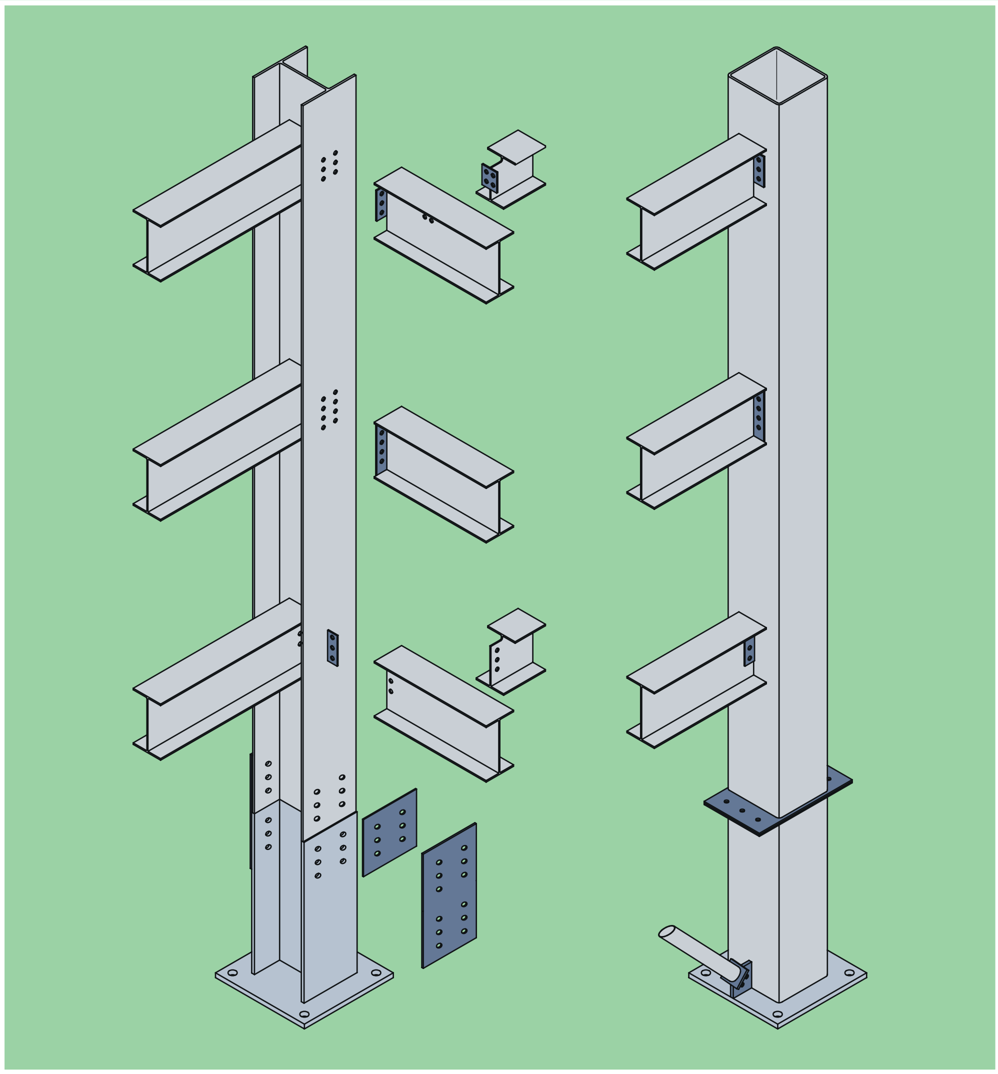
JOINTS IN STEEL CONSTRUCTION:

SIMPLE JOINTS TO EUROCODE 3



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Simple Joints to Eurocode 3

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FOREWORD

This publication is one of a series of “Green Books” that cover a range of steelwork connections. This publication provides guidance for nominally pinned joints that primarily carry vertical shear and, as an accidental limit state, tying forces, designed in accordance with Eurocode 3 and its UK National Annexes.

This publication is cited in the UK National Annex: joints designed in accordance with the principles within this publication can be classed as nominally pinned without calculation.

A companion publication (published in 2012) covers moment-resisting joints.

Guidance for nominally pinned joints designed in accordance with BS 5950 is available in publication P212 *Joints in Steel Construction; Simple Connections*.

The major changes in scope compared to P212 are:

- Double angle cleats are omitted from the current publication, as it was felt they are not commonly used in the UK.
- A new full depth end plate (i.e. welded to both flanges) has been introduced, which offers a significantly increased tying resistance compared to a partial depth end plate.
- The tying resistance of partial depth end plates is calculated using Eurocode provisions. The revised design model results in an increased tying resistance compared to P212.

BS EN 1993-1-8 has clear definitions for connections and joints: the terms ‘joint’ and ‘connection’ refer to the zone where members are interconnected and to the location where elements meet, respectively. In this publication, the distinction in terminology is not emphasised and “connection” is used more generally, reflecting traditional practice in the UK.

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

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The BCSA/SCI Connections Group members (2014) comprise:

Mike Banfi	Arup
David Brown	SCI
Tom Cosgrove	BCSA
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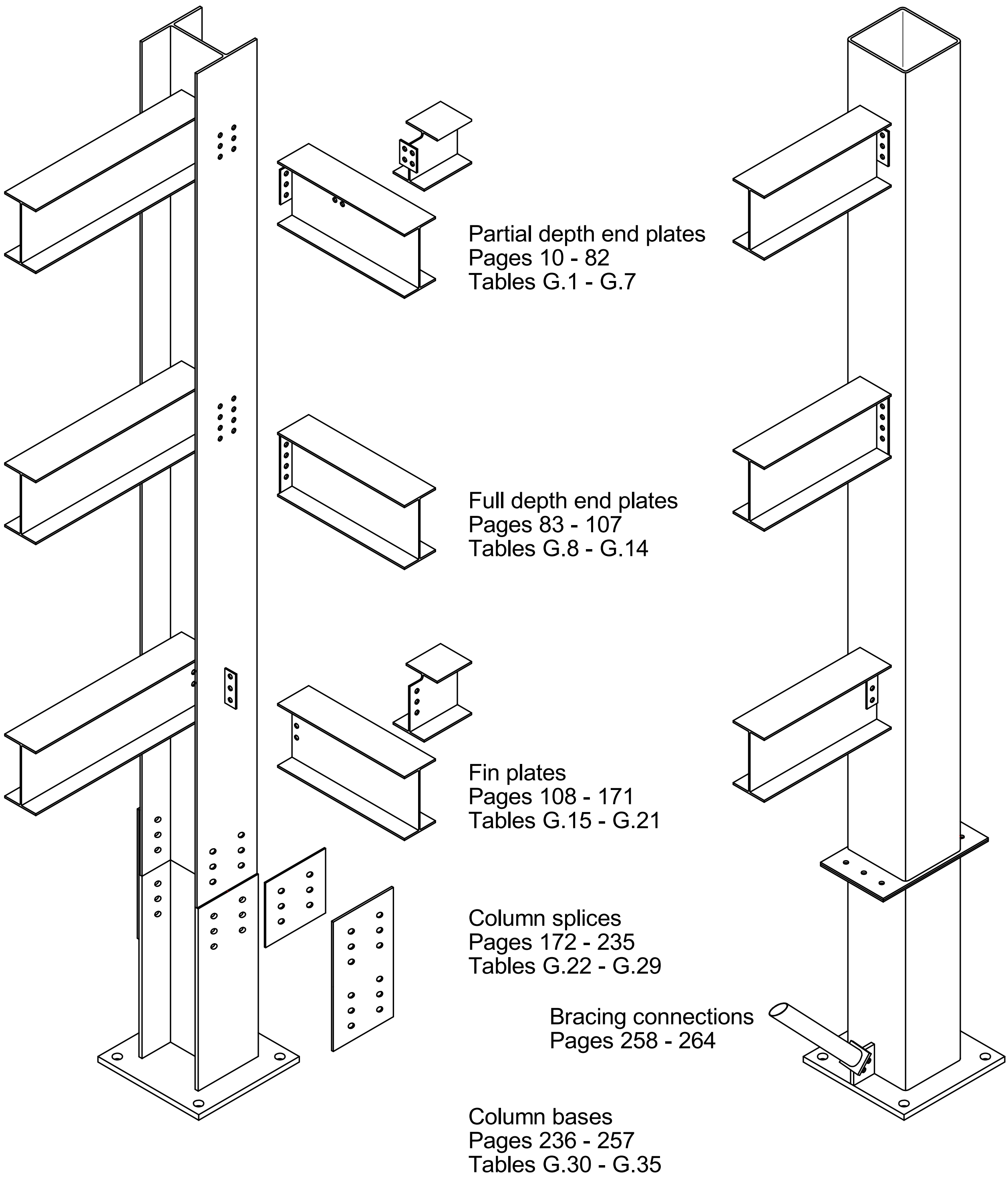
The primary drafter of this publication was Edurne Nunez Moreno, with assistance from Cyrill Tarbé (both formerly of SCI) and David Brown. Abdul Malik provided additional guidance.

The revisions in the 2014 reprint include revised tying resistances for full depth end plates for deeper beams, revised resistances for RHS bases, an additional example using blind bolts, revised guidance for bracing connections and typographical corrections. The revisions were completed by David Brown. Revisions are marked with a black line in the margin.

PICTORIAL INDEX



CONTENTS

1 INTRODUCTION

1.1 ABOUT THIS PUBLICATION

This publication provides procedures for designing joints in steel-framed structures in accordance with BS EN 1993-1-8[1] and its accompanying National Annex[2], and with BS EN 1993-1-1[3] and its National Annex[4].Connections between UK Beams (“universal beams”) and UK Columns (“universal columns”) using non-preloaded and preloaded bolts are included. Connections between UK Beams and hot finished structural hollow section columns using the Flowdrill and Hollo-Bolt systems are also included. Design procedures are provided for:

(a) Beam to beam and beam to column connections

- Partial depth end plates
- Full depth end plates
- Fin plates

(b) Column splices

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

(c) Column bases

Steel plates welded to column shafts.

(d) Bracing connections

General guidance is provided for connections between bracing members and main members, via a gusset plate. Typical details are shown.

(e) Special connections

General guidance is given on special connections, where, for example, members do not align on a common centreline, align at different levels or at an angle.

Steel grades

This publication covers steel conforming to BS EN 10025-2[5] and BS EN 10210[6].

Within this publication, reference is only made to the strength designation; the sub-grade is required for a complete specification.

Design procedures

Individual design procedures are included for all connection components. The procedures commence with the detailing requirements (joint geometry), then present the checks for each stage of the load transfer through the connection including welds, plates, bolts and the section webs or flanges as appropriate. The resistance checks on sections, welds and bolts are all based on Eurocode 3.

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

Resistance tables

Resistance tables for standard connections are provided in the yellow pages of this guide.

The resistance tables have been arranged so that the designer can simply select a connection and, with the minimum of calculation, check whether it has sufficient resistance.

Design examples

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

1.2 JOINT CONSIDERATIONS

Joint classification

According to BS EN 1993-1-8, nominally pinned joints:

(1) should be capable of transmitting the internal forces, without developing significant moments which might adversely affect the members or the structure as a whole and
(2) be capable of accepting the resulting rotations under the design loads

In addition, the joint must:

(3) provide the directional restraint to members which has been assumed in the member design
(4) have sufficient robustness to satisfy the structural integrity requirements (tying resistance).

BS EN 1993-1-8 requires that all joints must be classified; by stiffness, which is appropriate for elastic global analysis, or by strength, which is appropriate for rigid plastic global analysis, or by both stiffness and strength, which is appropriate for elastic-plastic global analysis.

Classification by stiffness: the initial rotational stiffness of the joint, calculated in accordance with BS EN 1993-1-8, 6.3.1 is compared with the classification boundaries given in BS EN 1993-1-8, 5.2.2.

Alternatively, joints may be classified based on experimental evidence, experience of previous satisfactory performance in similar cases or by calculations based on test evidence.

Classification by strength: the following two requirements must be satisfied in order to classify a joint as nominally pinned, based on its strength:

(1) the design moment resistance of the connection does not exceed 25% of the design moment resistance required for a full-strength joint

(2) the joint should be capable of accepting the rotations resulting from the design loads.

The UK National Annex to BS EN 1993-1-8 states that connections designed in accordance with this publication* may be classified as nominally pinned joints.

All the standard connections given in this publication may be classified as nominally pinned according to the strength requirements. Care should be taken before amending the standard details as the resulting connection may fall outside the provisions of the UK National Annex (UK NA). In particular:

- the rotation capacity of the standard fin plate details have been demonstrated by test
- the thickness of the full depth end plates have been limited to ensure the moment resistance is less than 25% of a full strength joint.

Structural integrity

The UK Building Regulations require that all buildings should be designed to avoid disproportionate collapse. Commonly, this is achieved by designing the joints in a steel frame (beam to column, beam to beam, beam through beam and column splices) for tying forces. Guidance on the design values of tying forces is given in BS EN 1991-1-7[7] Annex A, and its UK National Annex[8]. The requirements relate to the building Class, with a design value of horizontal tying force generally not less than 75 kN, and usually significantly higher. The full depth end plate details have been developed to provide an increased tying resistance compared to the partial depth details.

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

Connection 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 1.1.

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[9], enables reinforcement continuity to be allowed for in providing relatively simple full depth end plate connections with substantial moment resistance.

1.3 EXCHANGE OF INFORMATION

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

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

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

(3) The frame and its principal connections are designed by the Consulting Engineer.

If the frame and connections are not designed by a single authority, 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[10] gives guidance on the transfer of necessary information. 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 forces shown are factored or unfactored
- requirements for any particular type of fabrication detail and/or restriction on the type of connection to be used.

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 a Steelwork Contractor's workmanship rates can vary considerably and are dependant upon the level of investment in plant and machinery. However, the main objective is to reduce the work content. The material cost for fittings and bolts is 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.

* Strictly, the UK NA refers to P212[16], which accords with BS 5950. All the principles within P212 have been adopted in this publication; it is expected that the UK NA will refer to the present publication in due course.

1.5 SUSTAINABILITY

Standardised connections are efficient in their production. Steelwork Contractors can equip their workshops with specialist machinery that will increase the speed of fabrication, allowing them to produce fittings and prepare the members much more quickly than they would if the connection configuration were different each time.

The standardised details mean the steelwork is straightforward to erect, which provides a safer working environment for the steel erectors.

Due to the nature of most bolted joints, the connections are demountable at the end of the service life of the structure. The steelwork can be dismantled, reused or recycled, therefore reducing the environmental impact of the construction.

Table 1.1 Relative merits of beam end connection types

	Partial depth end plate	**Full depth end plate**	**Fin plate**
Design			
Shear resistance - percentage of beam resistance	Up to 75%	100%	Up to 50% *Up to 75% with two vertical lines of bolts*
Tying resistance	Fair	Good	Good
Special considerations			
Skewed Joints	Fair	Fair	Good
Beams eccentric to columns	Fair	Fair	Good
Connection to column webs	Good	Good	Fair *To facilitate erection, flange stripping may be required. Stiffening may be required for long fin plates* (Figure 5.6)
Fabrication and treatment			
Fabrication	Good	Good	Good *Stiffening may be required for long fin plates* (Figure 5.6)
Surface treatment	Good	Good	Good
Erection			
Ease of erection	Fair *Care needed for two-sided connections*	Fair *Care needed for two-sided connections*	Good
Site adjustment	Fair	Fair	Fair
Temporary stability	Fair	Good	Fair

1.6 MAJOR SYMBOLS

The major symbols used in this publication are listed below for reference purposes. Others are described where used.

A_v Shear area of element

a Effective throat of weld

b Width of section (Subscript c or b refers to column or beam. Subscript 1 refers to supported beam and subscript 2 refers to supporting element)

d Depth of web between fillets *or* diameter of a bolt

d_0 Hole diameter

e_1 End distance

e_2 Edge distance

$e_{2,b}$ Edge distance on beam side

f_y Yield strength of steel (Subscript c, b or p refers to column, beam or plate. Subscript 1 refers to supported beam and subscript 2 refers to supporting element)

f_u Ultimate tensile strength (Subscript c, b or p refers to column, beam or plate. Subscript 1 refers to supported beam and subscript 2 refers to supporting element)

f_{ub} Ultimate tensile strength of the bolt

F Tying force or resistance (subscripts Ed or Rd)

h Depth of section (Subscript c or b refers to column or beam. Subscript 1 refers to supported beam and subscript 2 refers to supporting element)

h_p Height of plate

n Total number of bolts

n_1 The number of horizontal bolt rows

n_2 The number of vertical bolt rows

p Bolt spacing (column bases)

p_1 Vertical bolt spacing for end plates, fin plates and column splices ('pitch')

p_2 Transverse distance between bolt centrelines in a fin plate connection

p_3 Transverse distance between bolt centrelines (gauge)

r Root radius of section

s Leg length of fillet weld

t_f Thickness of flange

t_w Thickness of web (Subscript 1 refers to supported beam and subscript 2 refers to supporting element)

t_p Thickness of plate

V Shear (Subscript Ed or Rd refer to design effect (force) or design resistance respectively)

W Section modulus (Subscript el or pl refers to elastic or plastic modulus)

γ_{M0} is the partial factor for the resistance of cross section (γ_{M0} = 1.0 as given in the UK NA to BS EN 1993-1-1)

γ_{M1} is the partial factor for the resistance of members to instability assessed by member checks (γ_{M1} = 1.0 as given in the UK NA to BS EN 1993-1-1)

γ_{M2} is the partial factor for the cross-sections to fracture (used with f_u) (γ_{M2} = 1.1 as given in the UK NA to BS EN 1993-1-1) AND the partial factor for resistance of bolts, welds, (γ_{M2} = 1.25 as given in the UK NA to BS EN 1993-1-8)

$\gamma_{M,u}$ is the partial factor for the resistance of components when verifying structural integrity. No partial factor is given in BS EN 1993-1-8. In this publication γ_{Mu} has been used. A value of γ_{Mu} = 1.1 is recommended.

γ_{M3} is the partial factor for slip resistance at ULS (γ_{M3} = 1.25 as given in the National Annex).

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 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.

Use of standard connections, where the fittings, bolts, welds and geometry are fully defined, offers the following benefits:

- 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 standard components for fittings which:
  - improves availability
  - leads to a reduction in material costs
  - reduces buying, storage, and handling time.
- The use of one property class and diameter of fully threaded bolt in a limited range of lengths which:
  - saves time changing drills or punches in the workshop
  - leads to faster erection and fewer errors on site
  - leads to economy of bulk purchase.
- The use of small, single pass fillet welds which:
  - avoids the need for any edge preparation
  - reduces the amount of 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 publication is shown in Table 2.1.

Table 2.1 Recommended components

Component	Preferred Option	Notes
Fittings	Material of grade S275	– (see Table 2.2)
Bolts	M20 8.8 Bolts, fully threaded	– Some heavily loaded connections may need larger diameter bolts – Foundation bolts may be M20, M24, M30, 8.8 or 4.6
Holes	Generally 22 mm diameter, punched or drilled	– 26 mm diameter for M24 bolts – 6 mm oversize for foundation bolts
Welds	Fillet welds generally 6 mm or 8 mm 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, fittings may be manufactured from standard flats or cut by machine from larger plates. Table 2.2 lists the recommended range of fitting sizes which have been adopted throughout this publication.

It is recommended that all fitting material should be S275. Other material grades should not be used without demonstrating that the joint classification remains nominally pinned.

Table 2.2 Recommended sizes of end plates and fin plates

Fittings		Location	
Size mm	Thickness mm	End Plate	Fin Plate
100	10		●
120	10		●
150	10	●	●
160	10		●
180	10	●	●
200	12	●	

Bolts

Structural bolting practice for both rolled section members and hollow section members is based predominantly on property class 4.6 and 8.8 non-preloaded bolts used in 2 mm clearance holes. Such bolts are specified in BS EN ISO 4014, 4016, 4017 and 4018[11].

The recommended option of M20 8.8 fully threaded bolts is readily available. Property class 4.6 bolts are generally used only for fixing lighter components such as purlins or sheeting rails, when 12 mm or 16 mm bolts may be adopted. For holding down bolts, see Section 7.2.

There may be situations, for example a column splice subjected to large load reversals in a braced bay, where the designer considers that joint slip is unacceptable. In these cases property Class 8.8 preloaded bolts to BS EN 14399-3[12] should be used.

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

The determination of bolt shear resistance should assume that the threads are in the shear plane. All the resistance tables and examples in this publication follow this assumption.

Fully threaded bolts

Common practice is to specify fully threaded bolts, meaning one bolt size can be universally used for a large number of connections. The use of M20, 8.8 fully threaded bolts 60 mm long is recommended.

Research[13] 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. If additional deformation might be of concern, the use of preloaded bolts is recommended.

Bolting to square and rectangular hollow sections

Bolted connections to square and rectangular hollow sections using the Flowdrill, Blind Bolt or Hollo-Bolt systems may also be standardised but the layout of the fasteners may need revision to accommodate larger holes (Hollo-Bolt) and the proximity of the side wall of the hollow section.

For further information on the Flowdrill system, see Appendix D and Tables G.58, G.59 and G.68.

For further information on the Hollo-Bolt system, see Appendix E and Tables G.60, G.61 and G69.

For further information on Blind Bolts, see Appendix F and Tables G.62 and G63.

Holes

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

- Bolt diameter (d) ≤ 24 mm: d + 2 mm
- Bolt diameter (d) > 24 mm: d + 3 mm
- Holding down bolts: d + 6 mm

With slab bases thicker than 60 mm, the normal clearance of 6 mm may need to be increased.

Holes of 22 mm or 26 mm can be safely punched through grade S275 material up to 12 mm thick. Further guidance on hole punching can be found in the *National Structural Steelwork Specification*[10].

Welds

All the welds used for connections in this publication are simple fillet welds carried out by a metal arc process in accordance with BS EN 1011-1[14].

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 (Figure 2.1).

Notches are generally fabricated with a radius at the notch.

The notch length should provide a nominal clearance of 10 mm 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.

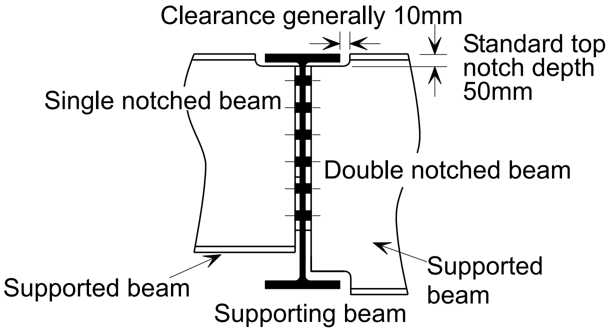


Figure 2.1 Beam notches

A standard top notch depth of 50 mm is recommended and is adequate for all but the largest UKB and UKC sections. Specific notch dimensions for a particular rolled section can be found in the dimension tables in the yellow pages.

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 can often be avoided by the careful selection of beam and column sizes.

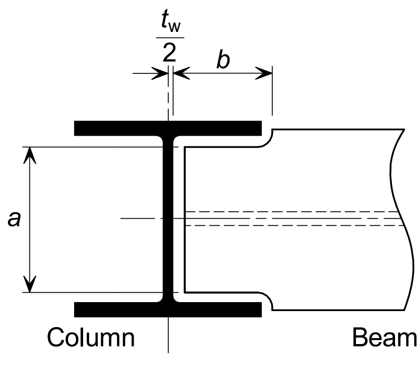


Figure 2.2 Flange trims

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

Flange chamfers

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 20 mm bolts at 90 mm or greater cross centres.

Vertical bolt layout

The recommended vertical bolt layout using M20 bolts adopted throughout this publication is shown in Figure 2.4.

In the standard details, the tops of the partial depth end plates and fin plates are located 50 mm below the top of the beam, which for beams with a standard notch, positions them flush with the top of the notch. This leads to the first bolt row being set down a constant 90 mm from the top of the beam, which is generally the setting out point.

For the partial depth end plates and fin plates, a top and bottom edge distance to the bolts of 40 mm has been used. This complies with the 2*d* minimum specified for fin plates in design procedure Check 1 (Section 5) and will prevent premature bearing failure with M20 bolts.

The standard pitch of 70 mm which is recommended in this publication 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.

Bolt gauge

For UKB and UKC sections, the bolt gauge (cross centres) has been set at 90 mm or 140 mm for end plates. These dimensions are designed to provide reasonable clearance for bolt access as well as giving sufficient width for flexibility in end plate connections.

The bolt gauge may have to depart from these standard dimensions when connecting to the face of a hollow section because the gauge should be at least 0.3 × the hollow section face width.

Hollo-bolts

The recommended layout of Hollo-Bolts is given in Figure 4.5.

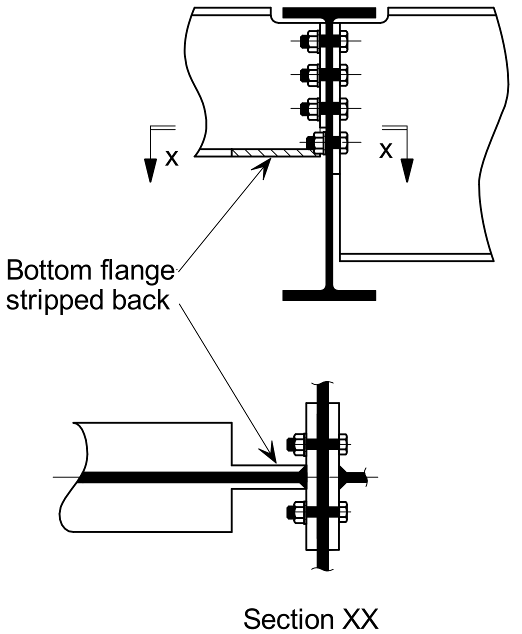


Figure 2.3 Flange chamfers

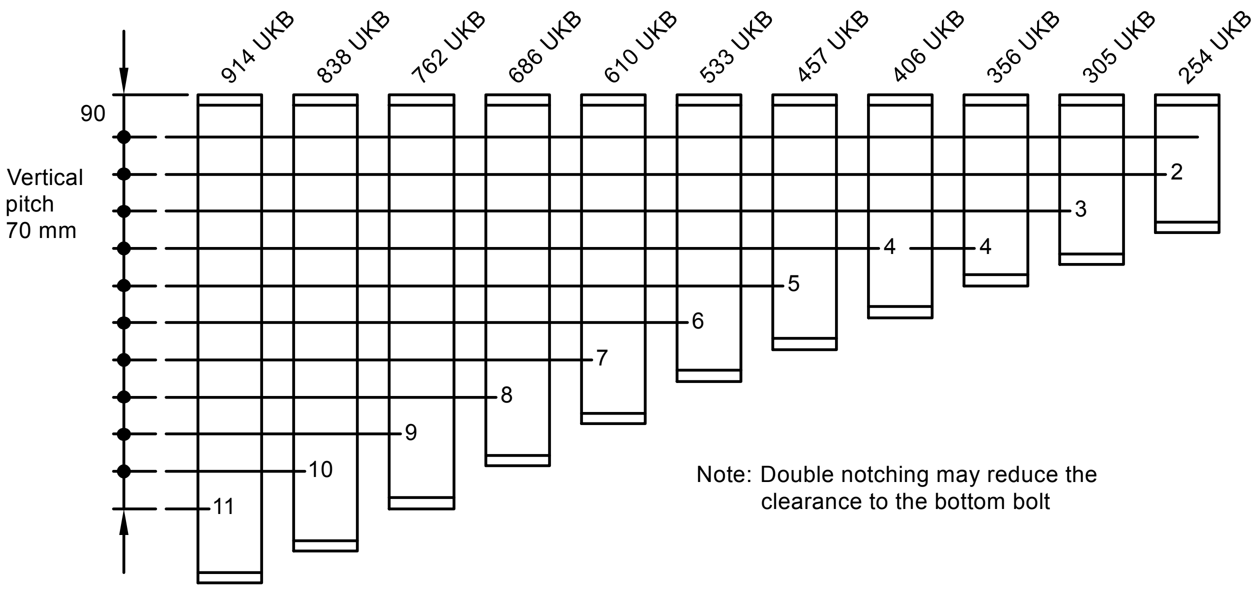


Figure 2.4 Vertical bolt layout (beam end connections using M20 bolts)

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 can be a laborious process and so a full set of resistance tables has been included in the yellow pages of this publication.

