13.4
	12.5 ~	42.8	54.6	0.468	10.9
160x80	4.0 ~	14.4	18.4	0.470	32.6
	5.0 ~	17.8	22.7	0.467	26.2
	6.3	22.2	28.2	0.464	20.9
	8.0	27.6	35.2	0.459	16.6
	10.0 ~	33.7	42.9	0.454	13.5
200x100	5.0 ~	22.6	28.7	0.587	26.0
	6.3 ~	28.1	35.8	0.584	20.8
	8.0	35.1	44.8	0.579	16.5
	10.0	43.1	54.9	0.574	13.3
	12.5	52.7	67.1	0.568	10.8
200x120	5.0 ~	24.1	30.7	0.627	26.0
	6.3 ~	30.1	38.3	0.624	20.7
	8.0 ~	37.6	48.0	0.619	16.5
	10.0 ~	46.3	58.9	0.614	13.3
200x150	8.0 ~	41.4	52.8	0.679	16.4
	10.0 ~	51.0	64.9	0.674	13.2
250x100	10.0 ~	51.0	64.9	0.674	13.2
	12.5 ~	62.5	79.6	0.668	10.7
250x150	5.0 ~	30.4	38.7	0.787	25.9
	6.3	38.0	48.4	0.784	20.6
	8.0 ~	47.7	60.8	0.779	16.3
	10.0 ~	58.8	74.9	0.774	13.2
	12.5	72.3	92.1	0.768	10.6
	16.0 ~	90.3	115	0.759	8.41
300x100	8.0 ~	47.7	60.8	0.779	16.3
	10.0 ~	58.8	74.9	0.774	13.2
300x200	6.3 ~	47.9	61.0	0.984	20.5
	8.0 ~	60.3	76.8	0.979	16.2
	10.0 ~	74.5	94.9	0.974	13.1
	12.5 ~	91.9	117	0.968	10.5
	16.0	115	147	0.959	8.34
400x200	8.0 ~	72.8	92.8	1.18	16.2
	10.0 ~	90.2	115	1.17	13.0
	12.5 ~	112	142	1.17	10.5
	16.0 ~	141	179	1.16	8.26
450x250	8.0 ~	85.4	109	1.38	16.2
	10.0 ~	106	135	1.37	12.9
	12.5 ~	131	167	1.37	10.5
	16.0	166	211	1.36	8.19
500x300	8.0 ~	98.0	125	1.58	16.1
	10.0 ~	122	155	1.57	12.9
	12.5 ~	151	192	1.57	10.4
	16.0 ~	191	243	1.56	8.17
	20.0 ~	235	300	1.55	6.60

~ Check availability in S275.
^ Check availability in S355.

Joints in steel construction: Simple Connections

(Publication P212, 2002)

(a) Corrigendum 1, October 2002
Tying Capacity of Fin Plate Connections with Single Line of Bolts

The values of tying capacity given in Table H.27 (pages 410 to 414) and Table H.29 (pages 420 to 424) should be amended to values that are the lesser of the tabulated values and the shear capacity of the bolt groups. The shear capacity of the bolt group = $n.P_S$ where n is the number of bolts and P_S is the shear capacity per bolt (= 91.9 kN for M20, grade 8.8 bolt from Table H.49).

The reason for this change is that, as stated in Table H.24, the tabulated tying capacities for fin plate connections were based on the minimum values from Checks 11(i), 11(ii), 12(i) and 12(ii). None of these checks relate to the shear capacity of the bolt group. Where there is a single line of bolts, the shear capacity of the bolt group may be less than the tabulated tying capacity.

When carrying out the full design procedure (in accordance with Section 6.5) an additional check for "structural integrity" should be made for the shear capacity of the bolt group. This additional check, which may be referred to as Check 13, is: Tie force ≤ $n.P_S$.

In practice these changes will only be of significance in the unusual case when the tie force is greater than the shear force on the beam.

(b) Corrigendum 2, July 2005
Shear Capacity and Minimum Support Thickness of Fin Plate Connections

(i) Shear capacity of the connection

In some cases the tabulated shear capacity values in P212 (2002) for fin plate connections to S355 beams (Tables H.29 and H.30) should be increased to take full account of the material strength of the beam. The changes should be made when check 2 (bearing of bolts on fin plate or beam web) is quoted as the critical design check. Where the capacity is increased, there will be a corresponding decrease in the maximum notch length.

The reason for this is that, even for S355 beams the tabulated values (P212, 2002) assumed S275 beam strengths for check 2.

(ii) Minimum support thickness

The tabulated minimum support thicknesses in P212 (2002), Tables H.27 to H.30 for fin plate connections should be amended. All the changes increase the value of the minimum support thickness.

There are three reasons for the amendment:

Firstly, and most significantly, in the calculation of the minimum support thickness (using check 10) the shear force (F_V) was not divided by two, to allow for the two shear planes either side of the fin plate.

Secondly, the shear force (F_V) was taken as the double notch shear capacity, whereas it should be taken as the un-notched/single notch shear capacity.

Thirdly, some shear capacity values have been amended, as noted in (a) above.

Corrected capacity tables, taking account of corrigendum 1 and corrigendum 2, are available on www.steelbiz.org.

(c) Supplementary capacity tables for flexible end plates

Supplementary capacity tables for flexible end plates (pages 387-S to 396-S) are available on www.steelbiz.org as part of advisory desk note AD 291.

The supplementary tables give the double notch shear capacity with the corresponding critical design check, based on a standard double notch length.