Verifying the strength of a nominally pinned joint involves three stages:

(1) Ensuring that the joint is detailed such that it develops only nominal moments which do not adversely affect the members or the joint itself. The joint should be detailed so that it behaves in a ductile manner.

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

(3) Checking the resistance of each component.

For normal design there are ten design procedure checks for all the parts of a beam to beam or beam to column joint for vertical shear.

A further six checks are necessary to verify the tying resistance of the joint. Beam to column connections must be able to resist lateral tying forces unless these forces are resisted by other means within the structure, such as the floor slabs.

Table 3.1 summarises the design procedure checks required for Partial Depth End Plates (PDEP), Full Depth End Plates (FDEP) and Fin Plates (FP). The design procedures are described in Sections 4.5, **Error! Reference source not found.** and 5.5.

Table 3.1 Design procedure for beam connections - summary table

Design procedure checks		PDEP	FDEP	FP
1	Recommended detailing practice			
2	Supported beam	Welds		Bolt Group
3	Supported beam	N/A	N/A	Fin plate
4	Supported beam		Web in shear	
5	Supported beam	Resistance at a notch	N/A	Resistance at a notch
6	Supported beam	Local stability of notched beam	N/A	Local stability of notched beam
7	Unrestrained supported beam	Overall stability of notched beam	N/A	Overall stability of notched beam
8	Connection	Bolt group		Welds
9	Connection	End plate in shear	N/A	N/A
10	Supporting beam/column		Shear and bearing	
11	Tying resistance		Plate and bolts	
12	Tying resistance		Supported beam web	
13	Tying resistance		Welds	
14	Tying resistance		Supporting column web (UKC or UKB)	
15	Tying resistance		Supporting column wall (RHS or SHS)	
16	Tying resistance	N/A	N/A	Supporting column wall (CHS)

Note:

(1) Checks on the bending, shear, local and lateral buckling resistance 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 END PLATES

4.1 INTRODUCTION

Typical end plate connections are shown in Figure 4.1. The end plate, which may be partial depth or full depth, is welded to the supported beam in the workshop. The beam is then bolted to the supporting beam or column on site. Flowdrill or Hollo-Bolts are used for connections to hollow section columns.

End plates are probably the most popular of the simple beam connections currently in use in the UK. They can be used with skewed beams and can tolerate moderate offsets in beam to column joints.

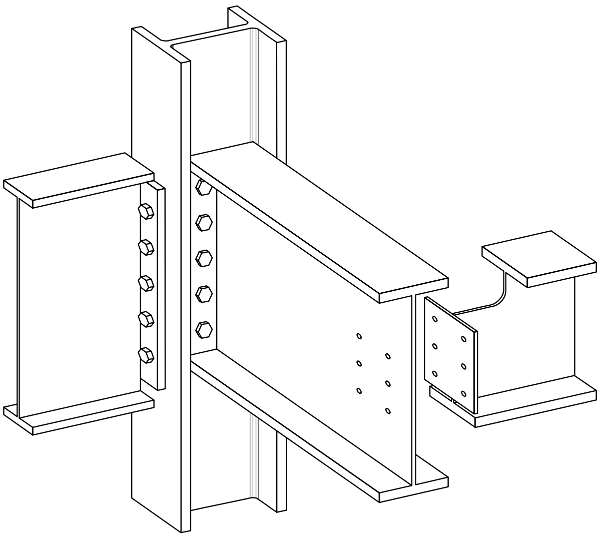


Figure 4.1 End plate beam to column and beam to beam connections

4.2 PRACTICAL CONSIDERATIONS

Partial depth end plates

Normal practice is for partial depth end plates to be welded to the beam web only, usually with modest size fillet welds. In this publication, the welds are sized to be full strength based on the web thickness, as explained in Appendix C. The weld should not be continued across the top and bottom of the plate.

It is quite common, particularly with thinner plates, to experience bowing of the plate due to weld shrinkage. 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.

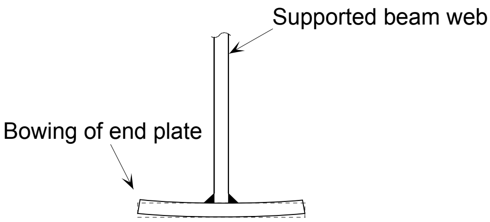


Figure 4.2 Distortion after welding

Two sizes of end plate have been recommended for use in this publication. A 150 × 10 mm flat with bolts at 90 mm cross centres will generally be adequate for beams up to 457 mm deep. For deeper beams, a 200 × 12 mm flat is recommended with a 140 mm 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 UKC or 203 UKC column web will have to be fitted with a narrower end plate.

According to BS EN 1991-1-7[7], all significant buildings must resist a minimum tying force as given in Annex A of that Standard. Although Class 1 structures have no minimum tying force, other guidance[15] recommends a minimum tying force of 75 kN even for Class 1 buildings. For other classes of building, the tying force can be considerable.

The tying resistance of partial depth end plates may be less than required. A thicker end plate will generally increase the tying resistance, although nominally pinned behaviour must be maintained. The thicknesses recommended in this publication, compared with thicknesses recommended in P212[16] are shown in Table 4.1. The new recommendations are 2 mm thicker than the previous details. The increase in thickness does not change the fundamental joint behaviour or affect the joint classification as nominally pinned.

Table 4.1 Recommended end plate size

End plate connection	Previous advice from P212[16]	Current advice
Ordinary, Blind Bolt or Flowdrill bolts Beams ≤ 457×191UKB	150 × 8	150 × 10
Ordinary, Blind Bolt or Flowdrill bolts Beams > 457×191UKB	200 × 10	200 × 12
Hollo-Bolts Beams ≤ 457×191UKB	180 × 8	180 × 10
Hollo-Bolts Beams > 457×191UKB	200 × 10	200 × 12

If the tying resistance achieved with partial depth end plates is not sufficient to comply with the structural integrity requirements, the designer is advised to use a full depth end plate connection (see next paragraph and the design procedures in Section **Error! Reference source not found.**). Simply specifying a thicker partial depth end plate could result in a stiff joint with insufficient rotation capacity to be classified as "nominally pinned". In S275 steel, end plates thicker than 12 mm do not satisfy Eurocode requirements for rotation capacity and cannot be classified as nominally pinned (see Section 1.2).

Full depth end plates

End plates which extend the full depth of the beam, and are welded to the beam flanges may also be used, as shown in Figure 4.3.

In general, full depth end plates welded all around the supported beam will provide an increased resistance to tying and (where vertical resistance is governed by web shear) an enhanced vertical resistance compared to partial depth end plates.

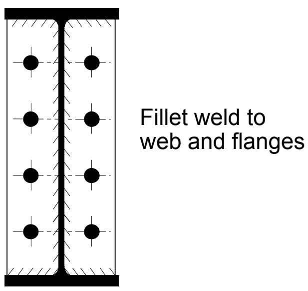


Figure 4.3 Full depth end plate

If the recommended detailing rules given in Check 1 are followed, then a full depth end plate has sufficient flexibility to be classified as a nominally pinned joint. If a stiffer arrangement is provided, the joint may become semi-rigid, 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.

However, 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[17] and at The University of Sheffield[18]. This second study highlights the situations where these moments may safely be ignored.

Using the design rules given in Section 4.5, the connection moment (which is indeterminate but small) can be neglected.

Connections with thicker end plates are discussed in another publication in this series, Joints in Steel Construction: Moment-resisting joints to Eurocode 3[19].

Erection

Connections with end plates have 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 fifth beam of a continuous run. The number of packs should be kept to a minimum (less than 4) and allowance made for the reduction in shear resistance of the bolts (clause 3.6.1(12) of BS EN 1993-1-8).

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 4.4.

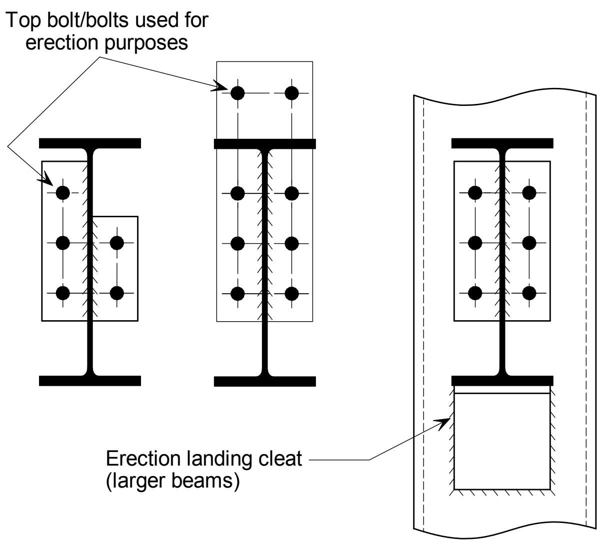


Figure 4.4 Erection aids

4.3 RECOMMENDED GEOMETRY

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

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

(1) in partial depth end plates, the end plate should be positioned close to the top flange in order to provide positional restraint to that flange

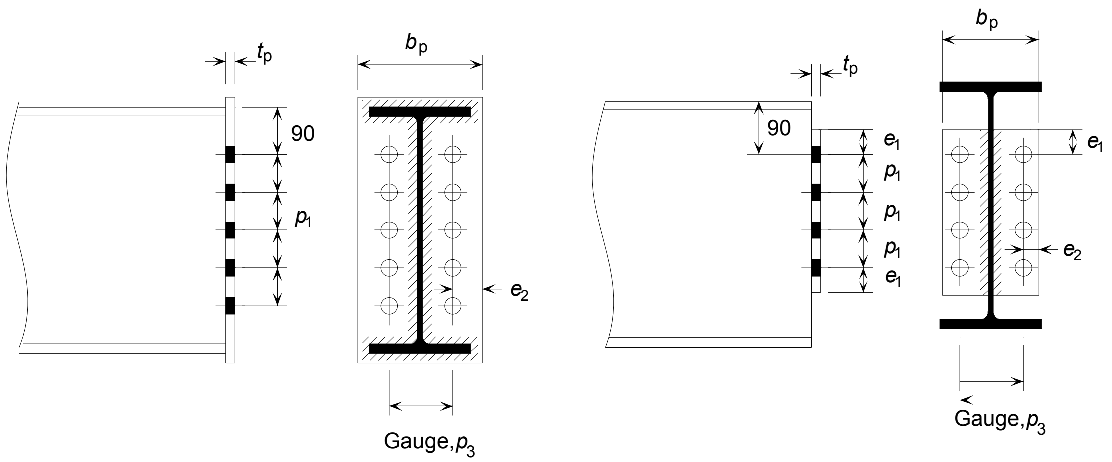
(2) in partial depth end plates, the end plate depth should be at least 0.6 × the supported beam depth in order to provide the beam with adequate torsional restraint

(3) the end plate should be relatively thin (10 mm or 12 mm)

(4) the bolts should be at reasonable gauge (cross centres) of 90 mm or 140 mm.

The first two requirements ensure that in cases where the beam is laterally unrestrained, it can be designed with a buckling length of 1.0 *L*. The last two requirements ensure adequate flexibility and ductility to classify as "nominally pinned joints".

These requirements, together with the standard geometry presented in Section 2, have been used to create the 'standard connection' shown in Figure 4.5.



Ordinary and Flowdrill bolts

Supported beam	Recommended end plate size $b_p \times t_p$	Bolt gauge p_3
Up to 533 UKB	150 × 10	90
533 UKB and above	200 × 12	140

Bolts: M20 in 22 mm diameter holes
End plate: S275, minimum length $0.6h_{b1}$ where h_{b1} is the depth of supported beam
Vertical pitch p_1 = 70 mm
End distance e_1 = 40 mm
Edge distance e_2 = 30 mm

Hollo-Bolts

Supported beam	Recommended end plate size $b_p \times t_p$	Bolt gauge p_3
Up to 533 UKB	180 × 10	90
533 UKB and above	200 × 12	110

End plate: S275, minimum length $0.6h_{b1}$ where h_{b1} is the depth of supported beam
Vertical pitch p_1 = 80 mm
End distance e_1 = 45 mm
Edge distance e_2 = 45 mm

Figure 4.5 Standard end plate connections

4.4 DESIGN

The full design procedure is presented in Section 4.5.

The end plate has a vertical shear resistance of around 50% to 70% of that 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 4.5 and Table G.2 is adopted, it will be found that the connection shear resistance is generally governed:

- For ordinary/Flowdrill bolts: either by shear in the beam web, (Check 4) or shear in the bolts (Check 8)
- 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 4.6. For partial depth end plates there are basically two stages:

(1) the unhindered rotation of the connection until,

(2) the lower beam flange bears against the support.

Structural integrity

As noted in Section 1.2, all buildings must satisfy certain structural integrity requirements. For some classes of buildings it will be necessary to check the connection for tying forces determined from BS EN 1991-1-7. The verification of the tying resistance and vertical resistance are completed as independent checks, not in combination. The calculation of tying resistance generally involves ultimate strengths, and it anticipates that irreversible deformation of the connection components will take place. BS EN 1993-1-8 does not provide a factor for the resistance of elements under accidental actions. γ_{Mu} is adopted and taken as 1.1 in this publication.

For connections to UKC flanges, the critical mode of failure of partial depth end plates will be design Check 11, the tension resistance of the end plate. Use of a full depth end plate will generally increase the tying resistance.

Although full depth end plate connections will be stiffer than those with partial depth end plates, the standard connections in this publication have been shown to be classified as nominally pinned in accordance with BS EN 1993-1-8.

Worked examples

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

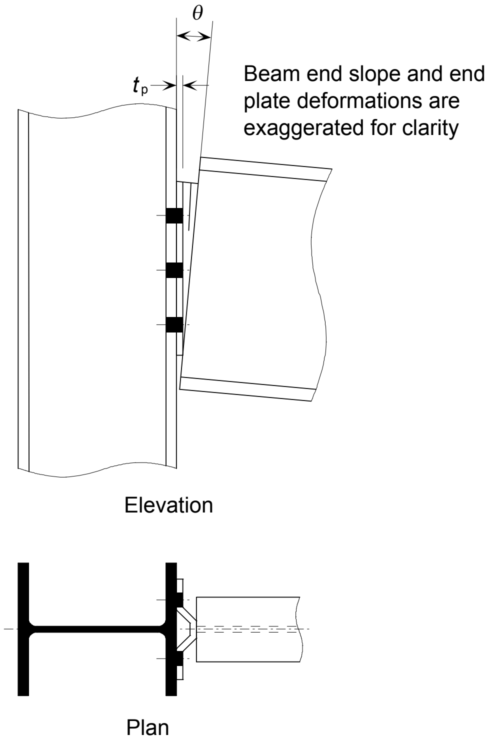


Figure 4.6 Behaviour of partial depth end plate connections

Resistance tables

Resistance tables for standardised partial depth and full depth end plates for S275 and S355 beams are provided as shown in Table 4.2.

Table 4.2 Resistance tables for end plates

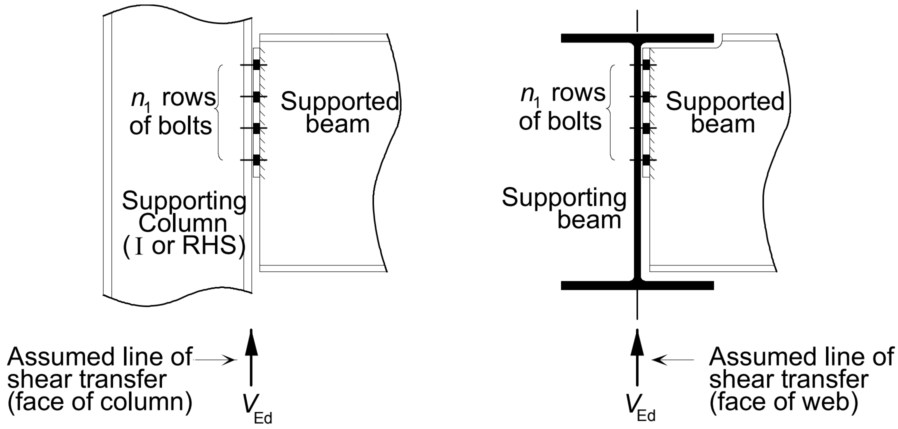
	Partial depth		Full depth	
	Ordinary & Flowdrill	**Hollo-Bolt**	**Ordinary & Flowdrill**	**Hollo-Bolt**
S275	G.4	G.6	G.11	G.13
S355	G.5	G.7	G.12	G.14

Values of the connection shear and tying resistances are tabulated together with simple aids to check the necessary support thickness and the beam notch (if applicable).

4.5 DESIGN PROCEDURES FOR PARTIAL DEPTH END PLATES

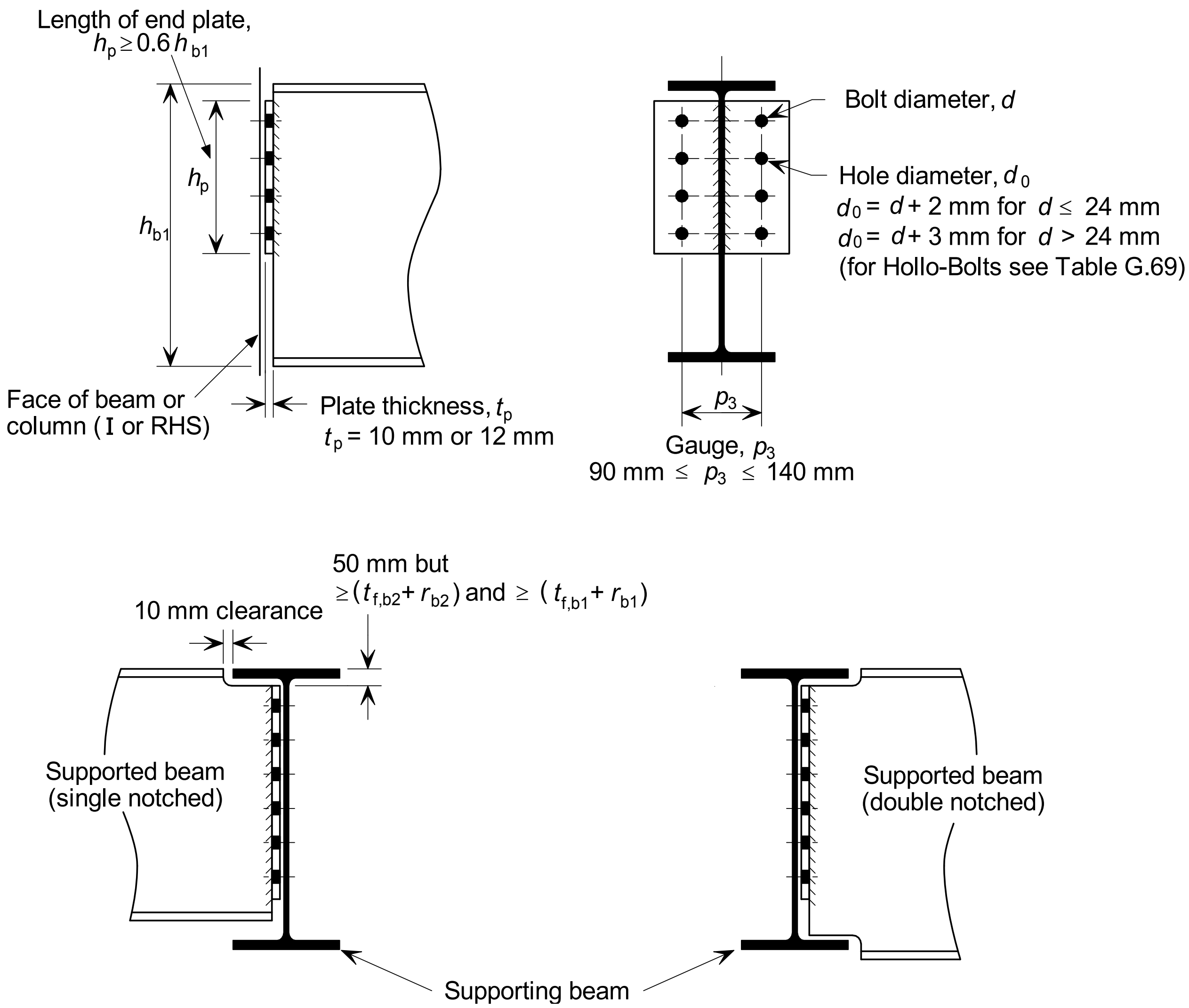
The design procedures used in this publication are in accordance with traditional UK design practice and are based on the simply supported beam end reaction.

The design procedures apply to beams connected to a column flange, a column web, a supporting beam web or to a rolled hollow section column.



Check 1	Recommended detailing practice		
Check 2	Supported beam	–	Welds
Check 3	*Not applicable*		
Check 4	Supported beam	–	Web in shear
Check 5	Supported beam	–	Resistance at a notch
Check 6	Supported beam	–	Local stability of notched beam
Check 7	Unrestrained supported beam	–	Overall stability of notched beam
Check 8	Connection	–	Bolt group
Check 9	Connection	–	End plate in shear
Check 10	Supporting beam/column	–	Shear and bearing
Check 11	Tying resistance	–	Plate and bolts
Check 12	Tying resistance	–	Supported beam web
Check 13	Tying resistance	–	Welds
Check 14	Tying resistance	–	Supporting column web (UKC or UKB)
Check 15	Tying resistance	–	Supporting column wall (Hollow section)
Check 16	*Not applicable*		

CHECK 1	Recommended detailing practice



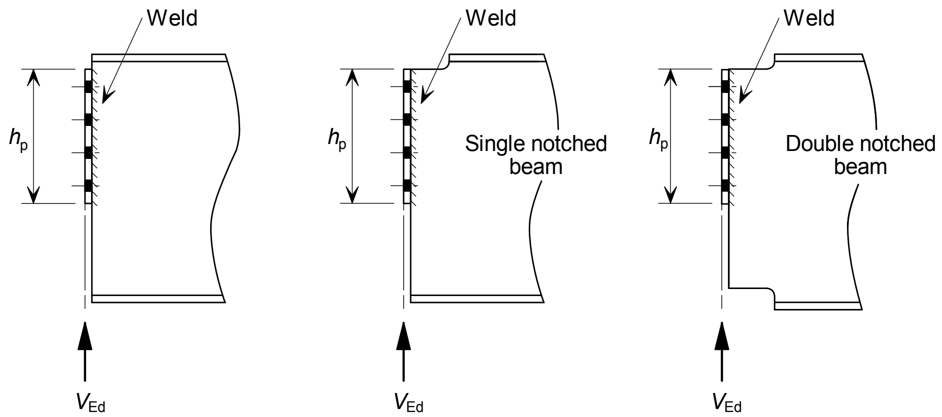
Subscript 1 refers to the supported beam

Subscript 2 refers to the supporting member

Note:

The end plate is generally positioned close to the top flange of the beam (50 mm is recommended), to provide adequate positional restraint. Plate length of at least $0.6h_{b1}$ is adopted to give adequate torsional restraint.

CHECK 2	Supported beam – Welds



Resistance of fillet welds connecting end plate to beam web:

Basic requirement:

$a \geq 0.40t_{w,b1}$ for S275 supported beam

$a \geq 0.48t_{w,b1}$ for S355 supported beam

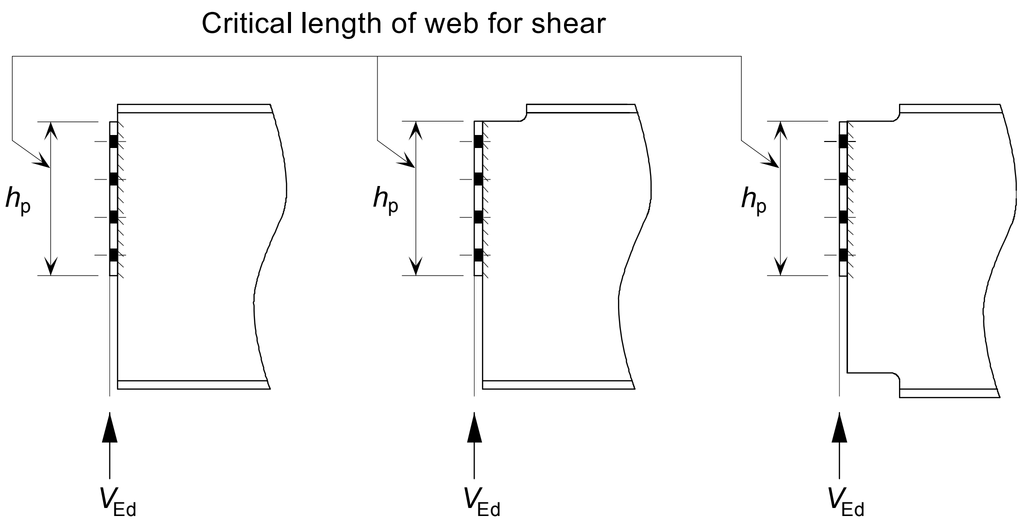
where:

a is the effective weld throat thickness $= 0.7s$ (normally)

s is the weld leg length

See Appendix C for more details about the weld requirements.

CHECK 4	Supported beam – Web in shear



Shear resistance of the beam web at the end plate

Basic requirement:

$V_{Ed} \leq V_{c,Rd}$

$V_{c,Rd}$ is the design shear resistance of the supported beam connected to the end plate

$$= V_{pl,Rd} = A_v \frac{f_{y,b1}/\sqrt{3}}{\gamma_{M0}}$$

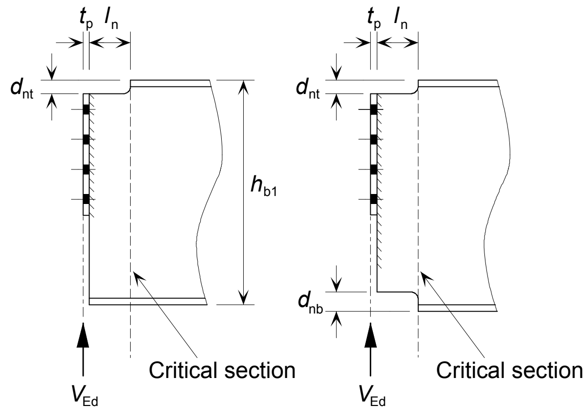
where:

A_v is the shear area $= 0.9 h_p t_{w,b1}$ *

$t_{w,b1}$ is the thickness of the supported beam web

* The factor of 0.9 is applied to rectangular areas as recommended in Access-Steel document SN014[20].

CHECK 5	Supported beam – Resistance at a notch



Shear and bending interaction at the notch:

Basic requirement:

$V_{Ed} \times (t_p + l_n) \leq M_{v,N,Rd}$ or $M_{v,DN,Rd}$

$M_{v,N,Rd}$ is the moment resistance of a single notched supported beam at the notch in the presence of shear

$M_{v,DN,Rd}$ is the moment resistance of a double notched supported beam at the notch in the presence of shear

For single notched beam:

For low shear (i.e. $V_{Ed} \leq 0.5V_{pl,N,Rd}$)

$$M_{v,N,Rd} = \frac{f_{y,b1} W_{el,N,y}}{\gamma_{M0}}$$

For high shear (i.e. $V_{Ed} > 0.5V_{pl,N,Rd}$)

$$M_{v,N,Rd} = \frac{f_{y,b1} W_{el,N,y}}{\gamma_{M0}} \left(1 - \left(\frac{2V_{Ed}}{V_{pl,N,Rd}} - 1 \right)^2 \right)$$

For double notched beam:

For low shear (i.e. $V_{Ed} \leq 0.5V_{pl,DN,Rd}$)

$$M_{v,DN,Rd} = \frac{f_{y,b1} t_{w,b1}}{6\gamma_{M0}} (h_{b1} - d_{nt} - d_{nb})^2$$

For high shear (i.e. $V_{Ed} > 0.5V_{pl,DN,Rd}$)

$$M_{v,DN,Rd} = \frac{f_{y,b1} t_{w,b1}}{6\gamma_{M0}} (h_{b1} - d_{nt} - d_{nb})^2 \left(1 - \left(\frac{2V_{Ed}}{V_{pl,DN,Rd}} - 1 \right)^2 \right)$$

Where:

t_p is the end plate thickness

$V_{pl,N,Rd}$ is the shear resistance at the notch for single notched beams

$$= \frac{A_{v,N} f_{y,b1}}{\sqrt{3}\,\gamma_{M0}}$$

$$A_{v,N} = A_{Tee} - b\,t_{f,b1} + (t_{w,b1} + 2r_{b1}) \frac{t_{f,b1}}{2}$$

$V_{pl,DN,Rd}$ is the shear resistance at the notch for double notched beams

$$= \frac{A_{v,DN} f_{y,b1}}{\sqrt{3}\,\gamma_{M0}}$$

$$A_{v,DN} = 0.9\,(h_{b1} - d_{nt} - d_{nb})\,t_{w,b1}$$

$t_{f,b1}$ is the flange thickness of the supported beam

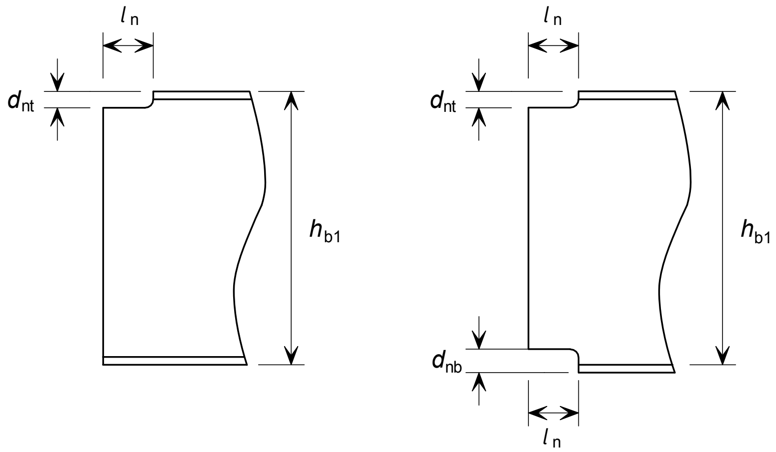
$t_{w,b1}$ is the web thickness of the supported beam

h_{b1} is the height of the supported beam

A_{Tee} is the area of the Tee section

$W_{el,N,y}$ is the elastic modulus of the Tee section at the notch

CHECK 6	Supported beam – Local stability of notched beam



No account need be taken of the local stability of the notched beam provided that the following conditions are met:

For one flange notched [21],[22]. Basic requirement:

$d_{nt} \leq h_{b1} / 2$ and:

$l_n \leq h_{b1}$	for	$h_{b1} / t_{w,b1} \leq$	54.3	(S275 steel)
$l_n \leq \dfrac{160000\, h_{b1}}{\left(h_{b1} / t_{w,b1}\right)^3}$	for	$h_{b1} / t_{w,b1} >$	54.3	(S275 steel)
$l_n \leq h_{b1}$	for	$h_{b1} / t_{w,b1} \leq$	48.0	(S355 steel)
$l_n \leq \dfrac{110000\, h_{b1}}{\left(h_{b1} / t_{w,b1}\right)^3}$	for	$h_{b1} / t_{w,b1} >$	48.0	(S355 steel)

For both flanges notched [22]. Basic requirement:

$max\,(d_{nt}, d_{nb}) \leq h_{b1} / 5$ and:

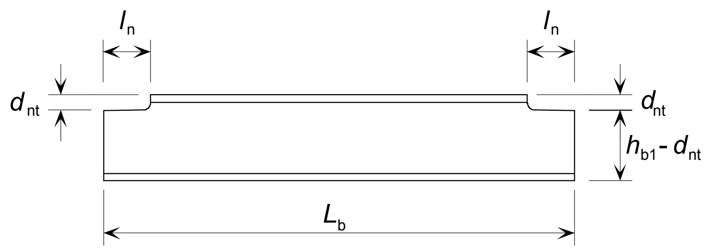
$l_n \leq h_{b1}$	for	$h_{b1} / t_{w,b1} \leq$	54.3	(S275 steel)
$l_n \leq \dfrac{160000\, h_{b1}}{\left(h_{b1} / t_{w,b1}\right)^3}$	for	$h_{b1} / t_{w,b1} >$	54.3	(S275 steel)
$l_n \leq h_{b1}$	for	$h_{b1} / t_{w,b1} \leq$	48.0	(S355 steel)
$l_n \leq \dfrac{110000\, h_{b1}}{\left(h_{b1} / t_{w,b1}\right)^3}$	for	$h_{b1} / t_{w,b1} >$	48.0	(S355 steel)

$t_{w,b1}$ is the web thickness of the supported beam

Where the notch length l_n exceeds these limits, either suitable stiffening should be provided or the notch should be checked to references 21, 22 and 26.

If the beam is unrestrained, overall stability should be checked in accordance with Check 7.

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 30.

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

(3) Beams should be checked for lateral torsional buckling to BS EN 1993-1-1, Clause 6.3.2, using an effective length in the calculation of M_{cr}, the elastic critical moment for lateral torsional buckling.

(4) The solution below gives an effective length (L_E) based on references 30, 31 and 32. It is only valid for l_n / L_b < 0.15 and d_{nt} / h_{b1} < 0.2. Beams with notches outside these limits should be checked as Tee sections, or stiffened.

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

$$K = K_0 / \lambda_b$$

$$\lambda_b = \frac{UVL_b}{i_z}$$

Where:

X, U, V and i_z are for the un-notched I beam section and are defined in P-363[23]

Conservatively:

U = 0.9 and

V = 1.0

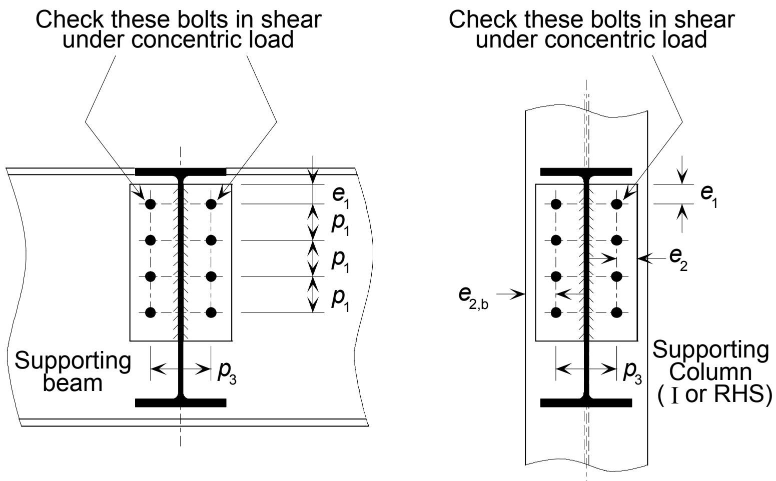
For $\lambda_b < 30$ $K_0 = 1.1\, g_0 X$ but $\leq 1.1K_{max}$

For $\lambda_b \geq 30$ $K_0 = g_0 X$ but $\leq K_{max}$

g_0 and K_{max} are as follows:

$\frac{l_n}{L_b}$	g_0	K_{max}	
		UKB section	**UKC 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	**Connection – Bolt group**



Shear and bearing resistance of the bolt group connecting the end plate to the supporting beam or column.

Basic requirement:

$V_{Ed} \leq F_{Rd}$

F_{Rd} is the resistance of the bolt group

If $F_{b,Rd} \leq 0.8F_{v,Rd}^*$ then $F_{Rd} = nF_{b,Rd}$

If $F_{b,Rd} > 0.8F_{v,Rd}^*$ then $F_{Rd} = 0.8n(F_{v,Rd})^*$

* The reduction factor 0.8 allows for the presence of tension in the bolts [24].

Shear resistance

$F_{v,Rd,}$ is the shear resistance of one bolt

$$= \frac{\alpha_v f_{ub} A}{\gamma_{M2}}$$

Bearing resistance

(See next page)

where:

α_v = 0.6 for 8.8 bolts

A is the tensile stress area of the bolt, A_s

n is the total number of bolts

γ_{M2} is the partial factor for resistance of bolts (γ_{M2} = 1.25 as given in the National Annex)

CHECK 8 (continued)	Connection – Bolt group

Bearing resistance:

$F_{b,Rd}$ is the minimum of the bearing resistance on the end plate and the bearing resistance on the supporting member for an individual bolt

$= \min\left(F_{b,Rd,p}\,;\quad F_{b,Rd,2}\right)$

Bearing on the end plate:

$$F_{b,Rd,p} = \frac{k_{1,p}\,\alpha_{b,p}\,f_{u,p}\,d\,t_p}{\gamma_{M2}}$$

Bearing on the supporting member:

$$F_{b,Rd,2} = \frac{k_{1,2}\,\alpha_{b,2}\,f_{u,2}\,d\,t_2}{\gamma_{M2}}$$

Subscript 2 refers to the supporting member

Subscript p refers to the end plate

where:

γ_{M2} = 1.25 (see shear resistance)

$f_{u,p}$ is ultimate tensile strength of plate

d is the diameter of the bolt

d_0 is the diameter of the holes

t_p is the thickness of the plate

t_2 is the thickness of the supporting element

For the end plate:

$$\alpha_{b,p} = \min\left(\frac{e_1}{3d_0};\quad \frac{p_1}{3d_0} - \frac{1}{4};\quad \frac{f_{ub}}{f_{u,p}};\quad 1.0\right)$$

For the supporting member:

$$\alpha_{b,2} = \min\left(\frac{p_1}{3d_0} - \frac{1}{4};\quad \frac{f_{ub}}{f_{u,2}};\quad 1.0\right)$$

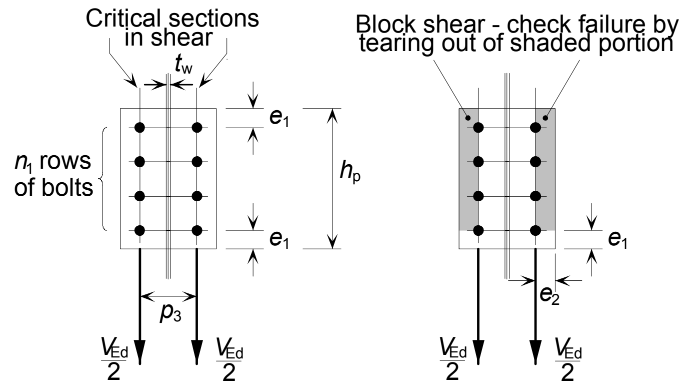
For the end plate:

$$k_{1,p} = \min\left(2.8\frac{e_2}{d_0} - 1.7;\ 1.4\frac{p_3}{d_0} - 1.7;\ 2.5\right)$$

For the supporting member:

$$k_{1,2} = \min\left(2.8\frac{e_{2,b}}{d_0} - 1.7;\ 1.4\frac{p_3}{d_0} - 1.7;\ 2.5\right)$$

CHECK 9	Connection – End plate in shear



Shear resistance of the end plate connected to the supporting beam or column

For shear:

Basic requirement:

$V_{Ed} \leq V_{Rd,min}$

$V_{Rd,min}$ is the shear resistance of the end plate = smaller of the gross section shear resistance $V_{Rd,g}$, net section shear resistance, $V_{Rd,n}$ and block tearing resistance, $V_{Rd,b}$

End plate in shear: gross section

$$V_{Rd,g} = \frac{2h_p t_p}{1.27} \frac{f_{y,p}}{\sqrt{3}\gamma_{M0}}$$

The coefficient 1.27 takes into account the reduction in shear resistance due to the presence of bending[25].

End plate in shear: net section

$$V_{Rd,n} = 2A_{v,net} \frac{f_{u,p}}{\sqrt{3}\gamma_{M2}}$$

End plate in shear: block tearing

$$V_{Rd,b} = 2\left(\frac{f_{u,p}A_{nt}}{\gamma_{M2}} + \frac{f_{y,p}A_{nv}}{\sqrt{3}\gamma_{M0}}\right)$$

But if $h_p < 1.36p_3$ and $n_1 > 1$ then:

$$V_{Rd,b} = 2\left(\frac{0.5f_{u,p}A_{nt}}{\gamma_{M2}} + \frac{f_{y,p}A_{nv}}{\sqrt{3}\gamma_{M0}}\right)$$

Additional requirements: End plate in-plane bending

If $h_p < 1.36p_3$ then in addition, $V_{Ed} \leq V_{Rd,ip}$

where $V_{Rd,ip} = \dfrac{2t_p h_p^2 f_{y,p}}{3(p_3 - t_w)\gamma_{M0}}$

where:

$A_{v,net} = t_p\left(h_p - n_1 d_0\right)$

$A_{nt} = t_p\left(e_2 - \dfrac{d_0}{2}\right)$

$A_{nv} = t_p\left(h_p - e_1 - (n_1 - 0.5)d_0\right)$

d_0 is the diameter of the holes

t_p is the thickness of the end plate

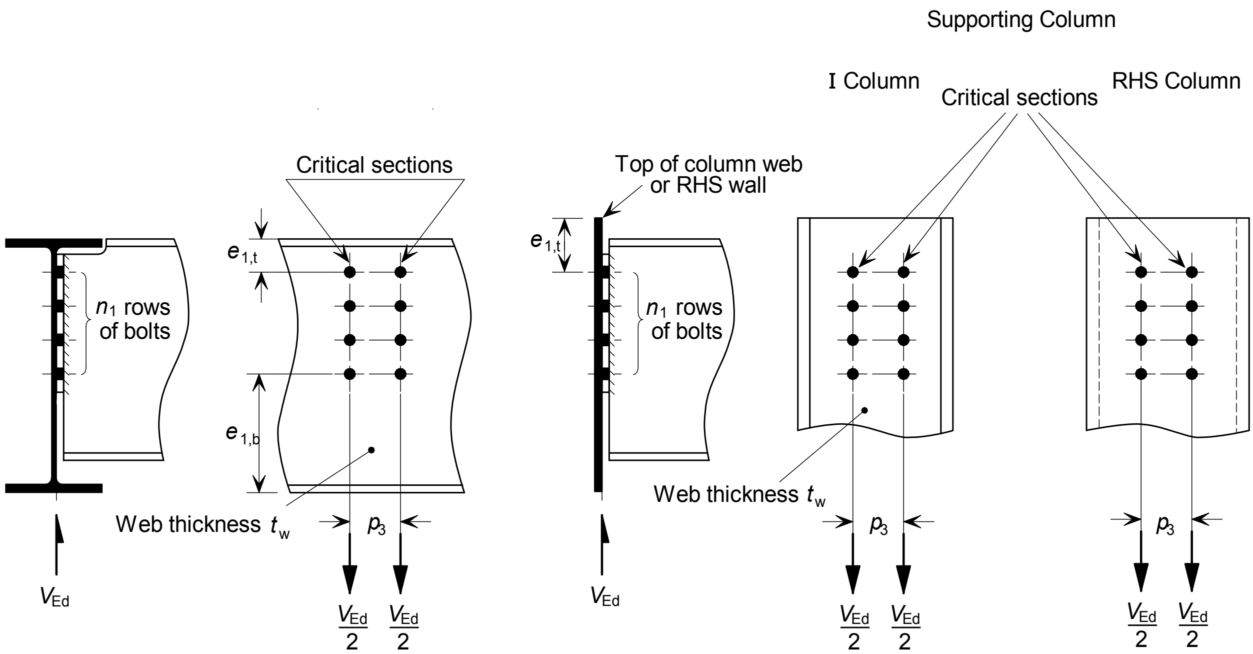
h_p is the height of the end plate

p_3 is the gauge (cross centres)

n_1 is the number of bolt rows

γ_{M2} is the partial factor for the ultimate tension resistance of cross sections (γ_{M2} = 1.1 as given in the National Annex to BS EN 1993-1-1)

CHECK 10	**Supporting beam/column – Shear and Bearing** **(with one supported beam)**



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

Shear:

Basic requirement:

$\frac{V_{Ed}}{2} \leq V_{Rd,min}$

$V_{Rd,min}$ is the local shear resistance of the supporting member

$$= \min\left(\frac{A_v f_{y,2}}{\sqrt{3}\gamma_{M0}}; \frac{A_{v,net} f_{u,2}}{\sqrt{3}\gamma_{M2}}\right)$$

A_v is the shear area of supporting member

$= t_2\ (e_t + (n_1 - 1)p_1 + e_b)$

$A_{v,net}$ is the net shear area of supporting member

$= A_v - n_1 d_0 t_2$

where:

e_t $= \min(e_{1,t};\ 5d)$

e_b $= \min(e_{1,b};\ p_3/2;\ 5d)$ (for supporting beam)

e_b $= \min(p_3/2;\ 5d)$ (for supporting column)

d_0 is the diameter of the holes

d is the diameter of the bolt

n_1 is the number of bolt rows

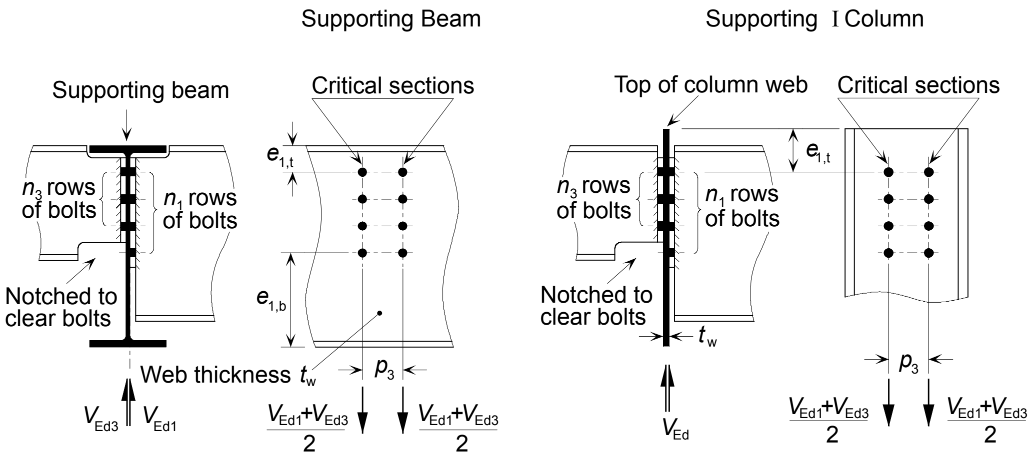
t_2 is the thickness of the supporting member (column web, beam web or column flange)

γ_{M2} is the partial factor for the ultimate tension resistance of cross sections (γ_{M2} = 1.1 as given in the National Annex to BS EN 1993-1-1)

Bearing resistance with one supported beam has been considered in Check 8

.

CHECK 10 (continued)	**Supporting beam/column – Shear and Bearing** (with two supported beams)



Where $n_1 \geq n_3$

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

Shear:

Basic requirement:

$$\frac{V_{Ed,1}}{2} + \frac{V_{Ed,3}}{2} \leq V_{Rd,min}$$

$V_{Rd,min}$ is the local shear resistance of the supporting member

$$= \min\left(\frac{A_v f_{y,2}}{\sqrt{3}\gamma_{M0}};\ \frac{A_{v,net} f_{u,2}}{\sqrt{3}\gamma_{M2}}\right)$$

A_v is the shear area of the supporting member

$= t_2 (e_t + (n_1 - 1)p_1 + e_b)$

$A_{v,net}$ is the net shear area of the supporting member

$= A_v - n_1 d_0 t_2$

Bearing:

Basic requirement:

$$\frac{V_{Ed,1}}{2n_1} + \frac{V_{Ed,3}}{2n_3} \leq F_{b,Rd}$$

$F_{b,Rd}$ is the bearing resistance of a single bolt. See Check 8 for equations to calculate this value.

where:

e_t = min ($e_{1,t}$; $5d$)

e_b = min ($e_{1,b}$; $p_3/2$; p_1 ; $5d$)

t_2 is the thickness of the supporting element (column web or beam web)

d_0 is the diameter of the holes

d is the diameter of the bolt

n_1 is the number of bolt rows in the larger connection side

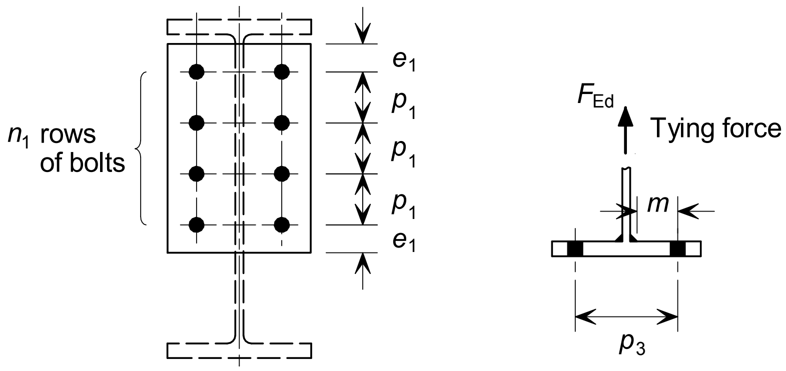
n_3 is the number of bolt rows in the smaller connection side

γ_{M2} is the partial factor for the ultimate tension resistance of cross sections (γ_{M2} = 1.1 as given in the National Annex to BS EN 1993-1-1)

Note:

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

CHECK 11 — Tying resistance – Plate and bolts



Resistance of the end plate

There are three modes of failure for end plates in bending:

Mode 1: complete yielding of the end plate

Mode 2: bolt failure with yielding of the end plate

Mode 3: bolt failure

Basic requirement:

$F_{Ed} \leq \min\left(F_{Rd,u,1}\,; F_{Rd,u,2}\,; F_{Rd,u,3}\right)$

Mode 1 (complete yielding of the end plate)

$$F_{Rd,u,1} = \frac{(8n - 2e_w)M_{pl,1,Rd,u}}{2mn - e_w(m+n)}$$

Mode 2 (bolt failure with yielding of the end plate)

$$F_{Rd,u,2} = \frac{2M_{pl,2,Rd,u} + n\Sigma F_{t,Rd,u}}{(m+n)}$$

Mode 3 (bolt failure)

$$F_{Rd,u,3} = \Sigma F_{t,Rd,u}$$

$F_{t,Rd,u} = \dfrac{k_2 f_{ub} A_s}{\gamma_{M,u}}$ for ordinary bolts.

(For Flowdrill, Hollo-Bolt and Blind Bolt resistances the value of $F_{t,Rd,u}$ should be taken from Tables G.59, G.61 and G.63 respctively)

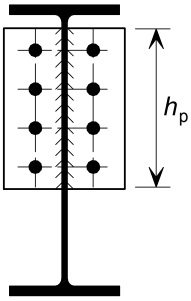
where:

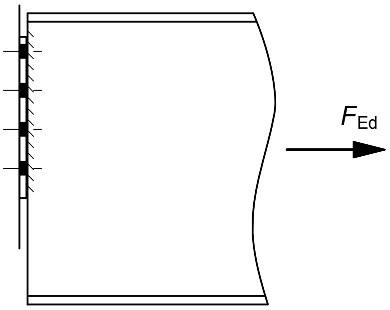
$$M_{pl,1,Rd,u} = \frac{0.25\Sigma l_{eff}\, t_p^2 f_{u,p}}{\gamma_{M,u}}$$

$M_{pl,2,Rd,u} = M_{pl,1,Rd,u}$

Σl_{eff}	is the effective length of the equivalent T-stub
	$= 2e_{1A} + (n_1 - 1)p_{1A}$
e_{1A}	$= e_1$
	but $e_{1A} \leq 0.5\left(p_3 - t_{w,b1} - 2a\sqrt{2}\right) + \dfrac{d_0}{2}$
p_{1A}	$= p_1$ but $\leq p_3 - t_{w,b1} - 2a\sqrt{2} + d_0$
t_p	is the end plate thickness
k_2	= 0.63 for countersunk bolts
	= 0.9 otherwise
A_s	is the tensile stress area of the bolt
m	$= \dfrac{p_3 - t_{w,b1} - 2 \times 0.8 \times a\sqrt{2}}{2}$
$t_{w,b1}$	is the web thickness of the supported beam
a	is the weld throat thickness
n	$= e_{min}$ but $n \leq 1.25m$
e_{min}	$= e_2$
e_w	$= \dfrac{d_w}{4}$
d_w	is the diameter of the washer or the width across points of the bolt or nut as relevant. Washers are not necessarily provided; it is conservative to assume washers are not used.

CHECK 12	Tying resistance – Supported beam web





Resistance of the beam web

Basic requirement:

$F_{Ed} \leq F_{Rd,u}$

$$F_{Rd,u} = \frac{t_{w,b1} h_p f_{u,b1}}{\gamma_{M,u}}$$

where:

$t_{w,b1}$ is the thickness of the supported beam web

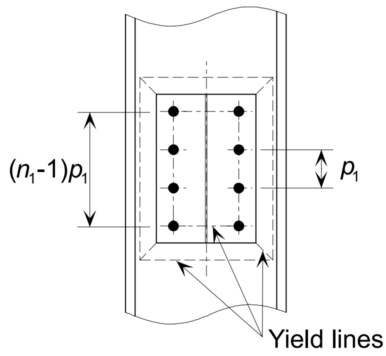
h_p is the height of the end plate

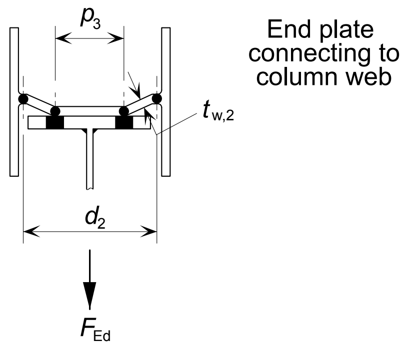
CHECK 13	Tying resistance – Welds

The weld size specified in check 2 will be adequate for tying resistance because the weld is chosen to be full strength.

See Appendix C for more details about the weld requirements.

CHECK 14	Tying resistance – supporting column web (UKC or UKB)





Supporting web resistance

Basic requirement:

$F_{Ed} \leq F_{Rd,u}$

$$F_{Rd,u} = \frac{8M_{pl,Rd,u}}{1-\beta_1}\left[\eta_1 + 1.5(1-\beta_1)^{0.5}(1-\gamma_1)^{0.5}\right]$$

$$M_{pl,Rd,u} = \frac{f_{u,2}\,t_{w,2}^{\;2}}{4\gamma_{M,u}}$$

The factor 1.5 in the equation for $F_{Rd,u}$ includes an allowance for the axial compression in the column.

where:

$$\eta_1 = \frac{(n_1-1)p_1 - \frac{n_1}{2}d_0}{d_2}$$

$$\beta_1 = \frac{p_3}{d_2}$$

$$\gamma_1 = \frac{d_0}{d_2}$$

d_2 is the depth of the column between fillets

d_0 is the diameter of the hole

$t_{w,2}$ is the thickness of the column web

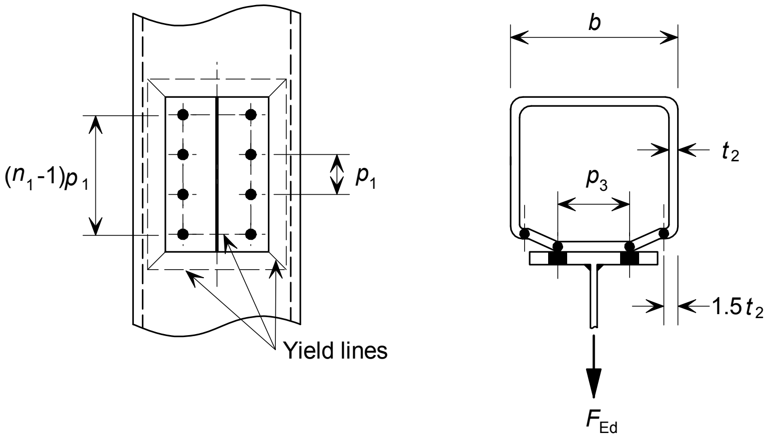
n_1 is the number of bolt rows

Note:

This check is required for either single-sided connections to the column web or unequally loaded double-sided connections to the column web.

If the beam is connected to a column flange, the tying resistance of the column flange may be assessed using the procedures given in Reference 19.

CHECK 15	Tying resistance – supporting column wall (hollow section)



Resistance of hollow section wall with axial compression in the column.

Tying resistance of hollow section wall

Basic requirement:

$F_{Ed} \quad \leq F_{Rd,u}$

$$F_{Rd,u} = \frac{8M_{pl,Rd,u}}{(1-\beta_1)}\left[\eta_1 + 1.5(1-\beta_1)^{0.5} \times (1-\gamma_1)^{0.5}\right]$$

$$M_{pl,Rd,u} = \frac{f_{u,2}\,t_2^{\,2}}{4\gamma_{M,u}}$$

The factor 1.5 in the equation for $F_{Rd,u}$ includes an allowance for the axial compression in the column.

See Appendix B for more details about prying effects.

where:

$$\eta_1 = \frac{(n_1-1)p_1 - \frac{n_1}{2}d_0}{(b-3t_2)}$$

$$\beta_1 = \frac{p_3}{(b-3t_2)}$$

$$\gamma_1 = \frac{d_0}{(b-3t_2)}$$

t_2 is the thickness of the hollow section column

d_0 is the diameter of hole in hollow section (the bolt diameter for Flowdrill or the hole diameter given from table G.69 of yellow pages for Hollo-Bolts)

n_1 is the number of rows of bolts

4.6 WORKED EXAMPLES WITH PARTIAL DEPTH END PLATES

The worked examples show design calculations for typical standard joints. Each example first demonstrates the use of the resistance tables (yellow pages) and then the 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 joints, 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 9 is shown for completeness, but is unlikely to be critical for end plates with bolts spaced at reasonable centres.

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 tying resistance. The magnitude of the tie force depends on the class of building. When the tie force is 75 kN, no checks are required, as every standard detail will accommodate this force. Calculations will be required for larger forces.

Example 1

Example 1 covers the design checks for a two-sided beam to beam connection. A 200 × 12 end plate is used for the large beam and a 200 × 10 end plate is used for the smaller beam, 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 a hollow section column using normal grade 8.8 bolts in Flowdrill threaded holes to connect a partial depth end plate to the column wall. The beam sizes and vertical reactions are as in Example 1, so only the checks which are different to those in the first example are shown. A 150 × 10 end plate is used for the smaller beam and a 200 × 12 for the larger beam since the connecting bolts to the column are not common to both beams.

Example 4

Example 4 covers the same beam for connections to a hollow section column as in Example 3 but uses Hollo-Bolts to connect a flexible end plate to the column wall. Hollo-Bolts require larger than normal holes so different standard details (see Table G.2) have to be used to maintain an adequate edge distance.

Example 5

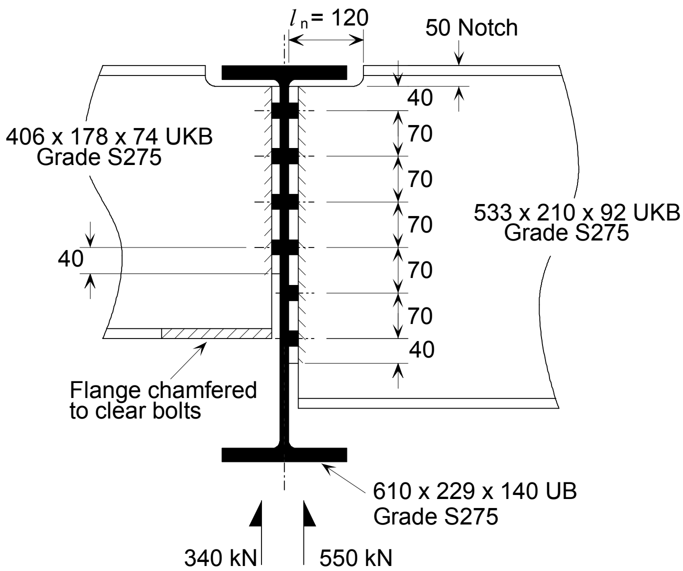
Example 4 covers the same beam for connections to a hollow section column as in Example 4 but uses blind bolts to connect a flexible end plate to the column wall.

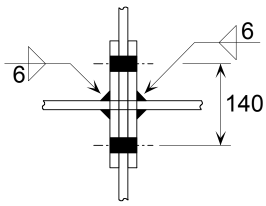
SCI Steel Knowledge **CALCULATION SHEET** BCSA	Job	*Joints in Steel Construction – Simple connections*		*Sheet 1 of 12*
	Title	*Example 1 – Partial depth end plate – Beam to beam*		
	Client	*Connections Group*		
	Calcs by *CZT*	Checked by *ENM*	Date	*Sept 2011*

DESIGN EXAMPLE 1

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

Yellow pages are used for the initial selection of the connection detail.





Partial depth end plates:

For 406 × 178 UKB: 200 × 10

For 533 × 210 UKB: 200 × 12

Design Information:

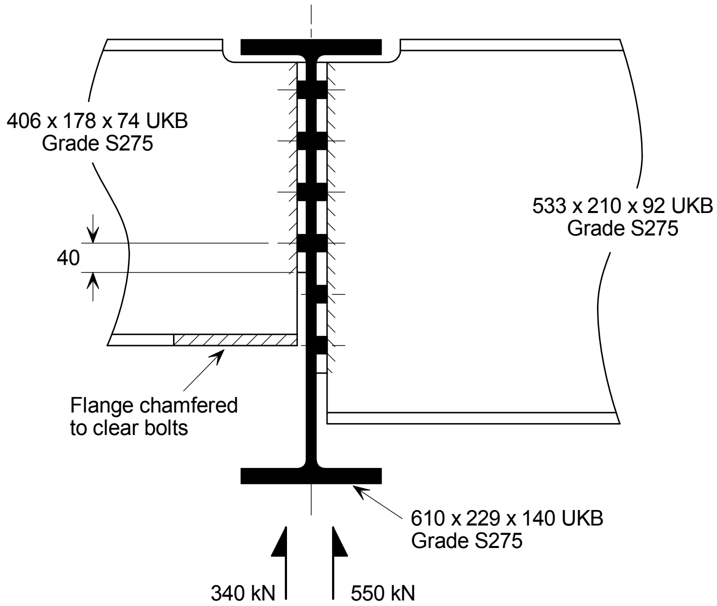
Bolts:	M20 8.8
Welds:	All 6 mm leg length fillet welds
Material:	All S275

Note:

End plate size and details are governed by the larger beam, hence the end plate for the smaller beam is 200 × 10 and not 150 × 10 as recommended.

Title Example 1 – Partial depth end plate – Beam to beam *Sheet 2 of 12*

CONNECTION DESIGN USING RESISTANCE TABLES



406 × 178 × 74 UKB Grade S275	533 × 210 × 92 UKB Grade S275
End plate, 200 × 10	**End plate, 200 × 12**
Note: End plate size is determined by the size of the opposing beam.	
Welds 6 mm fillet	Welds 6 mm fillet
Bolts M20 8.8	Bolts M20 8.8
Bolts at 140 mm cross centres	Bolts at 140 mm cross centres
4 rows of bolts	6 rows of bolts
From Table G.4	From Table G.4
The values obtained from the resistance Table G.4 are for a 150 × 10 end plate, but may be used for a 200 × 10 end plate	
Connection shear resistance (single notch) = 394 kN > 340 kN	Connection shear resistance (single notch) = 621 kN > 550 kN
Maximum notch length = 192 mm > 120 mm	Maximum notch length = 186 mm > 122 mm

Web thickness of supporting beam = 13.1 mm

Minimum support thickness = (3.7 + 3.9) = 7.6 mm < 13.1 mm **∴ O.K.**

Connection is adequate

Note that the minimum support thicknesses are based on the connection resistance, not the applied shear. If necessary, the actual minimum support thicknesses may be calculated as:

$$3.7 \times \frac{340}{394} + 3.9 \times \frac{550}{621} = 6.7 \text{ mm}$$

Title Example 1 – Partial depth end plate – Beam to beam *Sheet 3 of 12*

SUMMARY OF FULL DESIGN CHECKS FOR EXAMPLE 1

Sheet No.	CHECK		406 UKB (S275)		533 UKB (S275)		610 UKB (S275)	
			Resistance	Design force	Resistance	Design force	Resistance	Design force
4	***Check 1*** Recommended detailing practice		All recommendations adopted					
4	***Check 2*** Supported beam Welds	Resistance (kN)	Full strength welds adopted – not critical				Not applicable	
	Check 3		Not applicable				Not applicable	
4	***Check 4*** Supported beam Web in shear	Shear resistance (kN)	394	340	621	550	Not applicable	
5	***Check 5*** Supported beam Resistance at a notch	Bending resistance (kNm)	85	41	140	67	Not applicable	
7	***Check 6*** Supported beam Local stability at a notch	Notch length (mm)	413	110	533	110	Not applicable	
			Notch length l_n < specified limits					
	Check 7 Supported beam Overall stability	–	Not applicable (beam restrained)				Not applicable	
7	***Check 8*** Connection Bolt group	Bolt group resistance (kN)	602	340	902	550	Not applicable	
9	***Check 9*** Connection End plate in shear	Shear resistance (kN)	691	340	1195	550	Not applicable	
11	***Check 10*** Supporting beam Shear and bearing	Shear resistance (kN)	Not applicable				742	354
		Bearing resistance (kN)					174	88

Title *Example 1 – Partial depth end plate – Beam to beam* — *Sheet 4 of 12*

CONNECTION DESIGN FOLLOWING THE DESIGN PROCEDURES

Check 1: Recommended detailing practice

End plates: 200 × 10 mm and 200 × 12 mm

Height of plates: h_p = 290 mm (> $0.6h_{b1}$ for 406UKB)

= 430 mm (> $0.6h_{b1}$ for 533UKB)

Bolts: M20, 8.8 at 140 mm cross centres

Check 2: Supported beam – Welds

Resistance of fillet welds to connecting end plate to beam web.

Basic requirement: $a \geq 0.40\ t_{w,b1}$

406 × 178 × 74 UKB, S275

Throat thickness of 6 mm fillet weld, $a = \dfrac{6}{\sqrt{2}}$ = 4.24 mm

$0.40 \times t_{w,b1}$ = 0.40 × 9.5 = 3.8 mm

A = 4.24 mm > 3.8 mm — ∴ O.K.

533 × 210 × 92 UKB, S275

Throat thickness of 6 mm fillet weld, $a = \dfrac{6}{\sqrt{2}}$ = 4.24 mm

$0.40 \times t_{w,b1}$ = 0.40 × 10.1 = 4.04 mm

a = 4.24 mm > 4.04 mm — ∴ O.K.

Check 4: Supported beam – Web in shear

Shear resistance of beam web at the end plate

Basic requirement: $V_{Ed} \leq V_{c,Rd}$

Shear resistance of beam web at the end plate: $V_{c,Rd} = \dfrac{A_v f_{y,b1} / \sqrt{3}}{\gamma_{M0}}$

406 × 178 × 74 UKB, S275

Shear area of beam web at the connection:

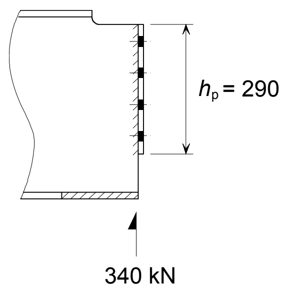
A_v = 0.9 × 290 × 9.5 = 2480 mm^2

t_f = 16.0 mm, hence $f_{y,b1}$ = 275 N/mm^2

γ_{M0} = 1.0

Shear resistance of beam web at the connection:

$\therefore V_{pl,Rd} = \dfrac{2480 \times 275 / \sqrt{3}}{1.0} \times 10^{-3}$ = 394 kN



V_{Ed} = 340 kN ≤ 394 kN — **∴ O.K.**

533×210×92 UKB, S275

Shear area of beam web at the connection:

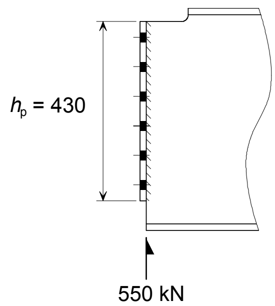
A_v = 0.9 × 430 × 10.1 = 3909 mm^2

t_f = 15.6 mm, hence $f_{y,b1}$ = 275 N/mm^2

γ_{M0} = 1.0 — **∴ O.K.**

Shear resistance of beam web at the connection:

$\therefore V_{pl,Rd} = \dfrac{3909 \times 275 / \sqrt{3}}{1.0} \times 10^{-3}$ = 621 kN



V_{Ed} = 550 kN $\leq$ 621 kN

Check 5: Supported beam – Resistance at a notch

Shear and bending interaction at the notch

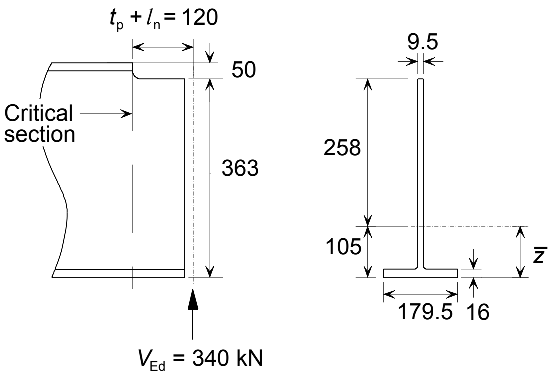
Basic requirement: $V_{Ed}(t_p + l_n) \leq M_{v,N,Rd}$

406 × 178 × 74 UKB, S275

For low shear conditions:

$V_{Ed} \leq 0.5V_{pl,N,Rd}$

$$V_{pl,N,Rd} = \frac{A_{v,N} f_{y,b1} / \sqrt{3}}{\gamma_{M0}}$$



Total area of T-section:

A_{Tee} = $(363 - 16) \times 9.5 + 16 \times 179.5$

= 6169 mm^2

Shear area of T-section:

$A_{v,N}$ = $6169 - 179.5 \times 16 + (9.5 + 2 \times 10.2)\frac{16}{2}$ = 3536 mm^2

$$V_{pl,N,Rd} = \frac{3536 \times 275 / \sqrt{3}}{1.0} \times 10^{-3} = 561 \text{ kN}$$

$0.5V_{pl,N,Rd} = 0.5 \times 561$ = 281 kN

V_{Ed} = 340 > 281 kN therefore high shear conditions apply and the moment resistance is given by:

$$M_{v,N,Rd} = \frac{f_{y,b1} W_{el,N,y}}{\gamma_{M0}} \left(1 - \left(\frac{2V_{Ed}}{V_{pl,N,Rd}} - 1 \right)^2 \right)$$

Calculation of elastic section modulus, $W_{el,N,y}$

Position of the neutral axis: take moments of area about the bottom of the bottom flange:

$$(179.5 \times 16 \times 8) + \left(347 \times 9.5 \times \left(\frac{347}{2} + 16 \right) \right) = ((179.5 \times 16) + (347 \times 9.5)) \times \bar{z}$$

$\therefore \bar{z}$ = 105 mm from the bottom of the section

Second moment of area about the neutral axis:

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

$\therefore I_{yy}$ = 8370 cm^4

Elastic section modulus:

$$W_{el,N,y} = \frac{I_{yy}}{z_{max}} = \frac{8370 \times 10^4}{258} \times 10^{-3} = 324 \text{ cm}^3$$

Moment resistance of the beam at the notch in the presence of shear

$$M_{v,N,Rd} = \frac{324 \times 10^3 \times 275}{1.0} \left(1 - \left(\frac{2 \times 340}{561} - 1 \right)^2 \right) \times 10^{-6} = 85.1 \text{ kNm}$$

Eccentric moment

$V_{Ed}(t_p + l_n)$ = $340 \times (10 + 110) \times 10^{-3}$ = 41 kNm

41 kNm $\leq$ 85.1 kNm **∴ O.K.**

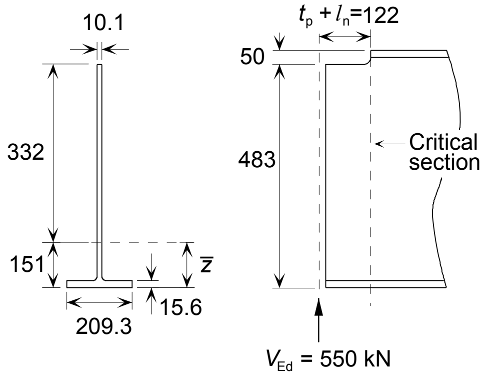
Title *Example 1 – Partial depth end plate – Beam to beam* *Sheet 6 of 12*

533 × 210 × 92 UKB, S275

For low shear conditions:

$V_{Ed} \leq 0.5 V_{pl,N,Rd}$

$$V_{pl,N,Rd} = \frac{A_{v,N} f_{y,b1} / \sqrt{3}}{\gamma_{M0}}$$



Total area of T-section:

A_{Tee} = (483 – 15.6) × 10.1 + 15.6 × 209.3
= 7986 mm^2

Shear area of T-section:

$$A_{v,N} = 7986 - 209.3 \times 15.6 + (10.1 + 2 \times 12.7) \times \frac{15.6}{2} = 4998 \text{ mm}^2$$

$$V_{pl,N,Rd} = \frac{4998 \times 275 / \sqrt{3}}{1.0} \times 10^{-3} = 794 \text{ kN}$$

$0.5V_{pl,N,Rd}$ = 0.5 × 794 = 397 kN

V_{Ed} = 550 kN > 397 kN therefore high shear conditions apply and the moment resistance is given by:

$$M_{v,N,Rd} = \frac{f_{y,b1} W_{el,N,y}}{\gamma_{M0}} \left(1 - \left(\frac{2V_{Ed}}{V_{pl,N,Rd}} - 1 \right)^2 \right)$$

Calculation of elastic section modulus, $W_{el,N,y}$

Position of the neutral axis: take moments of area about the bottom of the bottom flange:

$$\left(209.3 \times 15.6 \times \frac{15.6}{2} \right) + \left(467.4 \times 10.1 \times \left(\frac{483}{2} + 15.6 \right) \right) = \left((209.3 \times 15.6) + (483 \times 10.1) \right) \times \bar{z}$$

$\therefore \ \bar{z}$ = 151 mm from the bottom of the section

Second moment of area about neutral axis:

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

$\therefore \ I_{yy}$ = 19858 cm^4

Elastic section modulus:

$$W_{el,N,y} = \frac{I_{yy}}{z_{max}} = \frac{19858 \times 10^4}{332} \times 10^{-3} = 598 \text{ cm}^3$$

Moment resistance of the beam at the notch in the presence of shear

$$M_{v,N,Rd} = \frac{598 \times 10^3 \times 275}{1.0} \times \left(1 - \left(\frac{2 \times 550}{794} - 1 \right)^2 \right) \times 10^{-3} = 140 \text{ kNm}$$

Eccentric moment

$V_{Ed}(t_p + l_n)$ = 550 × (110 + 12) × 10^{-3} = 67 kNm

67 kNm ≤ 140 kNm **∴ O.K.**

Note:

The verification of the section class does not need to be checked as the length of the notched beam is relatively small. Thus it is assumed that the section is able to reach a full elastic resistance.

Title Example 1 – Partial depth end plate – Beam to beam	Sheet 7 of 12

Check 6: Supported beam – Local stability of notched beam

When the beam is restrained against lateral torsional buckling no account need be taken of stability of the notch stability provided that for a S275 beam with one flange notched the basic requirements are satisfied.

Basic requirements:

Notch depth: $d_{n,t} \leq h_{b1}/2$

Notch length: $l_n \leq h_{b1}$ for $\dfrac{h_{b1}}{t_{w,b1}} \leq 54.3$

$l_n \leq \dfrac{160000 h_{b1}}{\left(h_{b1}/t_{w,b1}\right)^3}$ for $\dfrac{h_{b1}}{t_{w,b1}} > 54.3$

406 × 178 × 74 UKB, S275

Notch depth:

$\dfrac{h_{b1}}{2} = \dfrac{412.8}{2} = 206.4$ mm

$d_{n,t} = 50$ mm ≤ 206.4 mm ∴ **O.K.**

Notch length:

$\dfrac{h_{b1}}{t_{w,b1}} = \dfrac{412.8}{9.5} = 43 \leq 54.3$

$l_n = 110$ mm ≤ 412.8 mm ∴ **O.K.**

533 × 210 × 92 UKB, S275

Notch depth:

$\dfrac{h_{b1}}{2} = \dfrac{533.1}{2} = 266.6$ mm

$d_n = 50$ mm ≤ 266.6 mm ∴ **O.K.**

Notch length:

$\dfrac{h_{b1}}{t_{w,b1}} = \dfrac{533.1}{10.1} = 52.8 \leq 54.3$

$l_n = 110$ mm ≤ 533.1 mm ∴ **O.K.**

Check 8: Connection – Bolt group

Basic requirement: $V_{Ed} \leq F_{Rd}$

The resistance of the bolt group, F_{Rd}, is as follows:

If $F_{b,Rd} \leq 0.8F_{v,Rd}$ then $F_{Rd} = nF_{b,Rd}$

if $F_{b,Rd} > 0.8F_{v,Rd}$ then $F_{Rd} = 0.8nF_{v,Rd}$

Shear resistance of a single bolt:

$$F_{v,Rd} = \frac{\alpha_v f_{ub} A}{\gamma_{M2}}$$

For M20 8.8 bolts:

$$F_{v,Rd} = \frac{0.6 \times 800 \times 245}{1.25} \times 10^{-3} = 94 \text{ kN}$$

Bearing resistance of a single bolt:

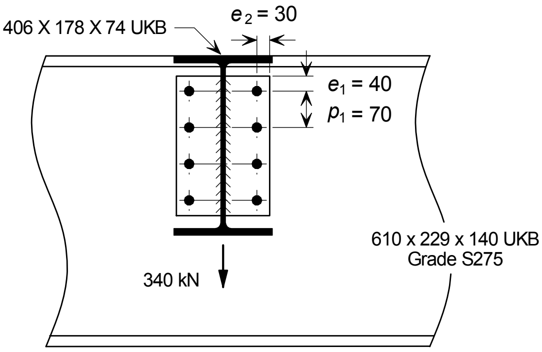
$F_{b,Rd} = \min(F_{b,Rd,p};\ F_{b,Rd,2})$

$$F_{b,Rd,p} = \frac{k_{1,p}\,\alpha_{b,p}\,f_{u,p}\,d t_p}{\gamma_{M2}}$$

$$F_{b,Rd,2} = \frac{k_{1,2}\,\alpha_{b,2}\,f_{u,2}\,d t_2}{\gamma_{M2}}$$

Bearing on the supporting beam will be verified in Check 10

406 × 178 × 74 UKB, S275



Bearing on the end plate:

$$k_{1,p} = \min\left(2.8\frac{e_2}{d_0} - 1.7;\ 1.4\frac{p_3}{d_0} - 1.7; 2.5\right) = \min\left(2.8\times\frac{30}{22} - 1.7;\ 1.4\times\frac{140}{22} - 1.7;\quad 2.5\right)$$

$= \min(2.12;\ 7.21;\ 2.5)$

$\therefore\ k_{1,p} = 2.12$

$$\alpha_{b,p} = \min\left(\frac{e_1}{3d_0};\quad \frac{p_1}{3d_0} - \frac{1}{4};\quad \frac{f_{ub}}{f_{u,p}};\quad 1.0\right)$$

$$\alpha_{b,p} = \min\left(\frac{40}{3\times 22};\quad \frac{70}{3\times 22} - 0.25;\quad \frac{800}{410};\quad 1.0\right) = \min(0.61;\quad 0.81;\quad 1.95;\quad 1.0)$$

$\therefore\ \alpha_{b,p} = 0.61$

$$F_{b,Rd,p} = \frac{2.12\times 0.61\times 410\times 20\times 10}{1.25}\times 10^{-3} = 85 \text{ kN}$$

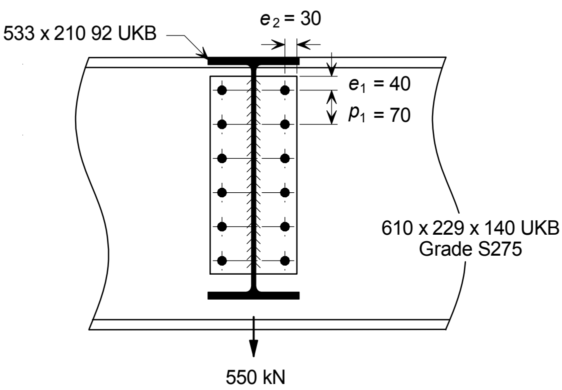
$0.8 \times F_{v,Rd} = 0.8 \times 94 = 75.2$ kN

85 kN > 75.2 kN

$\therefore\ F_{Rd} = 0.8 n F_{v,Rd} = 0.8 \times 8 \times 94 = 602$ kN

$\therefore\ V_{Ed} = 340$ kN ≤ 602 kN **∴ O.K.**

533 × 210 × 92 UKB, S275



Bearing on the end plate:

$$k_{1,p} = \min\left(2.8\frac{e_2}{d_0} - 1.7; 1.4\frac{p_3}{d_0} - 1.7; 2.5\right) = \min\left(2.8\times\frac{30}{22} - 1.7; 1.4\times\frac{140}{22} - 1.7; 2.5\right)$$

$= \min(2.12;\ 7.21;\ 2.5)\ \therefore\ k_{1,p} = 2.12$

$$\alpha_{b,p} = \min\left(\frac{e_1}{3d_0};\quad \frac{p_1}{3d_0} - \frac{1}{4};\quad \frac{f_{ub}}{f_{u,p}};\quad 1.0\right)$$

$$\alpha_{b,p} = \min\left(\frac{40}{3\times 22};\quad \frac{70}{3\times 22} - 0.25;\quad \frac{800}{410};\quad 1.0\right) = \min\left(0.61;\quad 0.81;\quad 1.95;\quad 1.0\right)$$

$\therefore\ \alpha_{b,p} = 0.61$

$$F_{b,Rd,p} = \frac{2.12\times 0.61\times 410\times 20\times 12}{1.25}\times 10^{-3} = 102 \text{ kN}$$

$0.8 \times F_{v,Rd} = 0.8 \times 94 = 75.2$ kN

$\therefore\ F_{b,Rd} = 102 \text{ kN} > 75.2 \text{ kN}$

$\therefore\ F_{Rd} = 0.8nF_{v,Rd} = 0.8 \times 12 \times 94 = 902$ kN

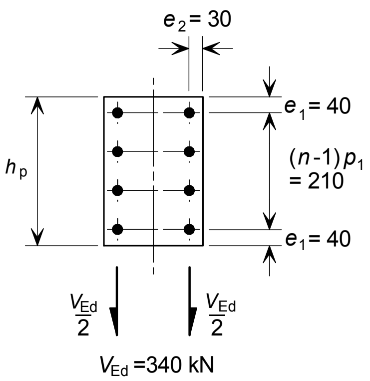
$\therefore\ V_{Ed} = 550 \text{ kN} \leq 902 \text{ kN}$ **$\therefore$ O.K.**

Check 9: Connection – End plate in shear

Basic requirement: $V_{Ed} \leq V_{Rd,min}$

$V_{Rd,min} = \min(V_{Rd,g};\quad V_{Rd,n};\quad V_{Rd,b})$

406 × 178 × 74 UKB, S275



Shear resistance of gross section:

$$V_{Rd,g} = \frac{2h_p t_p}{1.27}\frac{f_{y,p}}{\sqrt{3}\gamma_{M0}}$$

$$V_{Rd,g} = \frac{2\times 290\times 10}{1.27}\times\frac{275}{\sqrt{3}\times 1.0}10^{-3} = 725 \text{ kN}$$

Shear resistance of the net section:

$$V_{Rd,n} = 2A_{v,net}\frac{f_{u,p}}{\sqrt{3}\gamma_{M2}}$$

Net area:

$A_{v,net} = A - nd_0t_p = 290 \times 10 - 4 \times 22 \times 10 = 2020 \text{ mm}^2$

$$\therefore\ V_{Rd,n} = 2\times 2020\times\frac{410}{\sqrt{3}\times 1.1}\times 10^{-3} = 869 \text{ kN}$$

Block tearing resistance (block shear):

$1.36p_3 = 1.36 \times 140 = 190$ mm

$h_p = 290 \text{ mm} > 190$ mm, therefore

$$V_{Rd,b} = 2\left(\frac{f_{u,p}A_{nt}}{\gamma_{M2}} + \frac{f_{y,p}A_{nv}}{\sqrt{3}\gamma_{M0}}\right)$$

Net area subject to tension:

$$A_{nt} = t_p\left(e_2 - \frac{d_0}{2}\right) = 10\times\left(30 - \frac{22}{2}\right) = 190 \text{ mm}^2$$

Net area subject to shear:

$$A_{nv} = t_p\left(h_p - e_1 - \left(n_1 - 0.5\right)d_0\right) = 10\times\left(290 - 40 - \left(4 - 0.5\right)\times 22\right) = 1730 \text{ mm}^2$$

$\therefore\ V_{\mathrm{Rd,b}} = 2\left(\dfrac{410\times190}{1.1}+\dfrac{275\times1730}{\sqrt{3}\times1.0}\right)\times10^{-3} = 691\text{ kN}$

$V_{\mathrm{Rd,min}} = \min(725;\ 869;\ 691) = 691\text{ kN}$

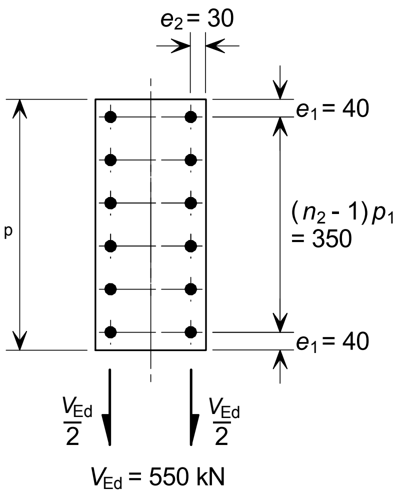
$\therefore\ V_{\mathrm{Ed}} = 340\text{ kN} < 691\text{ kN}$ **∴ O.K.**

533 × 210 × 92 UKB, S275

Shear resistance of gross section:

$V_{\mathrm{Rd,g}} = \dfrac{2h_{\mathrm{p}}t_{\mathrm{p}}}{1.27}\dfrac{f_{\mathrm{y,p}}}{\sqrt{3}\gamma_{\mathrm{M0}}}$

$V_{\mathrm{Rd,g}} = \dfrac{2\times430\times12}{1.27}\dfrac{275}{\sqrt{3}\times1.0}\times10^{-3} = 1290\text{ kN}$



Shear resistance of net section:

$V_{\mathrm{Rd,n}} = 2A_{\mathrm{v,net}}\dfrac{f_{\mathrm{u,p}}}{\sqrt{3}\gamma_{\mathrm{M2}}}$

Net area:

$A_{\mathrm{v,net}} = A - n\,d_0\,t_{\mathrm{p}}$

$A_{\mathrm{v,net}} = 430\times12 - 6\times22\times12 = 3576\text{ mm}^2$

$\therefore\ V_{\mathrm{Rd,n}} = 2\times3576\times\dfrac{410}{\sqrt{3}\times1.1}\times10^{-3} = 1539\text{ kN}$

Block tearing resistance (Block shear):

$1.36p_3 = 1.36\times140 = 190\text{ mm}$

$h_{\mathrm{p}} = 430\text{ mm} > 190\text{ mm}$, therefore

$V_{\mathrm{Rd,b}} = 2\left(\dfrac{f_{\mathrm{u,p}}A_{\mathrm{nt}}}{\gamma_{\mathrm{M2}}}+\dfrac{f_{\mathrm{y,p}}A_{\mathrm{nv}}}{\sqrt{3}\gamma_{\mathrm{M0}}}\right)$

Net area subject to tension:

$A_{\mathrm{nt}} = t_{\mathrm{p}}\left(e_2-\dfrac{d_0}{2}\right) = 12\left(30-\dfrac{22}{2}\right) = 228\text{ mm}^2$

Net area subject to shear:

$A_{\mathrm{nv}} = t_{\mathrm{p}}\left(h_{\mathrm{p}}-e_1-(n-0.5)d_0\right) = 12\times\left(430-40-(6-0.5)\times22\right) = 3228\text{ mm}^2$

$\therefore\ V_{\mathrm{Rd,b}} = 2\times\left(\dfrac{410\times228}{1.1}+\dfrac{275\times3228}{\sqrt{3}\times1.0}\right)\times10^{-3} = 1195\text{ kN}$

$V_{\mathrm{Rd,min}} = \min(1290;\ 1539;\ 1195) = 1195\text{ kN}$

$\therefore\ V_{\mathrm{Ed}} = 550\text{ kN} < 1195\text{ kN}$ **∴ O.K.**

Additional requirement: End plate in bending

For 406 × 178 × 74 UKB;

$h_{\mathrm{p}} = 290\text{ mm}$

$1.36\,p_3 = 1.36\times140 = 190$ mm, < 290, so no additional check required

For 533 × 210 × 92 UKB;

$h_{\mathrm{p}} = 430\text{ mm}$

$1.36\,p_3 = 1.36\times140 = 190$ mm, < 430, so no additional check required

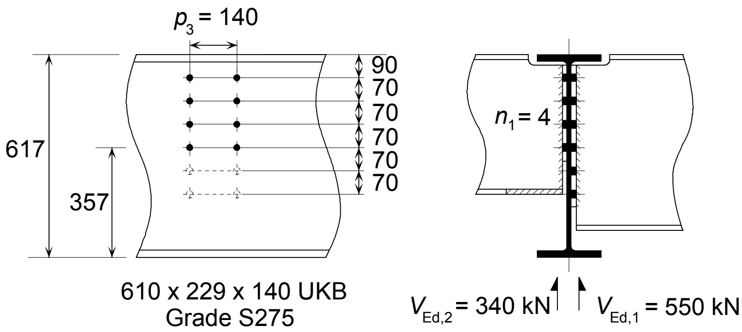
Title *Example 1 – Partial depth end plate – Beam to beam* *Sheet 11 of 12*

Check 10: Supporting beam – shear and bearing

Local shear and bearing resistance of beam web supporting two beams:

Shear

Basic requirement: $\dfrac{V_{\mathrm{Ed,1,A}}}{2} + \dfrac{V_{\mathrm{Ed,2}}}{2} \leq V_{\mathrm{Rd,min}}$



Applied shear over the top four rows of bolts:

$550 \times \dfrac{4}{6} + 340$ = 707 kN or 354 kN per plane

Shear resistance of supporting beam web:

$$V_{\mathrm{Rd,min}} = \min\left(\frac{A_{\mathrm{v}} f_{\mathrm{y,2}}}{\sqrt{3}\gamma_{\mathrm{M0}}};\ \frac{A_{\mathrm{v,net}} f_{\mathrm{u,2}}}{\sqrt{3}\gamma_{\mathrm{M2}}}\right)$$

Shear area of gross section:

$A_{\mathrm{v}} = \left(e_{\mathrm{t}} + (n_1 - 1)p_1 + e_{\mathrm{b}}\right)t_2$

$e_{\mathrm{t}} = \min\left(e_{3,\mathrm{t}};\ 5d\right) = \min\left(90;\ 100\right)$ = 90 mm

$e_{\mathrm{b}} = \min\left(e_{3,\mathrm{b}};\ p_3/2;\ 5d\right) = \min\left(357;\ 70;\ 100\right)$ = 70 mm

$A_{\mathrm{v}} = 13.1 \times \left(90 + (4-1) \times 70 + 70\right)$ = 4847 mm²

t_{f} = 22.1 mm > 16 mm, therefore f_{y} = 265 N/mm²

Therefore the shear resistance of the gross section is:

$$\frac{A_{\mathrm{v}} f_{\mathrm{y,2}}}{\sqrt{3}\gamma_{\mathrm{M0}}} = \frac{4847 \times 265}{\sqrt{3} \times 1.0} \times 10^{-3} = 742 \text{ kN}$$

Shear area of net section:

$A_{\mathrm{v,net}} = A_{\mathrm{v}} - n d_0 t_2 = 4847 - 4 \times 22 \times 13.1$ = 3694 mm²

Therefore the shear resistance of the net section is:

$$\frac{A_{\mathrm{v,net}} f_{\mathrm{u,2}}}{\sqrt{3}\,\gamma_{\mathrm{M0}}} = \frac{3694 \times 410}{\sqrt{3} \times 1.1} \times 10^{-3} = 795 \text{ kN}$$

$\therefore V_{\mathrm{Rd,min}} = \min(742;\ 795)$ = 742 kN

$\therefore$ 354 kN < 742 kN **∴ O.K.**

Title Example 1 – Partial depth end plate – Beam to beam	*Sheet 12 of 12*

Bearing

$$\frac{V_{\mathrm{Ed},1}}{2n_1}+\frac{V_{\mathrm{Ed},3}}{2n_3} \leq F_{\mathrm{b,Rd}}$$

Bearing resistance of beam web:

$$F_{\mathrm{b,Rd}} = \frac{k_{1,\mathrm{b2}}\,\alpha_{\mathrm{d}}\,f_{\mathrm{u,b2}}\,d t_{\mathrm{w,b2}}}{\gamma_{\mathrm{M2}}}$$

$$k_{1,\mathrm{b2}} = \min\left(1.4\frac{p_3}{d_0}-1.7;\quad 2.5\right)$$

$$k_{1,\mathrm{b2}} = \min\left(1.4\times\frac{140}{22}-1.7;\quad 2.5\right) = \min\left(7.2;\quad 2.5\right) = 2.5$$

$$\alpha_{\mathrm{d}} = \min\left(\frac{p_1}{3d_0}-0.25;\quad \frac{f_{\mathrm{ub}}}{f_{\mathrm{u,b2}}};\quad 1.0\right) = \min\left(0.81;\quad 1.95;\quad 1.0\right) = 0.81$$

$$\therefore\ F_{\mathrm{b,Rd}} = \frac{2.5\times0.81\times410\times20\times13.1}{1.25}\times10^{-3} = 174\text{ kN}$$

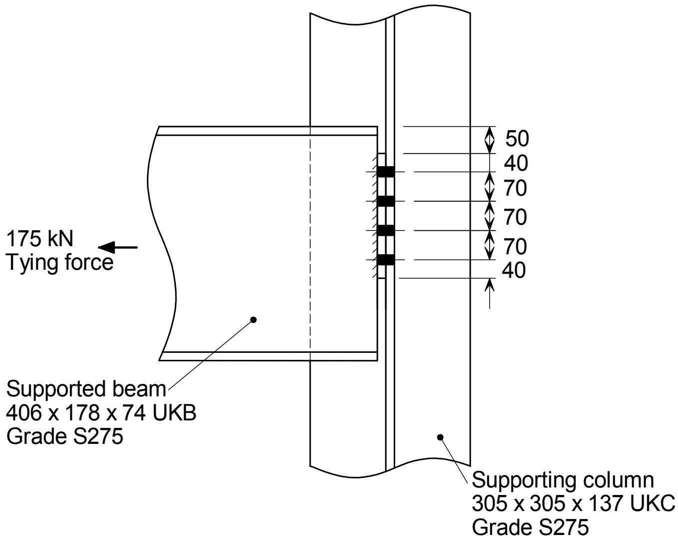
$$\therefore\ \frac{V_{\mathrm{Ed},1}}{2n_1}+\frac{V_{\mathrm{Ed},3}}{2n_3} = \frac{550}{2\times6}+\frac{340}{2\times4} = 88\text{ kN} < 174\text{ kN}$$

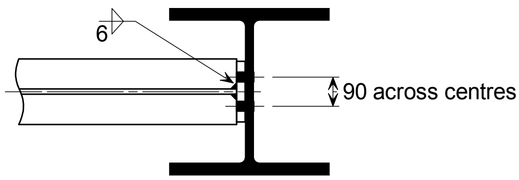
$\therefore$ **O.K.**

Steel Knowledge CALCULATION SHEET BCSA	Job *Joints in Steel Construction – Simple Connections*		Sheet 1 of 6
	Title *Example 2 – Partial depth end plate – Beam to column web – Tying resistance*		
	Client *Connections Group*		
	Calcs by *CZT*	Checked by *ENM*	Date *Sept 2011*

DESIGN EXAMPLE 2

Check the following beam to column joint for the tying force shown.



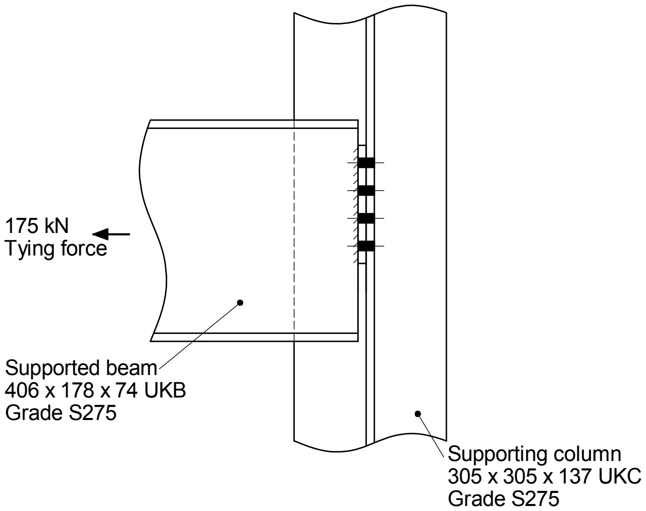


Design Information:

Bolts:	M20 8.8 @ 90 c/c
End plates:	150 × 10
Welds:	6 mm leg length fillet welds
Material:	All S275

Title Example 2 – Partial depth end plate – Beam to column web – Tying resistance	Sheet 2 of 6

CONNECTION DESIGN USING RESISTANCE TABLES



End plate: 150×10
Bolts: M20 8.8
at 90 cross centres
4 rows of bolts
Weld: 6 mm leg length fillet welds

From Table G.4: — Table G.4

Connection tying resistance = 381 kN

Tying force = 175 kN < 381 kN — ∴ **O.K.**

The beam side of the connection is adequate

Notes:

(1) The tying resistance of the connection given in the Tables is the least of the values obtained from Checks 11, 12 & 13.

(2) The column web must also be checked for web bending as shown in Check 14.

Title *Example 2 – Partial depth end plate – Beam to column web – Tying resistance* *Sheet 3 of 6*

SUMMARY OF FULL DESIGN CHECKS FOR EXAMPLE 2

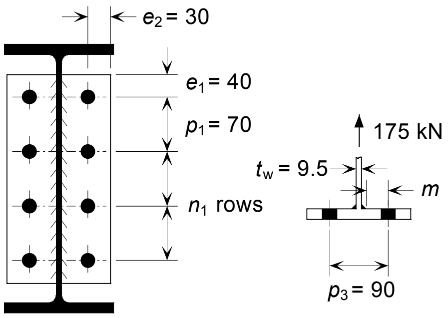
Sheet No.	CHECK	406 UKB (S275)		305 UKC (S275)	
		Resistance	**Design force**	**Resistance**	**Design force**
4	***Check 11*** Tying resistance Plate and bolts (kN)	381	175	Not applicable	
5	***Check 12*** Tying resistance Supported beam web (kN)	1027	175	Not applicable	
5	***Check 13*** Tying resistance Welds (kN)	Full strength welds adopted – not critical		Not applicable	
5	***Check 14*** Tying resistance Tying resistance of column web (kN)	Not applicable		405	175

Title *Example 2 – Partial depth end plate – Beam to column web – Tying resistance* *Sheet 4 of 6*

CONNECTION DESIGN FOLLOWING THE DESIGN PROCEDURES

Check 11: Tying resistance – Plates and bolts

Resistance of end plate



Basic requirement: $F_{Ed} \leq \min(F_{Rd,u,1};\, F_{Rd,u,2};\, F_{Rd,u,3})$

$$F_{Rd,u,1} = \frac{(8n - 2e_w)M_{pl,1,Rd,u}}{2mn - e_w(m+n)}$$

$\Sigma l_{eff} = 2e_{1A} + (n_1 - 1)p_{1A}$

$e_{1A} = e_1$ but $\leq 0.5\left(p_3 - t_{w,b1} - 2a\sqrt{2}\right) + \dfrac{d_0}{2}$

$$0.5 \times \left(90 - 9.5 - 2 \times 4.24 \times \sqrt{2}\right) + \frac{22}{2} = 45.3 \text{ mm}$$

$\therefore\ e_{1A} = 40$ mm

$p_{1A} = p_1$ but $\leq p_3 - t_{w,b1} - 2a\sqrt{2} + d_0$

$p_3 - t_{w,b1} - 2a\sqrt{2} + d_0 = 90 - 9.5 - 2 \times 4.24 \times \sqrt{2} + 22 = 90.5$ mm

$\therefore\ p_{1A} = 70$ mm

$\Sigma l_{eff} = 2\times40 + (4 - 1) \times 70 = 290$ mm

$$M_{pl,1,Rd,u} = \frac{0.25\Sigma l_{eff} t_p^2 f_{u,p}}{\gamma_{M,u}} = \frac{0.25 \times 290 \times 10^2 \times 410}{1.1} \times 10^{-6} = 2.7 \text{ kNm}$$

$$m = \frac{p_3 - t_{w,b} - 2 \times 0.8 \times a\sqrt{2}}{2} = \frac{90 - 9.5 - 2 \times 0.8 \times 4.24 \times \sqrt{2}}{2} = 35.4 \text{ mm}$$

$n = \min(e_2;\ 1.25m) = \min(30;\ 44.3) = 30$ mm

Assuming no washer is used, d_w is taken as the width across points from Table G.66 — Table G.66

$$e_w = \frac{d_w}{4} = \frac{33}{4} = 8.25 \text{ mm}$$

$$\therefore\ F_{Rd,u,1} = \frac{(8 \times 30 - 2 \times 8.25) \times 2.7 \times 10^6}{2 \times 35.4 \times 30 - 8.25 \times (35.4 + 30)} \times 10^{-3} = 381 \text{ kN}$$

$$F_{Rd,u,2} = \frac{2M_{pl,2,Rd,u} + n\Sigma F_{t,Rd,u}}{m+n}$$

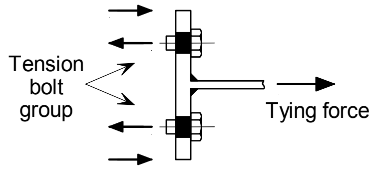
$M_{pl,2,Rd,u} = M_{pl,1,Rd,u} = 2.7$ kNm

$$F_{t,Rd,u} = \frac{k_2 f_{ub} A_s}{\gamma_{M,u}} = \frac{0.9 \times 800 \times 245}{1.1} \times 10^{-3} = 160 \text{ kN}$$

Title *Example 2 – Partial depth end plate – Beam to column web – Tying resistance*	*Sheet 5 of 6*

$$\therefore\ F_{\text{Rd,u,2}} = \frac{2\times 2.7\times 10^{6} + 30\times 8\times 160\times 10^{3}}{35.4+30}\times 10^{-3} = 670\text{ kN}$$

$F_{\text{Rd,u,3}} = \Sigma F_{\text{t,Rd,u}} = 8 \times 160 = 1280$ kN

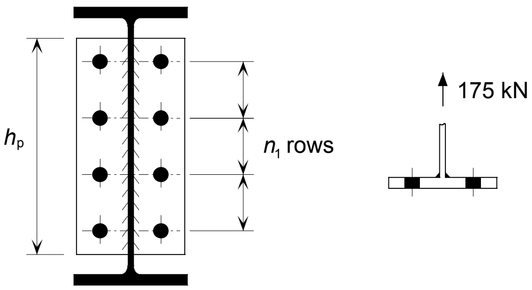


$\min\left(F_{\text{Rd,u,1}},\ F_{\text{Rd,u,2}},\ F_{\text{Rd,u,3}}\right) = \min(381;\ 670;\ 1280) = 381$ kN

F_{Ed} = 175 kN < 381 kN — **∴ O.K.**

Check 12: Tying resistance – Supported beam

Resistance of the beam web.



Basic requirement: $F_{\text{Ed}} \leq F_{\text{Rd}}$

$$F_{\text{Rd}} = \frac{t_{\text{w,b1}} h_{\text{p}} f_{\text{u,b1}}}{\gamma_{\text{M,u}}} = \frac{9.5\times 290\times 410}{1.1}\times 10^{-3} = 1027\text{ kN}$$

F_{Ed} = 175 kN < 1027 kN — **∴ O.K.**

Check 13: Tying resistance – Welds

Resistance of fillet welds connecting end plate to beam web.

Basic requirement: $a \geq 0.40\ t_{\text{w,b1}}$

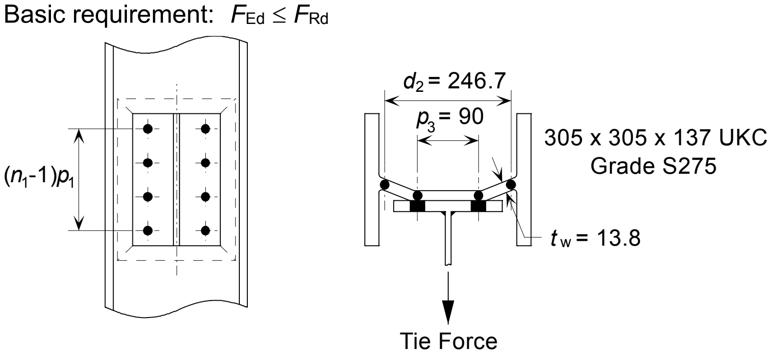
Throat thickness:

$0.40 \times t_{\text{w,b1}} = 0.40 \times 9.5 = 3.8$ mm

a = 4.24 mm > 3.8 mm — **∴ O.K.**

Check 14: Tying resistance – Resistance of supporting column web

Basic requirement: $F_{\text{Ed}} \leq F_{\text{Rd}}$



$$F_{\text{Rd}} = \frac{8 M_{\text{pl,Rd,u}}}{\left(1-\beta_1\right)}\left(\eta_1 + 1.5\left(1-\beta_1\right)^{0.5}\left(1-\gamma_1\right)^{0.5}\right)$$

Title Example 2 – Partial depth end plate – Beam to column web – Tying resistance	Sheet 6 of 6
$M_{\text{pl,Rd,u}} = \dfrac{f_{\text{u,2}}\,t_{\text{w,2}}^{\;2}}{4\gamma_{\text{M,u}}} = \dfrac{410 \times 13.8^2}{4 \times 1.1} \times 10^{-3} = 17.7$ kNm/mm	
$\beta_1 = \dfrac{p_3}{d_2} = \dfrac{90}{246.7} = 0.365$	
$\gamma_1 = \dfrac{d_0}{d_2} = \dfrac{22}{246.7} = 0.089$	
$\eta_1 = \dfrac{(n_1 - 1)p_1 - \dfrac{n_1}{2}d_0}{d_2} = \dfrac{(4-1)\times 70 - \dfrac{4}{2}\times 22}{246.7} = 0.673$	
$\therefore\ F_{\text{Rd}} = \dfrac{8 \times 17.7}{(1-0.365)}\left(0.673 + 1.5(1-0.365)^{0.5}(1-0.089)^{0.5}\right) = 405$ kN	
$F_{\text{Ed}} = 175$ kN < 405 kN	∴ **O.K.**

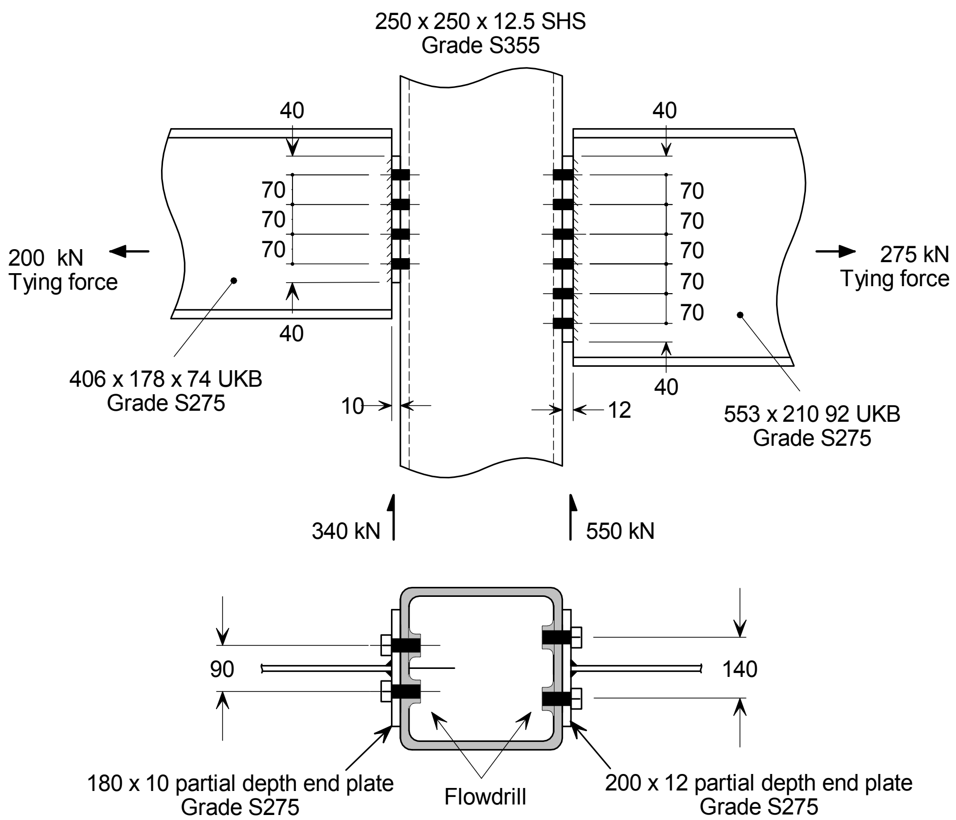
Steel Knowledge **CALCULATION SHEET** BCSA	Job	*Joints in Steel Construction – Simple Joints* Sheet *1* of *9*
	Title	*Example 3 – Partial depth end plate – Beam to hollow section column using Flowdrill*
	Client	*Connections Group*
	Calcs by *CZT*	Checked by *ENM* Date *Sept 2011*

DESIGN EXAMPLE 3

Check the following beam to hollow section column connection for the design forces shown using property class 8.8 bolts in Flowdrill threaded holes in the column.

Note:

The connections should be checked independently for (i) shear forces and (ii) tying forces and not for both forces acting at the same time.



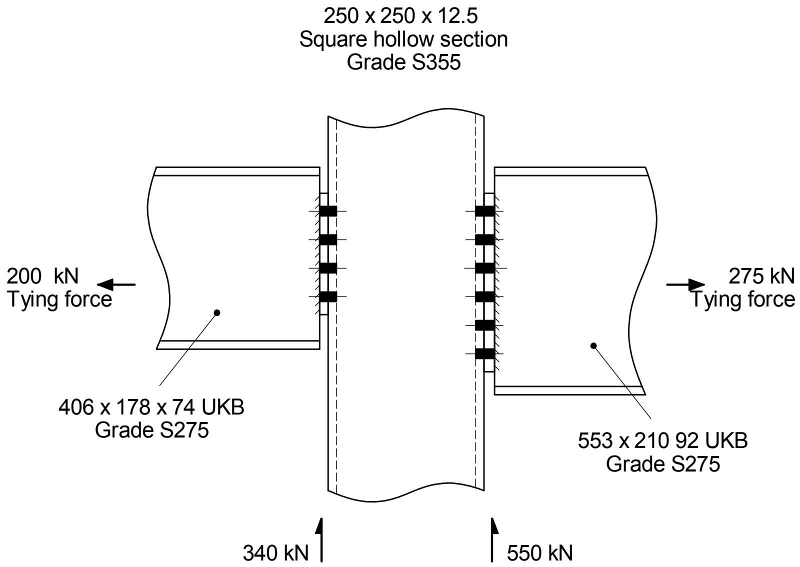
See Figure 4.5 and Table G.4

Design Information:

Bolts:	M20 8.8, Flowdrill
Welds:	All 6 mm leg length fillet welds
Column:	S355
Beams:	S275
End plates:	S275

Title	*Example 3 – Partial depth end plate – Beam to hollow section column using Flowdrill*	*Sheet 2 of 9*

CONNECTION DESIGN USING RESISTANCE TABLES



406 × 178 × 74 UKB Grade S275	533 × 210 × 92 UKB Grade S275	
End plate 150 × 10	**End plate 200 × 12**	
Welds 6 mm fillet	Welds 6 mm fillet	
Bolts M20 8.8	Bolts M20 8.8	
Bolts at 90 cross centres	Bolts at 140 cross centres	
4 rows of bolts	6 rows of bolts	
From Resistance Table G.4	From Resistance Table G.4	Table G.4
Connection shear resistance = 394 kN > 340 kN	Connection shear resistance = 621 kN > 550 kN	
Minimum support thickness = 3.7 mm < 12.5 mm	Minimum support thickness = 3.9 mm < 12.5 mm	
Connection tying resistance = 381 kN > 200 kN	Connection tying resistance = 450 kN > 275 kN	
The beam side of the connection is adequate		

Notes:

(1) The tying resistance of the connection given in the Tables is the least of the values obtained from Checks 11, 12 & 13.

(2) The hollow section wall must also be checked for bending as shown in Check 15.

Title	Example 3 – Partial depth end plate – Beam to hollow section column using Flowdrill	Sheet 3 of 9

SUMMARY OF FULL DESIGN CHECKS FOR EXAMPLE 3

Notes:

(1) Checks 1 to 7 and 9, where applicable, are generally as shown in Worked Example 1 and are not repeated in this example, but the calculated resistances are summarised below. For completeness, the bearing resistance of the bolts in the hollow section is checked following the procedures in Check 8

(2) Tying forces are ignored when calculating the shear resistance and shear is ignored when calculating the tying resistance.

Sheet No.	CHECK		406 UKB (S275)		533 UKB (S275)		SHS Column S355			
							406 UKB Side		533 UKB Side	
			Resistance	Design force	Resistance	Design force	Resistance	Design force	Resistance	Design force
	Check 1 Recommended detailing practice		All recommendations adopted							
	Check 2 Supported beam - welds (kN)		Full strength welds adopted – not critical				Not applicable			
	Check 3		Not applicable							
	Check 4 Supported beam Web in shear	Shear resistance (kN)	394	340	621	550	Not applicable			
	Check 5, 6, 7		Not applicable				Not applicable			
4	***Check 8*** Connection Bolt group	Bolt group (kN)	602	340	902	550	Not applicable			
	Check 9 Supporting beam End plate in shear	Shear resistance (kN)	691	340	1195	550	Not applicable			
4	***Check 10*** Supporting beam Shear and bearing	Shear and Bearing resistance (kN)	Not applicable				848	170	1233	275
5	***Check 11*** Tying resistance Plate and bolts	Tension (kN)	381	200	451	275	Not applicable			
7	***Check 12*** Tying resistance Supported beam web	Tension (kN)	1027	200	1619	275	Not applicable			
8	***Check 13*** Tying resistance Welds	Tension (kN)	Full strength welds adopted – not critical				Not applicable			
	Check 14		Not applicable				Not applicable			
8	***Check 15*** Tying resistance Supporting column wall	Tension (kN)	Not applicable				437	200	862	275

CONNECTION DESIGN FOLLOWING THE DESIGN PROCEDURES

Check 8: Connection – Bolt group

The calculation of bolt shear resistance and bearing in the end plate is identical to that shown in Example 1.

$F_{v,Rd}$ = 94 kN

Bearing resistance

Bearing on the supporting member:

$$F_{b,Rd,2} = \frac{k_{1,2}\alpha_{b,2} f_{u,2} d t_2}{\gamma_{M2}}$$

$$k_{1,2} = \min\left(2.8\frac{e_{2,b}}{d_0} - 1.7;\ 1.4\frac{p_3}{d_0} - 1.7;\ 2.5\right)$$

$e_{2,b}$ is not applicable in a hollow section, where there is no free edge. Therefore,

$$k_{1,2} = \min\left(1.4\frac{p_3}{d_0} - 1.7;\ 2.5\right) = \min\left(1.4\frac{90}{22} - 1.7;\ 2.5\right) = \min\left(4.03;\ 2.5\right) = 2.5$$

For the supporting member:

$$\alpha_{b,2} = \min\left(\frac{p_1}{3d_0} - \frac{1}{4};\ \frac{f_{ub}}{f_{u,2}};\ 1.0\right) = \min\left(\frac{70}{3\times 22} - \frac{1}{4};\ \frac{800}{470};\ 1.0\right)$$

$$= \min\left(0.84;\ 1.70;\ 1.0\right) = 0.81$$

$$F_{b,Rd,2} = \frac{2.5\times 0.81\times 470\times 20\times 12.5}{1.25}\times 10^{-3} = 190 \text{ kN}$$

$0.8 \times F_{v,Rd} = 0.8 \times 94 = 75.2$ kN

75.2 < 190 kN, therefore $F_{Rd} = 0.8nF_{v,Rd}$

For 406 × 178 × 74 UKB connection, $F_{Rd} = 0.8 \times 8 \times 94 = 602$ kN

For 533 × 210 × 92 UKB connection, $F_{Rd} = 0.8 \times 12 \times 94 = 902$ kN

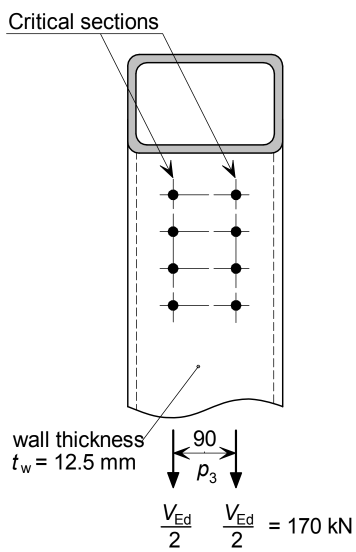
Check 10: Supporting column – Shear and bearing

Local shear and bearing resistance of column wall

Shear

Basic requirement: $\frac{V_{Ed}}{2} \le V_{Rd,min}$

$$V_{Rd,min} = \min\left(\frac{A_v f_{y,2}}{\sqrt{3}\gamma_{M0}};\ \frac{A_{v,net} f_{u,2}}{\sqrt{3}\gamma_{M2}}\right)$$



406 × 178 × 74 UKB, S275

Shear resistance of gross section:

$$A_v = t_2\left(e_t + \left(n_1 - 1\right)p_1 + e_b\right)$$

$$e_b = \min\left(\frac{p_3}{2};\ 5d\right)$$

$$= \min\left(\frac{90}{2};\ 5\times 20\right) = 45 \text{ mm}$$

$$e_t = \min\left(e_{1,t};\ 5d\right)$$

Since the connection is not near the top of the column $e_{1,t}$ is not applicable.

$e_t = 5 \times 20 = 100$ mm

$\therefore A_v = 12.5 \times (100 + (4-1) \times 70 + 45) = 4438 \text{ mm}^2$

t_w < 16 mm, hence $f_{y,2}$ = 355 N/mm²

$\therefore \dfrac{A_v f_{y,2}}{\sqrt{3}\gamma_{M0}} = \dfrac{4438 \times 355}{\sqrt{3} \times 1.0} \times 10^{-3} = 909 \text{ kN}$

Shear resistance of net section:

$A_{v,net} = A_v - n_1 d_0 t_2 = 4438 - 4 \times 20 \times 12.5 = 3438 \text{ mm}^2$

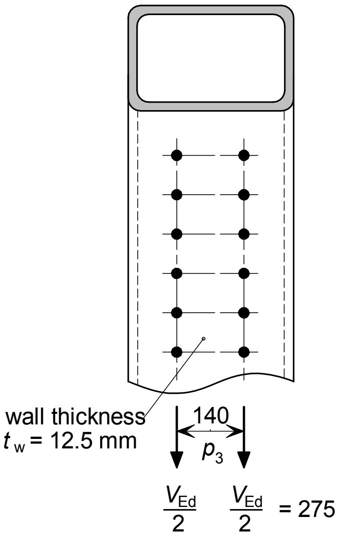
$\therefore \dfrac{A_{v,net} f_{u,2}}{\sqrt{3}\gamma_{M2}} = \dfrac{3438 \times 470}{\sqrt{3} \times 1.1} \times 10^{-3} = 848 \text{ kN}$

$V_{Rd,min}$ = min (909; 848) = 848 kN

$\dfrac{V_{Ed}}{2} = 170 \text{ kN} < 848 \text{ kN}$ $\therefore$ **O.K.**

533 × 210 × 92 UKB, S275

Shear resistance of gross section:



$A_v = t_2 (e_t + (n_1 - 1)p_1 + e_b)$

$e_b = \min\left(\dfrac{p_3}{2};\ 5d\right)$

$= \min\left(\dfrac{140}{2};\ 5 \times 20\right) = 70 \text{ mm}$

$e_t = \min(e_{1,t};\ 5d)$

Since the connection is not near the top of the column $e_{1,t}$ is not applicable.

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

$\therefore A_v = 12.5 \times (100 + (6-1) \times 70 + 70) = 6500 \text{ mm}^2$

$\therefore \dfrac{A_v f_{y,2}}{\sqrt{3}\gamma_{M0}} = \dfrac{6500 \times 355}{\sqrt{3} \times 1.0} \times 10^{-3} = 1332 \text{ kN}$

Shear resistance of net section:

$A_{v,net} = A_v - n_1 d_0 t_2 = 6500 - 6 \times 20 \times 12.5 = 5000 \text{ mm}^2$

$\therefore \dfrac{A_{v,net} f_{u,2}}{\sqrt{3}\gamma_{M2}} = \dfrac{5000 \times 470}{\sqrt{3} \times 1.1} \times 10^{-3} = 1233 \text{ kN}$

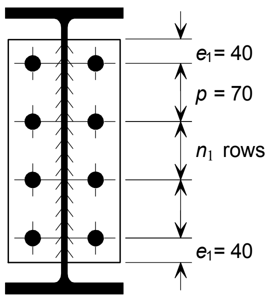
$V_{Rd,min}$ = min (1332; 1233) = 1233 kN

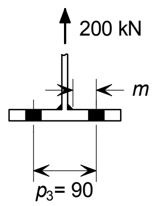
$\dfrac{V_{Ed}}{2} = 275 \text{ kN} < 1233 \text{ kN}$ $\therefore$ **O.K.**

Check 11: Tying resistance – Plate and bolts

Basic requirement: $F_{Ed} \le \min(F_{Rd,u,1};\ F_{Rd,u,2};\ F_{Rd,u,3})$

406 × 178 × 74 UKB, S275





$F_{Rd,u,1} = \dfrac{(8n - 2e_w)M_{pl,1,Rd,u}}{2mn - e_w(m+n)}$

$M_{pl,1,Rd,u} = \dfrac{0.25 \Sigma l_{eff,1} t_p^{\,2} f_u}{\gamma_{M,u}}$

$\Sigma l_{eff} = 2e_{1A} + (n_1 - 1)p_{1A}$

Title	*Example 3 – Partial depth end plate – Beam to hollow section column using Flowdrill*	*Sheet 6 of 9*

$e_{1A} = e_1$ but $\leq 0.5\left(p_3 - t_{w,b1} - 2a\sqrt{2}\right) + \dfrac{d_0}{2}$

$0.5 \times \left(90 - 9.5 - 2 \times 4.24 \times \sqrt{2}\right) + \dfrac{22}{2}$ = 45.3 mm

$\therefore e_{1A}$ = 40 mm

$p_{1A} = p_1$ but $\leq p_3 - t_{w,b1} - 2a\sqrt{2} + d_0$

$90 - 9.5 - 2 \times 4.24 \times \sqrt{2} + 22$ = 90.5 mm

$\therefore p_{1A}$ = 70 mm

Σl_{eff} = 2 × 40 + (4 – 1) × 70 = 290 mm

$M_{pl,1,Rd,u} = \dfrac{0.25 \times 290 \times 10^2 \times 410}{1.1} \times 10^{-6}$ = 2.7 kNm

$m = \dfrac{p_3 - t_{w,b1} - 2 \times 0.8 \times a\sqrt{2}}{2} = \dfrac{90 - 9.5 - 2 \times 0.8 \times 4.24 \times \sqrt{2}}{2}$ = 35.4 mm

Assuming no washer is used, d_w is taken as the width across points from Table G.66 — Table G.66

$e_w = \dfrac{d_w}{4} = \dfrac{33}{4}$ = 8.25 mm

$n = \min\left(e_2;\ \ 1.25m\right) = \min\left(30;\ \ 1.25 \times 35.4\right)$ = 30 mm

$\therefore F_{Rd,u,1} = \dfrac{\left(8 \times 30 - 2 \times 8.25\right) \times 2.7 \times 10^6}{2 \times 35.4 \times 30 - 8.25 \times \left(35.4 + 30\right)} \times 10^{-3}$ = 381 kN

$F_{Rd,u,2} = \dfrac{2M_{pl,2,Rd,u} + n\Sigma F_{t,Rd,u}}{m + n}$

$M_{pl,2,Rd,u} = M_{pl,1,Rd,u}$ = 2.7 kNm

$F_{t,Rd,u}$ = 125 kN (12.5 mm, S355) — Table G.59

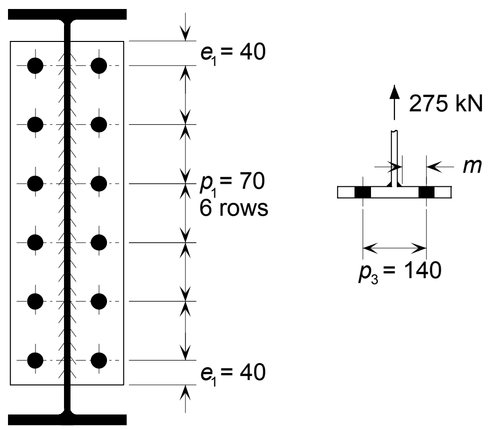
$\therefore F_{Rd,u,2} = \dfrac{2 \times 2.7 \times 10^6 + 30 \times 8 \times 125 \times 10^3}{35.4 + 30} \times 10^{-3}$ = 541 kN

$F_{Rd,u,3} = \Sigma F_{t,Rd,u}$ = 8 × 125 = 1000 kN

$\min\left(F_{Rd,u,1};\ \ F_{Rd,u,2};\ \ F_{Rd,u,3}\right) = \min\left(381;\ \ 541;\ \ 1000\right)$ = 381 kN

F_{Ed} = 200 kN < 381 kN — $\therefore$ **O.K.**

533 × 210 × 92 UKB, S275



Title	Example 3 – Partial depth end plate – Beam to hollow section column using Flowdrill	Sheet 7 of 9

$F_{\mathrm{Rd,u,1}} = \dfrac{(8n - 2e_{\mathrm{w}})M_{\mathrm{pl,1,Rd,u}}}{2mn - e_{\mathrm{w}}(m+n)}$

$\Sigma l_{\mathrm{eff}} = 2e_{\mathrm{1A}} + (n_1 - 1)p_{\mathrm{1A}}$

$e_{\mathrm{1A}} = e_1$ but $\leq 0.5\left(p_3 - t_{\mathrm{w,b1}} - 2a\sqrt{2}\right) + \dfrac{d_0}{2}$

$0.5 \times \left(140 - 10.1 - 2 \times 5.6 \times \sqrt{2}\right) + \dfrac{22}{2} = 68$ mm

$\therefore e_{\mathrm{1A}} = 40$ mm

$p_{\mathrm{1A}} = p_1$ but $\leq p_3 - t_{\mathrm{w,b1}} - 2a\sqrt{2} + d_0$

$140 - 10.1 - 2 \times 5.6 \times \sqrt{2} + 22 = 136$ mm

$\therefore p_{\mathrm{1A}} = 70$ mm

$\Sigma l_{\mathrm{eff}} = 2 \times 40 + (6 - 1) \times 70 = 430$ mm

$M_{\mathrm{pl,1,Rd,u}} = \dfrac{0.25 \Sigma l_{\mathrm{eff,1}} t_{\mathrm{p}}^{2} f_{\mathrm{u}}}{\gamma_{\mathrm{M,u}}} = \dfrac{0.25 \times 430 \times 12^2 \times 410}{1.1} \times 10^{-6} = 5.77$ kNm

$m = \dfrac{p_3 - t_{\mathrm{w,b1}} - 2 \times 0.8 \times a\sqrt{2}}{2} = \dfrac{140 - 10.1 - 2 \times 0.8 \times 4.2 \times \sqrt{2}}{2} = 60$ mm

Assuming no washer is used, d_{w} is taken as the width across points from Table G.66 — Table G.66

$e_{\mathrm{w}} = \dfrac{d_{\mathrm{w}}}{4} = \dfrac{33}{4} = 8.25$ mm

$n = \min\left(e_2;\ \ 1.25m\right) = \min\left(30;\ \ 1.25 \times 59\right) = 30$ mm

$\therefore F_{\mathrm{Rd,u,1}} = \dfrac{(8 \times 30 - 2 \times 8.25)\, 5.77 \times 10^6}{2 \times 60 \times 30 - 8.25(60 + 30)} \times 10^{-3} = 451$ kN

$F_{\mathrm{Rd,u,2}} = \dfrac{2M_{\mathrm{pl,2,Rd,u}} + n\Sigma F_{\mathrm{t,Rd,u}}}{m + n}$

$M_{\mathrm{pl,2,Rd,u}} = M_{\mathrm{pl,1,Rd,u}} = 5.77$ kNm

$F_{\mathrm{t,Rd,u}} = 125$ kN (12.5 mm, S355) — Table G.59

$\therefore F_{\mathrm{Rd,u,2}} = \dfrac{2 \times 5.77 \times 10^6 + 30 \times 12 \times 125 \times 10^3}{60 + 30} \times 10^{-3} = 628$ kN

$F_{\mathrm{Rd,u,3}} = \Sigma F_{\mathrm{t,Rd,u}} = 12 \times 125 = 1500$ kN

min ($F_{\mathrm{Rd,u,1}}$; $F_{\mathrm{Rd,u,2}}$; $F_{\mathrm{Rd,u,3}}$) = min (451; 628; 1500) = 451 kN

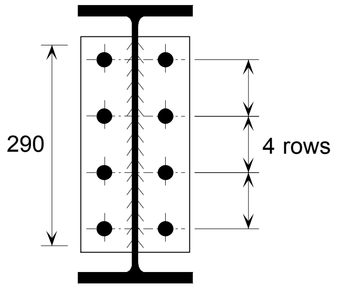
$F_{\mathrm{Ed}} = 275$ kN $<$ 451 kN — **∴ O.K.**

Check 12: Tying resistance – Supported beam web

Resistance of the beam web

Basic requirement: $F_{\mathrm{Ed}} \leq F_{\mathrm{Rd}}$

406 × 178 × 74 UKB, S275

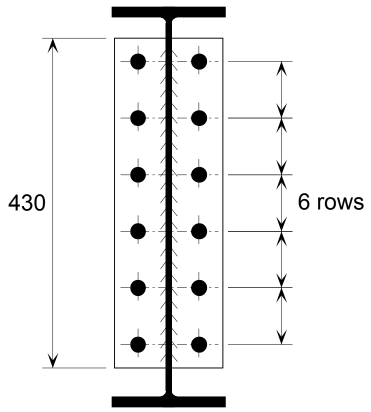


Title	Example 3 – Partial depth end plate – Beam to hollow section column using Flowdrill	Sheet 8 of 9

$F_{Rd} = \dfrac{t_{w,b1} h_p f_{u,b1}}{\gamma_{M,u}} = \dfrac{9.5 \times 290 \times 410}{1.1} \times 10^{-3} = 1027$ kN

$F_{Ed} = 200$ kN $<$ 1027 kN ∴ **O.K.**

533 × 210 × 92 UKB, S275



$F_{Rd} = \dfrac{t_{w,b1} h_p f_{u,b1}}{\gamma_{M,u}} = \dfrac{10.1 \times 430 \times 410}{1.1} \times 10^{-3} = 1619$ kN ∴ **O.K.**

$F_{Ed} = 275$ kN $<$ 1619 kN

Check 13: Tying resistance – Welds

Basic requirement: $a \geq 0.40\, t_{w,b1}$

406 × 178 × 74 UKB, S275

Throat thickness:

$a = \dfrac{6}{\sqrt{2}} = 4.24$ mm

$0.40\, t_{w,b1} = 0.40 \times 9.5 = 3.8$ mm

4.24 mm $\geq$ 3.8 mm ∴ **O.K.**

533 × 210 × 92 UKB, S275

Throat thickness:

$a = \dfrac{6}{\sqrt{2}} = 4.24$ mm

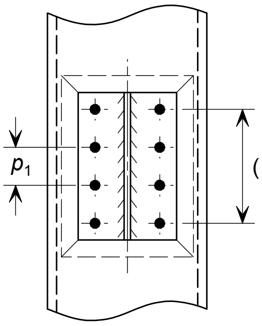
$0.40\, t_{w,b1} = 0.40 \times 10.1 = 4.04$ mm

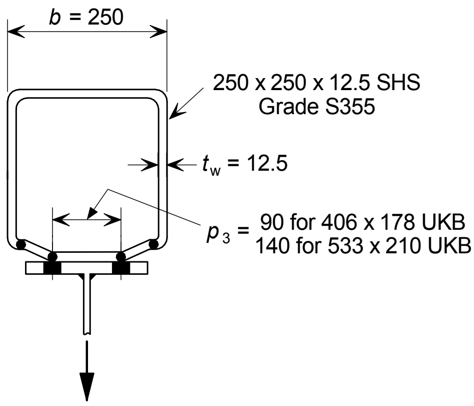
4.24 mm $\geq$ 4.04 mm ∴ **O.K.**

Check 15: Tying resistance – supporting column wall

Basic requirement: $F_{Ed} \leq F_{Rd,u}$

Title	*Example 3 – Partial depth end plate – Beam to hollow section column using Flowdrill*	Sheet 9 of 9





$$F_{Rd} = \frac{8M_{pl,Rd,u}}{(1-\beta_1)}\left[\eta_1 + 1.5(1-\beta_1)^{0.5}(1-\gamma_1)^{0.5}\right]$$

$$M_{pl,Rd,u} = \frac{f_{u,c}t_2^{\,2}}{4\gamma_{M,u}}$$

406 × 178 × 74 UKB, S275

$$M_{pl,Rd,u} = \frac{470\times 12.5^2}{4\times 1.1}\times 10^{-3} = 16.7 \text{ kNm/mm}$$

$$\beta_1 = \frac{p_3}{b-3t_2} = \frac{90}{250-3\times 12.5} = 0.424$$

$$\gamma_1 = \frac{d_0}{b-3t_2} = \frac{20}{250-3\times 12.5} = 0.094$$

$$\eta_1 = \frac{(n_1-1)p_1 - \frac{n_1}{2}d_0}{b-3t_2} = \frac{(4-1)\times 70 - \frac{4}{2}\times 20}{250-3\times 12.5} = 0.8$$

$$F_{Rd,u} = \frac{8\times 16.7}{(1-0.424)}\left(0.8 + 1.5\times(1-0.424)^{0.5}(1-0.094)^{0.5}\right) = 437 \text{ kN}$$

F_{Ed} = 200 kN < 437 kN ∴ **O.K.**

533 × 210 × 92 UKB, S275

$$M_{pl,Rd,u} = \frac{470\times 12.5^2}{4\times 1.1}\times 10^{-3} = 16.7 \text{ kNm/mm}$$

$$\beta_1 = \frac{p_3}{b-3t_2} = \frac{140}{250-3\times 12.5} = 0.66$$

$$\gamma_1 = \frac{d_0}{b-3t_2} = \frac{20}{250-3\times 12.5} = 0.094$$

$$\eta_1 = \frac{(n_1-1)p_1 - \frac{n}{2}d_0}{b-3t_2} = \frac{(6-1)\times 70 - \frac{6}{2}\times 20}{250-3\times 12.5} = 1.36$$

$$F_{Rd} = \frac{8\times 16.7}{(1-0.66)}\left(1.36 + 1.5(1-0.66)^{0.5}(1-0.094)^{0.5}\right) = 862 \text{ kN}$$

F_{Ed} = 275 kN < 862 kN ∴ **O.K.**

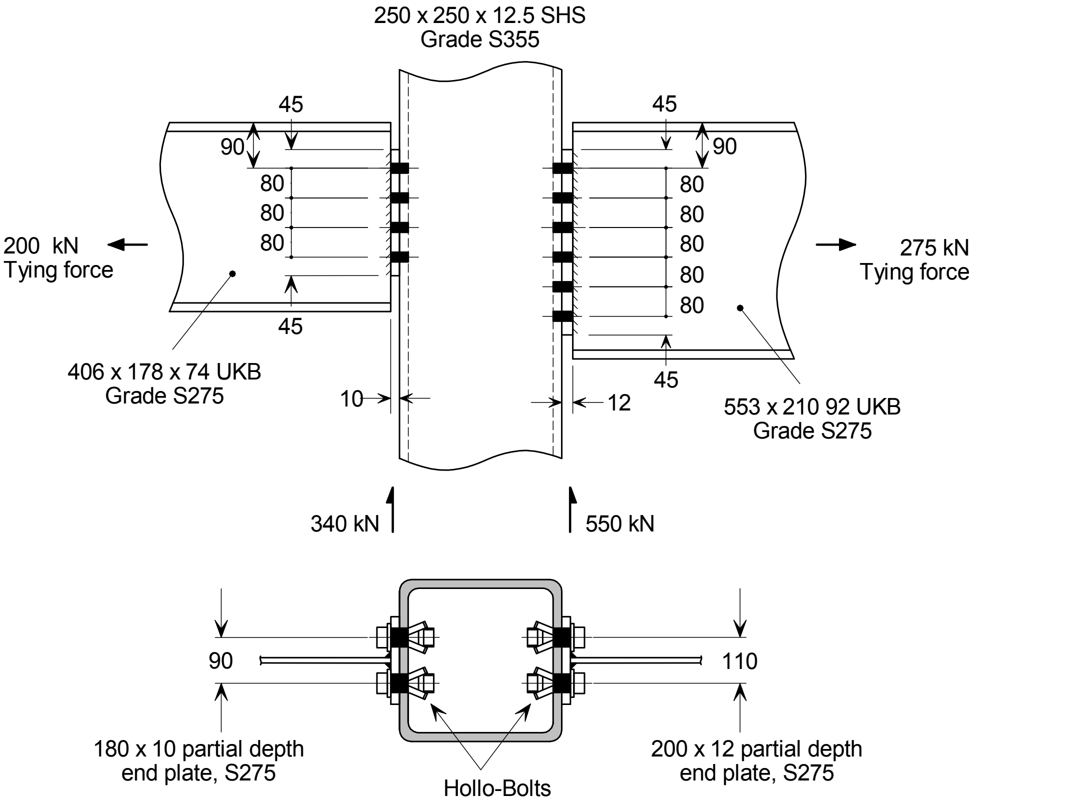
SCI Steel Knowledge **CALCULATION SHEET** BCSA	Job: *Joints in Steel Construction – Simple Joints*		Sheet 1 of 13
	Title: *Example 4 – Partial depth end plate – Beam to hollow section column using Hollo-Bolts*		
	Client: *Connections Group*		
	Calcs by *CZT*	Checked by *ENM*	Date *Sept 2011*

DESIGN EXAMPLE 4

Check the following beam to hollow section column joint for the design forces shown using Hollo-Bolt connectors to the column.

In this example the tie force is less than the shear force.

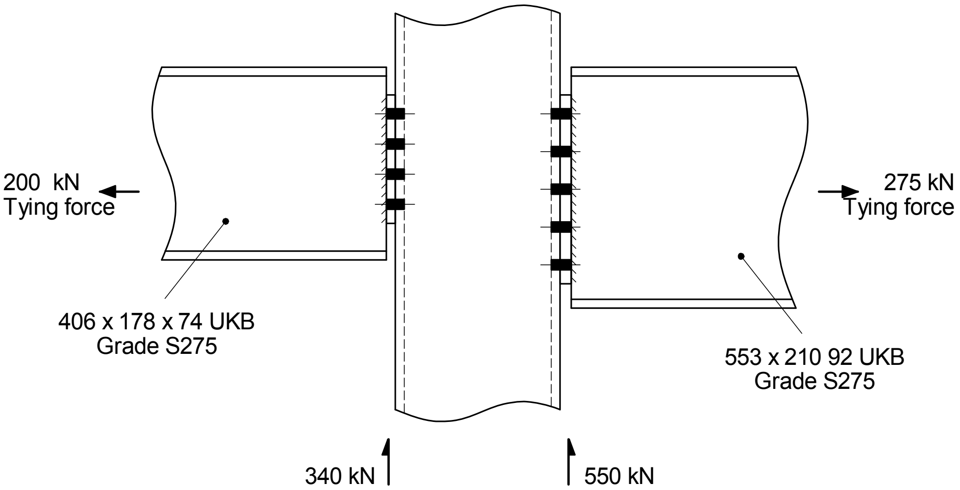
The connections should be checked independently for shear forces and tying forces and not for both forces acting at the same time.



Design Information:

Bolts:	M20 8.8, Hollo-Bolts
Welds:	6 mm leg length fillet welds
Column:	S355
Beams:	S275
End plates:	S275

CONNECTION DESIGN USING RESISTANCE TABLES



406 × 178 × 74 UKB, S275
End plate, 180 × 10
Welds 6 mm fillet
Bolts M20 8.8
Bolts at 90 cross centres

4 rows of bolts

From Resistance Table G.6
Connection shear resistance
= 427 kN > 340 kN

Minimum support thickness in S355
= 3.0 mm < 12.5 mm

Connection tying resistance
= 458 kN > 200 kN

The beam side of the connection is adequate

533 × 210 × 92 UKB, S275
End plate, 200 × 12
Welds 6 mm fillet
Bolts M20 8.8
Bolts at 110 cross centres

5 rows of bolts

From Resistance Table G.6
Connection shear resistance
= 592 kN > 550 kN

Minimum support thickness in S355
= 3.3 mm < 12.5 mm

Connection tying resistance
= 613 kN > 275 kN

The beam side of the connection is adequate

Table G.6

Note:

(1) The tying resistance of the connection given in the tables in the yellow pages is the least of the values obtained from Checks 11, 12 & 13.

(2) The hollow section wall must also be checked as shown in Check 15.

Title	*Example 4 – Partial depth end plate – Beam to hollow section column using Hollo-Bolts*	*Sheet 3 of 13*

SUMMARY OF FULL DESIGN CHECKS FOR EXAMPLE 4

Notes:

(1) Checks 1 to 4, where applicable, are generally as shown in Example 1. Check 4 is included because the length of the end plate varies from Example 1.

(2) Tying forces are ignored when checking the shear resistance and shear is ignored calculating the tying resistance.

Sheet No.	**CHECK**		**406 UKB (S275)**		**533 UKB (S275)**		**SHS Column, S355**			
							406 UKB Side		**533 UKB Side**	
			Resist	**Design force**	**Resist**	**Design force**	**Resist**	**Design force**	**Resist**	**Design force**
4	***Check 1*** Recommended detailing practice		All recommendations adopted							
4	***Check 2*** Supported beam Welds (kN)		Full strength welds adopted – Not critical				Not applicable			
4	***Check 3***		Not applicable							
4	***Check 4*** Supported beam Web in shear	Shear resistance (kN)	448	340	592	550	Not applicable			
4	***Checks 5, 6, 7***		Not applicable							
4	***Check 8*** Connection Bolt group	Bolt group (kN)	429	340	643	550	Not applicable			
6	***Check 9*** Connection End plate in shear	Shear resistance (kN)	721	340	1037	550	Not applicable			
8	***Check 10*** Supporting column Shear and bearing	Shear and Bearing resistance (kN)	Not applicable				756	170	925	275
10	***Check 11*** Tying resistance Plates and bolts	Tension (kN)	487	200	631	275	Not applicable			
12	***Check 12*** Structural Integrity Supported beam web	Tension (kN)	1169	200	1543	275	Not applicable			
12	***Check 13*** Structural Integrity Welds	Tension (kN)	Full strength welds adopted – not critical				Not applicable			
	Check 14		Not applicable							
13	***Check 15*** Structural Integrity Supporting column wall	Tension (kN)	Not applicable				427	200	566	275

Title	*Example 4 – Partial depth end plate – Beam to hollow section column using Hollo-Bolts*	*Sheet 4 of 13*

CONNECTION DESIGN FOLLOWING THE DESIGN PROCEDURES

Check 4: Supported beam – Web in shear

Shear resistance of beam web at the end plate

Basic requirement: $V_{Ed} \leq V_{c,Rd}$

Shear resistance of beam web at the end plate: $V_{c,Rd} = \dfrac{A_v f_{y,b1} / \sqrt{3}}{\gamma_{M0}}$

406 × 178 × 74 UKB, S275

Shear area of beam web at the connection:

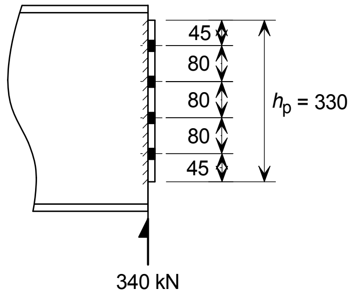
h_p = 45 + 3 × 80 + 45 = 330 mm

A_v = 0.9 × 330 × 9.5 = 2822 mm^2

t_f = 16.0 mm, hence $f_{y,b1}$ = 275 N/mm^2

γ_{M0} = 1.0

Shear resistance of beam web at the connection:



$\therefore\ V_{pl,Rd} = \dfrac{2822 \times 275 / \sqrt{3}}{1.0} \times 10^{-3}$ = 448 kN

V_{Ed} = 340 kN ≤ 448 kN — ∴ **O.K.**

533×210×92 UKB, S275

Shear area of beam web at the connection:

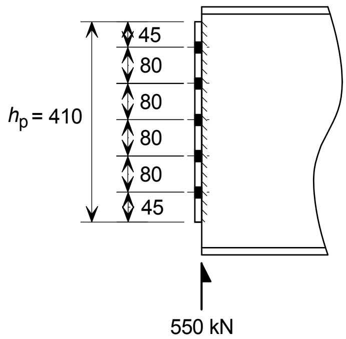
h_p = 45 + 4 × 80 + 45 = 410 mm

A_v = 0.9 × 410 × 10.1 = 3727 mm^2

t_f = 15.6 mm, hence $f_{y,b1}$ = 275 N/mm^2

γ_{M0} = 1.0

Shear resistance of beam web at the connection:



$\therefore\ V_{pl,Rd} = \dfrac{3727 \times 275 / \sqrt{3}}{1.0} \times 10^{-3}$ = 592 kN

V_{Ed} = 550 kN ≤ 592 kN — ∴ **O.K.**

Check 8: Connection – Bolt group

Basic requirement: $V_{Ed} \leq F_{Rd}$

The resistance of the bolt group, F_{Rd}, is as follows:

If $F_{b,Rd} \leq 0.8F_{v,Rd}$ then $F_{Rd} = \Sigma F_{b,Rd}$

if $F_{b,Rd} > 0.8F_{v,Rd}$ then $F_{Rd} = 0.8nF_{v,Rd}$

Shear resistance of a single bolt. For M20 Hollo-Bolts: — Table G.60

$F_{v,Rd}$ = 169 kN

Bearing resistance of a single bolt:

$F_{b,Rd} = \min(F_{b,Rd,p};\ F_{b,Rd,2})$

$$F_{b,Rd,p} = \frac{k_{1,p}\alpha_{b,p} f_{u,p} d t_p}{\gamma_{M2}}$$

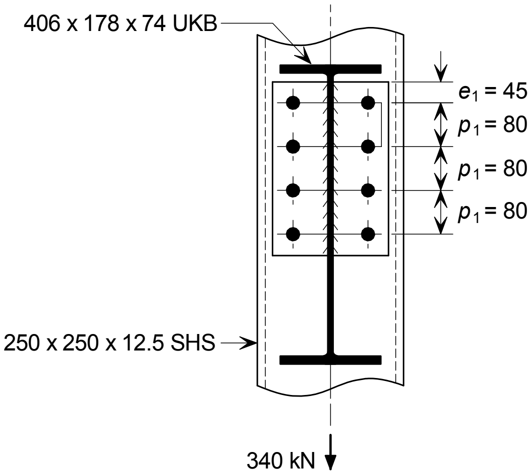
$$F_{b,Rd,2} = \frac{k_{1,2}\alpha_{b,2} f_{u,2} d t_2}{\gamma_{M2}}$$

Bearing on the supporting column will be verified in Check 10.

$\therefore F_{b,Rd} = F_{b,Rd,p}$

Title	Example 4 – Partial depth end plate – Beam to hollow section column using Hollo-Bolts	Sheet 5 of 13

406 × 178 × 74 UKB, S275



Bearing on the end plate:

$$k_{1,p} = \min\left(2.8\frac{e_2}{d_0} - 1.7;\quad 1.4\frac{p_2}{d_0} - 1.7;\quad 2.5\right)$$

$$= \min\left(2.8\times\frac{45}{35} - 1.7;\quad 1.4\times\frac{90}{35} - 1.7;\quad 2.5\right) = \min\left(1.9;\ 1.9;\quad 2.5\right) = 1.9$$

d_0 from Table G.69

$$\alpha_{b,p} = \min\left(\frac{e_1}{3d_0};\ \frac{p_1}{3d_0} - \frac{1}{4};\ \frac{f_{ub}}{f_{u,p}};\ 1.0\right) = \min\left(\frac{45}{3\times35};\ \frac{80}{3\times35} - 0.25;\ \frac{800}{410};\ 1.0\right)$$

$$= \min\left(0.43;\quad 0.51;\quad 1.95;\quad 1.0\right) = 0.43$$

$$F_{b,Rd,p} = \frac{1.9\times0.43\times410\times20\times10}{1.25}\times10^{-3} = 53.6 \text{ kN}$$

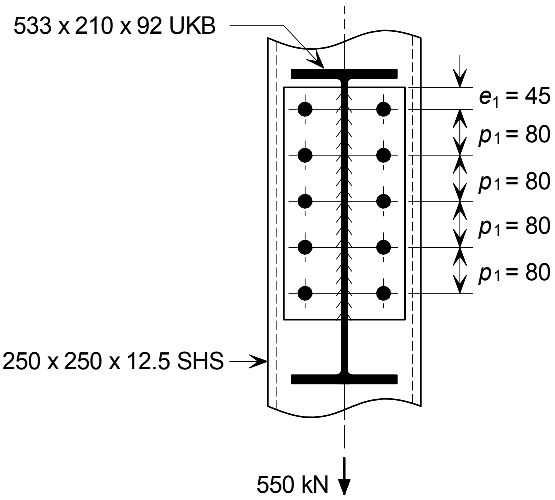
$0.8F_{v,Rd}$ = 0.8 × 169 = 135.2 kN

$\therefore F_{b,Rd}$ = 53.6 kN < 135.2 kN

$\therefore F_{Rd}$ = $\Sigma F_{b,Rd}$ = 8 × 53.6 = 429 kN

$\therefore V_{Ed}$ = 340 kN < 429 kN $\quad\therefore$ **O.K.**

533 × 210 × 92 UKB, S275



Title	*Example 4 – Partial depth end plate – Beam to hollow section column using Hollo-Bolts*	*Sheet 6 of 13*

Bearing on the end plate:

$k_{1,p} = \min\left(2.8\frac{e_2}{d_0} - 1.7;\quad 1.4\frac{p_2}{d_0} - 1.7;\quad 2.5\right)$

$= \min\left(2.8\times\frac{45}{35} - 1.7;\ 1.4\times\frac{110}{35} - 1.7;\ 2.5\right) = \min\left(1.9;\ 2.7;\ 2.5\right) = 1.9$

$\alpha_{b,p} = \min\left(\frac{e_1}{3d_0};\ \frac{p_1}{3d_0} - \frac{1}{4};\ \frac{f_{ub}}{f_{u,p}};\ 1.0\right) = \min\left(\frac{45}{3\times35};\ \frac{80}{3\times35} - 0.25;\ \frac{800}{410};\ 1.0\right)$

$= \min\left(0.43;\ 0.51;\ 1.95;\ 1.0\right) = 0.43$

$F_{b,Rd,p} = \frac{1.9\times0.43\times410\times20\times12}{1.25}\times10^{-3} = 64.3\text{ kN}$

$\therefore\ F_{b,Rd} = 64.3\text{ kN} < 135.2\text{ kN}$

$\therefore\ F_{Rd} = \Sigma F_{b,Rd} = 10\times64.3 = 643\text{ kN}$

$\therefore\ V_{Ed} = 550\text{ kN} < 643\text{ kN}$ **∴ O.K.**

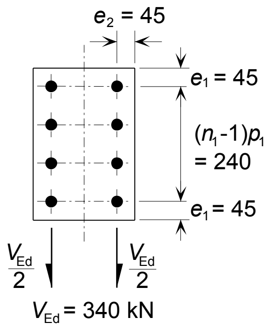
Check 9: Connection – End plate in shear

Basic requirement: $V_{Ed} \le V_{Rd,min}$

$V_{Rd,min} = \min(V_{Rd,g};\ V_{Rd,n};\ V_{Rd,b})$

406 × 178 × 74 UKB, S275

Shear resistance of gross section



$V_{Rd,g} = \frac{2h_p t_p}{1.27}\frac{f_{y,p}}{\sqrt{3}\gamma_{M0}}$

$V_{Rd,g} = \frac{2\times330\times10}{1.27}\times\frac{275}{\sqrt{3}\times1.0}10^{-3} = 825\text{ kN}$

Shear resistance of net section:

$V_{Rd,n} = 2A_{v,net}\frac{f_{u,p}}{\sqrt{3}\gamma_{M2}}$

Net area:

$A_{v,net} = A - nd_0t_p$

$A_{v,net} = 330\times10 - 4\times35\times10 = 1900\text{ mm}^2$

$\therefore\ V_{Rd,n} = 2\times1900\times\frac{410}{\sqrt{3}\times1.1}\times10^{-3} = 818\text{ kN}$

Block tearing resistance (block shear):

$1.36p_3 = 1.36\times90 = 122\text{ mm}$

$h_p = 330\text{ mm} > 122\text{ mm}$, therefore

$V_{Rd,b} = 2\times\left(\frac{f_{u,p}A_{nt}}{\gamma_{M2}} + \frac{f_{y,p}A_{nv}}{\sqrt{3}\gamma_{M0}}\right)$

Net area subject to tension:

$A_{nt} = t_p\left(e_2 - \frac{d_0}{2}\right) = 10\times\left(45 - \frac{35}{2}\right) = 275\text{ mm}^2$

Net area subject to shear:

$A_{nv} = t_p\left(h_p - e_1 - (n_1 - 0.5)d_0\right) = 10\times\left(330 - 45 - (4 - 0.5)\times35\right) = 1625\text{ mm}^2$

$\therefore\ V_{Rd,b} = 2\times\left(\frac{410\times275}{1.1} + \frac{275\times1625}{\sqrt{3}\times1.0}\right)\times10^{-3} = 721\text{ kN}$

Title	*Example 4 – Partial depth end plate – Beam to hollow section column using Hollo-Bolts*	*Sheet 7 of 13*

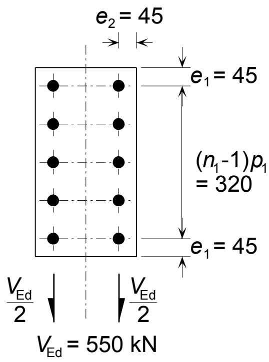
$V_{Rd,min}$ = $\min(825;\ 818;\ 721)$ = 721 kN

$\therefore\ V_{Ed}$ = 340 kN < 721 kN — **∴ O.K.**

533 × 210 × 92 UKB, S275

Shear resistance of gross section

$$V_{Rd,g} = \frac{2h_p t_p}{1.27}\frac{f_{y,p}}{\sqrt{3}\gamma_{M0}}$$



$$V_{Rd,g} = \frac{2\times410\times12}{1.27}\times\frac{275}{\sqrt{3}\times1.0}10^{-3} = 1230\text{ kN}$$

Shear resistance of net section:

$$V_{Rd,n} = 2A_{v,net}\frac{f_{u,p}}{\sqrt{3}\gamma_{M2}}$$

Net area:

$A_{v,net}$ = $A - nd_0t_p$

$A_{v,net}$ = 410 × 12 – 5 × 35 × 12 = 2820 mm²

$$\therefore\ V_{Rd,n} = 2\times2820\times\frac{410}{\sqrt{3}\times1.1}\times10^{-3} = 1214\text{ kN}$$

Block tearing resistance (Block shear):

$1.36p_3$ = 1.36 × 110 = 150 mm

h_p = 410 mm > 150 mm, therefore

$$V_{Rd,b} = 2\times\left(\frac{f_{u,p}A_{nt}}{\gamma_{M2}}+\frac{f_{y,p}A_{nv}}{\sqrt{3}\gamma_{M0}}\right)$$

Net area subject to tension:

$$A_{nt} = t_p\left(e_2-\frac{d_0}{2}\right) = 12\times\left(45-\frac{35}{2}\right) = 330\text{ mm}^2$$

Net area subject to shear:

$$A_{nv} = t_p\left(h_p-e_1-(n-0.5)d_0\right) = 12\times\left(410-45-(5-0.5)\times35\right) = 2490\text{ mm}^2$$

$$\therefore\ V_{Rd,b} = 2\times\left(\frac{410\times330}{1.1}+\frac{275\times2490}{\sqrt{3}\times1.0}\right)\times10^{-3} = 1037\text{ kN}$$

$V_{Rd,min}$ = min(1230; 1214; 1037) = 1037 kN

$\therefore\ V_{Ed}$ = 550 kN < 1037 kN — **∴ O.K.**

Additional requirement: End plate in bending

For 406 × 178 × 74 UKB;

h_p = 330 mm

$1.36\ p_3$ = 1.36 × 90 = 122 mm, < 330 mm, so no additional check required

For 533 × 210 × 92 UKB;

h_p = 410 mm

$1.36\ p_3$ = 1.36 × 110 = 150 mm, < 410 mm, so no additional check required

Title	*Example 4 – Partial depth end plate – Beam to hollow section column using Hollo-Bolts*	*Sheet 8 of 13*

Check 10: Supporting column – Shear and bearing

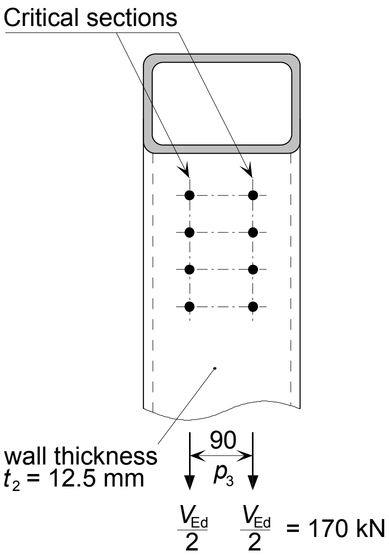
Local shear and bearing resistance of the hollow section column wall

(i) Shear:

Basic requirement: $\frac{V_{Ed}}{2} \leq V_{Rd,min}$

$$V_{Rd,min} = \min\left(\frac{A_v f_{y,2}}{\sqrt{3}\gamma_{M0}};\quad \frac{A_{v,net} f_{u,2}}{\sqrt{3}\gamma_{M2}}\right)$$

406 × 178 × 74 UKB, S275



Shear area of gross section: $A_v = t_2\,(e_t + (n_1 - 1)p_1 + e_b)$

$$e_b = \min\left(\frac{p_3}{2};\quad 5d\right) = \min\left(\frac{90}{2};\quad 5 \times 20\right) = 45 \text{ mm}$$

$$e_t = \min\left(e_{1,t};\quad 5d\right)$$

Since the connection is not near the top of the column $e_{1,t}$ is not applicable.

$e_t = 5 \times 20 = 100$ mm

$\therefore A_v = 12.5 \times (100 + (4 - 1) \times 80 + 45) = 4813 \text{ mm}^2$

$t_2 < 16$ mm, hence $f_{y,2} = 355 \text{ N/mm}^2$

Therefore the resistance of the gross section is:

$$\therefore \frac{A_v f_{y,2}}{\sqrt{3}\gamma_{M0}} = \frac{4813 \times 355}{\sqrt{3} \times 1.0} \times 10^{-3} = 986 \text{ kN}$$

Shear area of net section: $A_{v,net} = A_v - n_1 d_0 t_2$

$\therefore A_{v,net} = 4813 - 4 \times 35 \times 12.5 = 3063 \text{ mm}^2$

Therefore the resistance of the net section is:

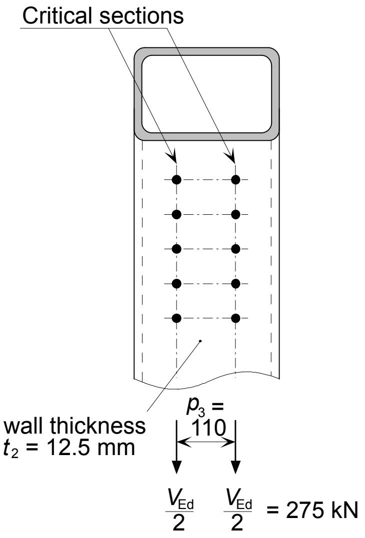
$$\therefore \frac{A_{v,net} f_{u,2}}{\sqrt{3}\gamma_{M2}} = \frac{3063 \times 470}{\sqrt{3} \times 1.1} \times 10^{-3} = 756 \text{ kN}$$

$\therefore V_{Rd,min} = \min\left(986;\quad 756\right) = 756$ kN

$\frac{V_{Ed}}{2} = 170 \text{ kN} < 756 \text{ kN}$ **∴ O.K.**

Title	*Example 4 – Partial depth end plate – Beam to hollow section column using Hollo-Bolts*	Sheet 9 of 13

533 × 210 × 92 UKB, S275



Shear area of gross section: $A_v = t_2\,(e_t + (n_1 - 1)p_1 + e_b)$

$e_b \quad = \min\left(\frac{p_3}{2};\quad 5d\right) = \min\left(\frac{110}{2};\quad 5\times 20\right) = 55 \text{ mm}$

$e_t \quad = \min\left(e_{1,t};\quad 5d\right)$

Since the connection is not near the top of column $e_{1,t}$ is not applicable.

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

$\therefore A_v \quad = 12.5 \times (100 + (5 - 1) \times 80 + 55) = 5938 \text{ mm}^2$

Therefore the resistance of the gross section is:

$$\therefore \frac{A_v f_{y,2}}{\sqrt{3}\gamma_{M0}} = \frac{5938 \times 355}{\sqrt{3} \times 1.0} \times 10^{-3} = 1217 \text{ kN}$$

Shear area of net section: $A_{v,net} = A_v - n_1 d_0 t_2$

$\therefore A_{v,net} \quad = 5938 - 5 \times 35 \times 12.5 = 3751 \text{ mm}^2$

Therefore the resistance of the net section is:

$$\therefore \frac{A_{v,net} f_{u,2}}{\sqrt{3}\gamma_{M2}} = \frac{3751 \times 470}{\sqrt{3} \times 1.1} \times 10^{-3} = 925 \text{ kN}$$

$\therefore V_{Rd,min} = \min\left(1217;\quad 925\right) = 925 \text{ kN}$

$\frac{V_{Ed}}{2} \quad = 275 \text{ kN} < 925 \text{ kN}$ $\qquad$ $\therefore$ **O.K.**

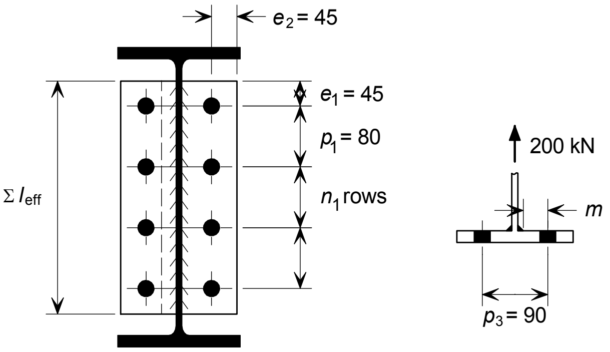
Title	Example 4 – Partial depth end plate – Beam to hollow section column using Hollo-Bolts	Sheet 10 of 13

Check 11: Tying resistance – Plate and bolts

Resistance of end plate

Basic requirement: $F_{Ed} \leq \min(F_{Rd,u,1};\ F_{Rd,u,2};\ F_{Rdu,3})$

406 × 178 × 74 UKB, S275



Mode 1:

$$F_{Rd,u,1} = \frac{(8n - 2e_w)M_{pl,1,Rd,u}}{2mn - e_w(m+n)}$$

$$M_{pl,1,Rd,u} = \frac{0.25\Sigma l_{eff}\, t_p^{\,2} f_{u,b1}}{\gamma_{M,u}} = \frac{0.25 \times 330 \times 10^2 \times 410}{1.1} \times 10^{-6} = 3.1 \text{ kNm}$$

$$m = \frac{p_3 - t_{w,b1} - 2 \times 0.8 \times a\sqrt{2}}{2} = \frac{90 - 9.5 - 2 \times 0.8 \times 6}{2} = 35.5 \text{ mm}$$

$$e_w = \frac{d_w}{4} = \frac{46}{4} = 11.5 \text{ mm}$$

d_w from Table G.69

$$n = \min(e_2;\ 1.25m) = \min(45;\ 1.25 \times 35.5) = 44 \text{ mm}$$

$$\therefore F_{Rd,u,1} = \frac{(8 \times 44 - 2 \times 11.5) \times 3.1 \times 10^6}{2 \times 34 \times 44 - 11.5 \times (34 + 44)} \times 10^{-3} = 487 \text{ kNm}$$

Mode 2:

$$F_{Rd,u,2} = \frac{2M_{pl,2,Rd,u} + n\Sigma F_{t,Rd,u}}{m+n}$$

$M_{pl,2,Rd,u} = M_{pl,1,Rd,u} = 3.1$ kNm

$F_{t,Rd,u} = 113$ kN — Table G.60

$$F_{Rd,u,2} = \frac{2 \times 3.1 \times 10^6 + 44 \times 8 \times 113 \times 10^3}{34 + 44} \times 10^{-3} = 589 \text{ kN}$$

Mode 3:

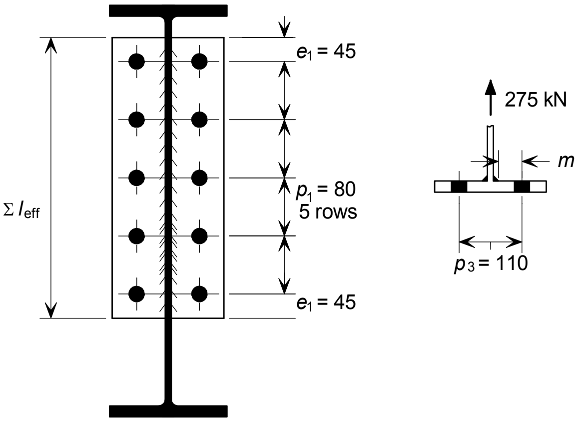
$F_{Rd,u,3} = \Sigma F_{t,Rd,u} = 8 \times 113 = 904$ kN

$\min(F_{Rd,u,1},\ F_{Rd,u,2},\ F_{Rd,u,3}) = \min(487,\ 589;\ 904) = 487$ kN

$F_{Ed} = 200 \text{ kN} < 487 \text{ kN}$ — $\therefore$ **O.K.**

Title	Example 4 – Partial depth end plate – Beam to hollow section column using Hollo-Bolts	Sheet 11 of 13

533 × 210 × 92 UKB, S275



Mode 1:

$$F_{Rd,u,1} = \frac{(8n - 2e_w)M_{pl,1,Rd,u}}{2mn - e_w(m+n)}$$

$$M_{pl,1,Rd,u} = \frac{0.25\Sigma l_{eff}\, t_p^{\,2} f_{u,b1}}{\gamma_{M,u}} = \frac{0.25 \times 410 \times 12^2 \times 410}{1.1} \times 10^{-6} = 5.50 \text{ kNm}$$

$$m = \frac{p_3 - t_{w,b1} - 2\times 0.8 \times a\sqrt{2}}{2} = \frac{110 - 10.1 - 2\times 0.8 \times 5.6 \times \sqrt{2}}{2} = 44 \text{ mm}$$

$$e_w = \frac{d_w}{4} = \frac{46}{4} = 11.5 \text{ mm}$$

Table G.69

$$n = \min(e_2;\ \ 1.25m) = \min(45;\ \ 1.25\times 43.6) = 45 \text{ mm}$$

$$\therefore\ F_{Rd,u,1} = \frac{(8\times 45 - 2\times 11.5)\times 5.50\times 10^6}{2\times 44\times 45 - 11.5\times(44+45)}\times 10^{-3} = 631 \text{ kN}$$

Mode 2:

$$F_{Rd,u,2} = \frac{2M_{pl,2,Rd,u} + n\Sigma F_{t,Rd,u}}{m+n}$$

$M_{pl,2,Rd,u}$ = $M_{pl,1,Rd,u}$ = 5.50 kNm

$F_{t,Rd,u}$ = 113 kN

$$F_{Rd,u,2} = \frac{2\times 5.50\times 10^6 + 45\times 10\times 113\times 10^3}{44+45}\times 10^{-3} = 694 \text{ kN}$$

Mode 3:

$F_{Rd,u,3}$ = $\Sigma F_{t,Rd,u}$ = 10 × 113 = 1130 kN

$\min(F_{Rd,u,1},\ \ F_{Rd,u,2},\ \ F_{Rd,u,3}) = \min(631,\ \ 694;\ \ 1130)$ = 631 kN

F_{Ed} = 275 kN < 631 kN

∴ **O.K.**

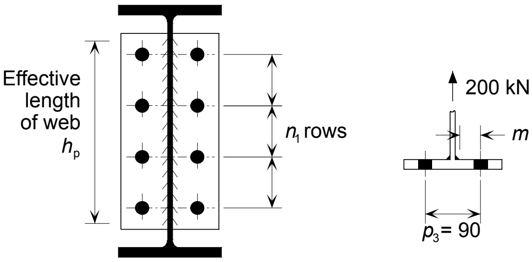
Title	*Example 4 – Partial depth end plate – Beam to hollow section column using Hollo-Bolts*	Sheet 12 of 13

Check 12: Tying resistance – Supported beam web

Resistance of the beam web

Basic requirement: $F_{Ed} \le F_{Rd}$

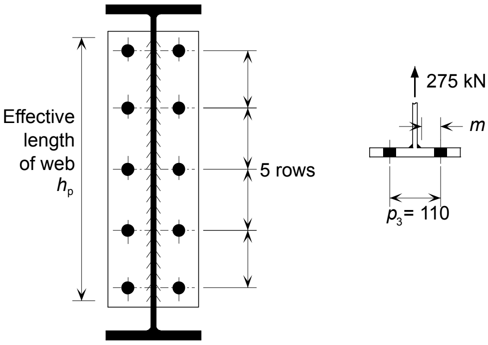
406 × 178 × 74 UKB, S275



$$F_{Rd} = \frac{t_{w,b1} h_p f_{u,b1}}{\gamma_{M,u}} = \frac{9.5 \times 330 \times 410}{1.1} \times 10^{-3} = 1169 \text{ kN}$$

F_{Ed} = 200 kN < 1169 kN **∴ O.K.**

533 × 210 × 92 UKB, S275



$$F_{Rd} = \frac{t_{w,b1} h_p f_{u,b1}}{\gamma_{M,u}} = \frac{10.1 \times 410 \times 410}{1.1} \times 10^{-3} = 1543 \text{ kN}$$

F_{Ed} = 275 kN < 1543 kN **∴ O.K.**

Check 13: Tying resistance – Welds

Basic requirement: $a \le 0.40\, t_{w,b1}$

406 × 178 × 74 UKB, S275

Throat thickness:

$$a = \frac{6}{\sqrt{2}} = 4.24 \text{ mm}$$

$0.40\, t_{w,b1} = 0.40 \times 9.5 = 3.8$ mm

a = 4.24 mm ≥ 3.8 mm **∴ O.K.**

533 × 210 × 92 UKB, S275

Throat thickness:

$$a = \frac{6}{\sqrt{2}} = 4.24 \text{ mm}$$

$0.40\, t_{w,b1} = 0.40 \times 10.1 = 4.04$ mm

a = 4.24 mm ≥ 4.04 mm **∴ O.K.**

Title	*Example 4 – Partial depth end plate – Beam to hollow section column using Hollo-Bolts*	*Sheet 13 of 13*

Check 15: Tying resistance – Supporting column wall

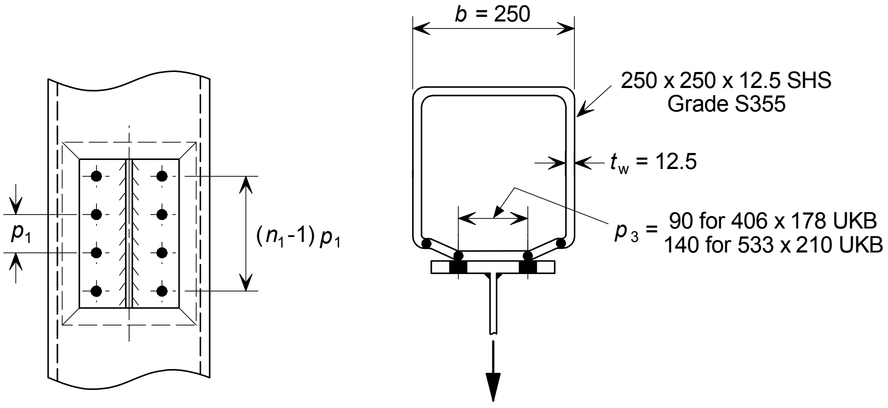
Resistance of hollow section wall

Basic requirement: $F_{Ed} \leq F_{Rd}$

$$F_{Rd} = \frac{8M_{pl,Rd,u}}{(1-\beta_1)}\left(\eta_1 + 1.5(1-\beta_1)^{0.5} \times (1-\gamma_1)^{0.5}\right)$$

$$M_{pl,Rd,u} = \frac{f_{u,c}\, t_2^{\,2}}{4\gamma_{M,u}}$$

406 × 178 × 74 UKB, S275



$$M_{pl,Rd,u} = \frac{470 \times 12.5^2}{4 \times 1.1} \times 10^{-3} = 16.7 \text{ kNm/mm}$$

$$\beta_1 = \frac{p_3}{b-3t_2} = \frac{90}{250-3\times 12.5} = 0.424$$

$$\gamma_1 = \frac{d_0}{b-3t_2} = \frac{35}{250-3\times 12.5} = 0.165$$

$$\eta_1 = \frac{(n_1-1)p_1 - \frac{n_1}{2}d_0}{b-3t_2} = \frac{(4-1)\times 80 - \frac{4}{2}\times 35}{250-3\times 12.5} = 0.8$$

$$F_{Rd,u} = \frac{8\times 16.7}{(1-0.424)} \times \left(0.8 + 1.5\times(1-0.424)^{0.5} \times (1-0.165)^{0.5}\right) = 427 \text{ kN}$$

F_{Ed} = 200 kN < 427 kN ∴ **O.K.**

533 × 210 × 92 UKB, S275

$$M_{pl,Rd,u} = \frac{470 \times 12.5^2}{4 \times 1.1} \times 10^{-3} = 16.7 \text{ kNm/mm}$$

$$\beta_1 = \frac{p_3}{b-3t_2} = \frac{110}{250-3\times 12.5} = 0.518$$

$$\gamma_1 = \frac{d_0}{b-3t_2} = \frac{35}{250-3\times 12.5} = 0.165$$

$$\eta_1 = \frac{(n_1-1)p_1 - \frac{n_1}{2}d_0}{b-3t_2} = \frac{(5-1)\times 80 - \frac{5}{2}\times 35}{250-3\times 12.5} = 1.09$$

$$F_{Rd,u} = \frac{8\times 16.7}{(1-0.518)} \times \left(1.09 + 1.5\times(1-0.518)^{0.5} \times (1-0.165)^{0.5}\right) = 566 \text{ kN}$$

F_{Ed} = 275 kN < 566 kN ∴ **O.K.**

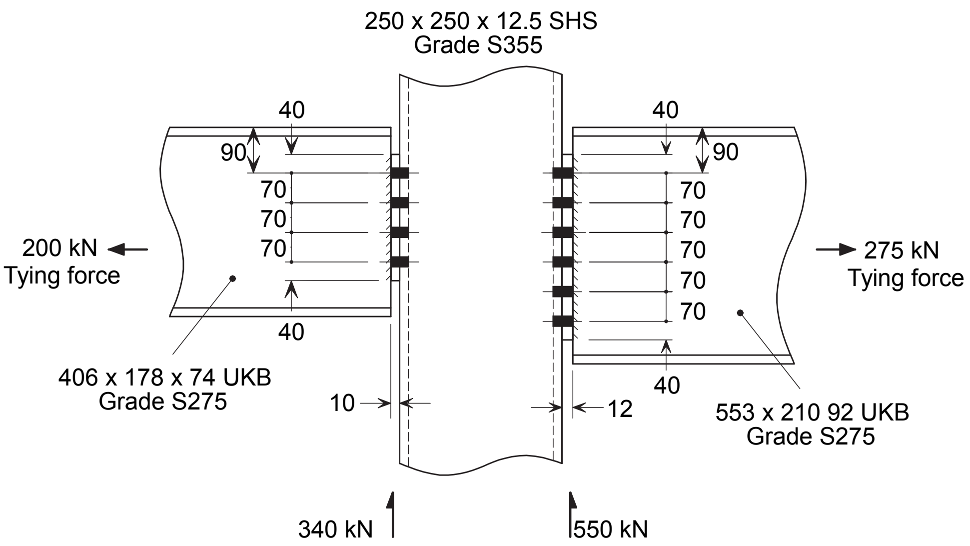
SCI Steel Knowledge **CALCULATION SHEET** BCSA	Job *Joints in Steel Construction – Simple Joints*		Sheet *1* of *11*
	Title *Example 5 – Partial depth end plate – Beam to hollow section column using Blind Bolts*		
	Client *Blind Bolts*		
	Calcs by *CZK*	Checked by *DGB*	Date *December 2013*

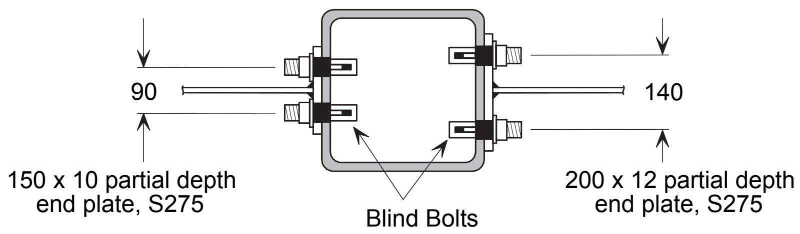
DESIGN EXAMPLE 5

Check the following beam to hollow section column joint for the design forces shown using Blind Bolts to the column.

In this example the tie force is less than the shear force.

The connections should be checked independently for shear forces and tying forces and not for both forces acting at the same time.



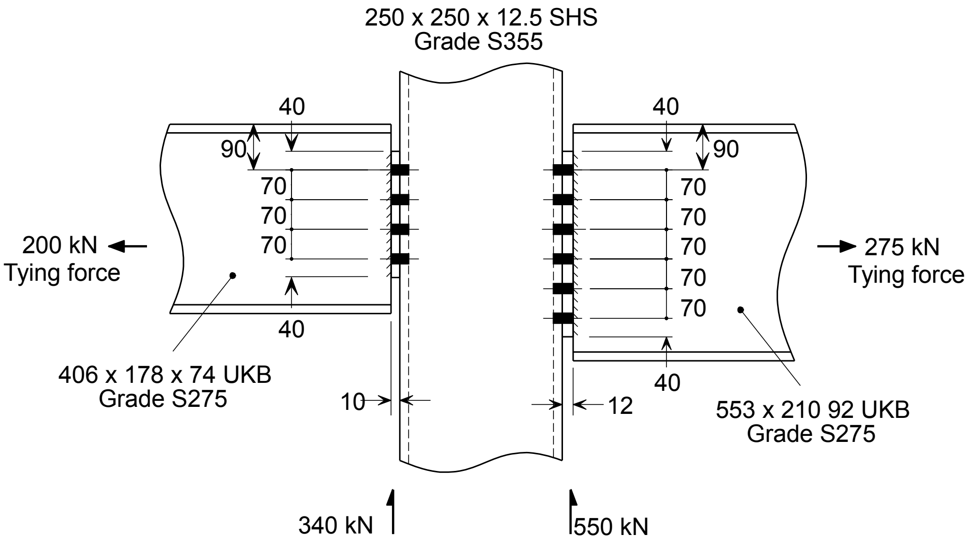


Design Information:

Bolts:	M20 Blind Bolts
Welds:	6 mm leg length fillet welds
Column:	S355
Beams:	S275
End plates:	S275

Title	*Example 5 – Partial depth end plate – Beam to hollow section column using Hollo-Bolts*	*Sheet 2 of 11*

CONNECTION DESIGN USING RESISTANCE TABLES



Although the connection resistance tables are based on ordinary bolts, they may be used to determine the vertical shear resistance of connections with Blind Bolts, because bolt shear resistance is generally not critical. The tables for ordinary bolts cannot be used to determine the connection tying resistance, as the bolt tension resistance has a significant influence on the tying resistance of the connection.

406 × 178 × 74 UKB, S275

End plate, 150 × 10 S275

Welds 6 mm fillet

Bolts M20

Bolts at 90 cross centres

4 rows of bolts

From Resistance Table G.4:

Connection shear resistance

= 394 kN > 340 kN

Minimum support thickness in S355

= 3.2 mm < 12.5 mm

Connection tying resistance

Table cannot be used

533 × 210 × 92 UKB, S275

End plate, 200 × 12 S275

Welds 6 mm fillet

Bolts M20

Bolts at 140 cross centres

6 rows of bolts

From Resistance Table G.4

Connection shear resistance

= 621 kN > 550 kN

Minimum support thickness in S355

= 3.4 mm < 12.5 mm

Connection tying resistance

Table cannot be used

Table G.4

Note:

(1) For connections using Blind Bolts, the tying resistance of the connection is the least of the values obtained from Checks 11, 12 & 13.

(2) The hollow section wall must also be checked as shown in Check 15.

Title *Example 5 – Partial depth end plate – Beam to hollow section column using Hollo-Bolts* *Sheet 3 of 11*

SUMMARY OF FULL DESIGN CHECKS FOR EXAMPLE 5

Notes:

(1) Checks 1 to 4 and 9 are as shown in Example 1.

(2) Tying forces are ignored when checking the shear resistance and shear is ignored calculating the tying resistance.

Sheet No.	CHECK		406 UKB (S275)		533 UKB (S275)		SHS Column, S355			
							406 UKB Side		533 UKB Side	
			Resist	Design force	Resist	Design force	Resist	Design force	Resist	Design force
	Check 1 Recommended detailing practice		All recommendations adopted							
	Check 2 Supported beam Welds (kN)		Full strength welds adopted – Not critical				Not applicable			
	Check 3		Not applicable							
	Check 4 Supported beam Web in shear	Shear resistance (kN)	394	340	621	550	Not applicable			
	Checks 5, 6, 7		Not applicable							
4	***Check 8*** Connection Bolt group	Bolt group (kN)	487	340	731	550	Not applicable			
	Check 9 Connection End plate in shear	Shear resistance (kN)	691	340	1195	550	Not applicable			
6	***Check 10*** Supporting column Shear and bearing	Shear and Bearing resistance (kN)	Not applicable				823	170	1196	275
8	***Check 11*** Tying resistance Plates and bolts	Tension (kN)	323	200	390	275	Not applicable			
10	***Check 12*** Structural Integrity Supported beam web	Tension (kN)	1027	200	1619	275	Not applicable			
10	***Check 13*** Structural Integrity Welds	Tension (kN)	Full strength welds adopted – not critical				Not applicable			
	Check 14		Not applicable							
11	***Check 15*** Structural Integrity Supporting column wall	Tension (kN)	Not applicable				431	200	850	275

Title	*Example 5 – Partial depth end plate – Beam to hollow section column using Blind Bolts*	*Sheet 4 of 11*

CONNECTION DESIGN FOLLOWING THE DESIGN PROCEDURES

Check 8: Connection – Bolt group

Basic requirement: $V_{Ed} \le F_{Rd}$

The resistance of the bolt group, F_{Rd}, is as follows:

If $F_{b,Rd} \le 0.8F_{v,Rd}$ then $F_{Rd} = nF_{b,Rd}$

if $F_{b,Rd} > 0.8F_{v,Rd}$ then $F_{Rd} = 0.8nF_{v,Rd}$

Shear resistance of a single bolt. For M20 Blind Bolts:

Conservatively asume the shear plane is over the slot:

$F_{v,Rd} = F_{v,Rd,slot} = 76.1$ kN — Table G.62

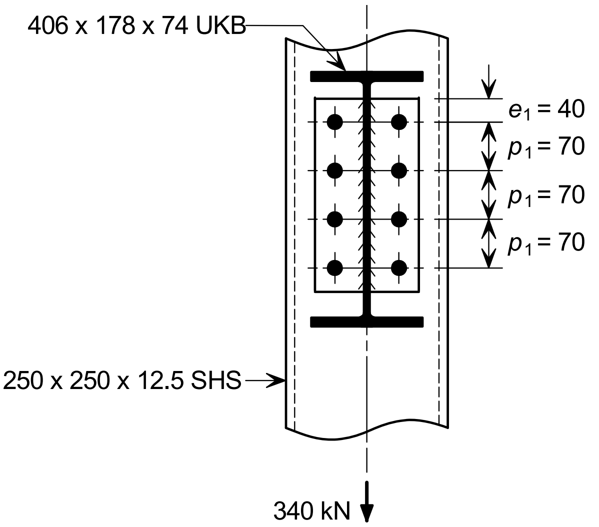
Bearing resistance of a single bolt:

$F_{b,Rd} = \min(F_{b,Rd,p};\ F_{b,Rd,2})$

$$F_{b,Rd,p} = \frac{k_{1,p}\alpha_{b,p}f_{u,p}dt_p}{\gamma_{M2}}$$

$$F_{b,Rd,2} = \frac{k_{1,2}\alpha_{b,2}f_{u,2}dt_2}{\gamma_{M2}}$$

406 × 178 × 74 UKB, S275



Since plate is 150 mm wide and $p_3 = 90$ mm then: $e_2 = 30$ mm — Appendix F

For an M20 Blind Bolt: $d = 20$ mm $d_0 = 22$ mm $f_{u,b} = 1000$ N/mm^2

Bearing on the end plate:

$$k_{1,p} = \min\left(2.8\frac{e_2}{d_0} - 1.7;\quad 1.4\frac{p_3}{d_0} - 1.7;\quad 2.5\right)$$

$$= \min\left(2.8\times\frac{30}{22} - 1.7;\quad 1.4\times\frac{90}{22} - 1.7;\quad 2.5\right) = \min(2.1;\ 4.0;\quad 2.5) = 2.1$$

$$\alpha_{b,p} = \min\left(\frac{e_1}{3d_0};\ \frac{p_1}{3d_0} - \frac{1}{4};\ \frac{f_{ub}}{f_{u,p}};\ 1.0\right) = \min\left(\frac{40}{3\times22};\ \frac{70}{3\times22} - 0.25;\ \frac{1000}{410};\ 1.0\right)$$

$f_{u,p}$ from Table 7 of EN 10025-2

$$= \min(0.61;\quad 0.81;\quad 2.44;\quad 1.0) = 0.61$$

$$F_{b,Rd,p} = \frac{2.1\times0.61\times410\times20\times10}{1.25}\times10^{-3} = 84.0\text{ kN}$$

Bearing on the supporting column:

Since the hollow section wall is 12.5 mm thick and S355, clearly the end plate is critical.

Therefore can be assumed that:

$F_{b,Rd,2} > F_{b,Rd,p}$

$F_{b,Rd} = \min(F_{b,Rd,p};\ F_{b,Rd,2}) = 84.0$ kN

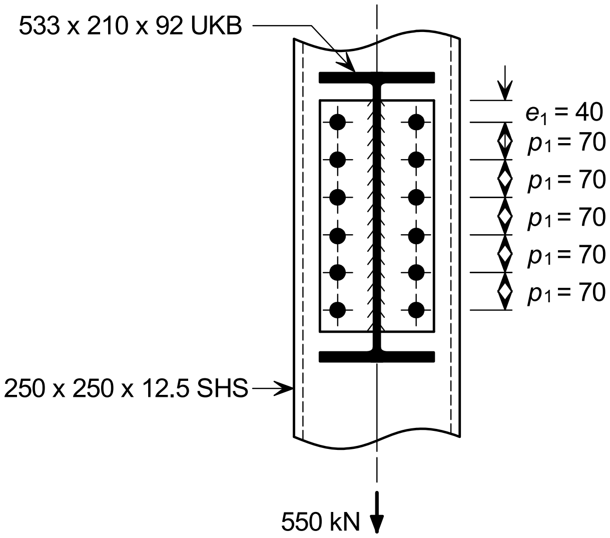
$0.8F_{v,Rd} = 0.8 \times 76.1 = 60.9$ kN

$\therefore\ F_{b,Rd} = 84.0$ kN > 60.9 kN

$\therefore\ F_{Rd} = 0.8nF_{v,Rd} = 0.8 \times 8 \times 76.1 = 487$ kN

$\therefore\ V_{Ed} = 340$ kN < 487 kN ∴ **O.K.**

533 × 210 × 92 UKB, S275



Since plate is 200 mm wide and p_3 = 140 mm then: e_2 = 30 mm

Bearing on the end plate:

$$k_{1,p} = \min\left(2.8\frac{e_2}{d_0} - 1.7;\quad 1.4\frac{p_3}{d_0} - 1.7;\quad 2.5\right)$$

$$= \min\left(2.8 \times \frac{30}{22} - 1.7;\ 1.4 \times \frac{140}{22} - 1.7;\ 2.5\right) = \min(2.1;\ 7.2;\ 2.5) \quad = 2.1$$

$$\alpha_{b,p} = \min\left(\frac{e_1}{3d_0};\ \frac{p_1}{3d_0} - \frac{1}{4};\ \frac{f_{ub}}{f_{u,p}};\ 1.0\right) = \min\left(\frac{40}{3 \times 22};\ \frac{70}{3 \times 22} - 0.25;\ \frac{1000}{410};\ 1.0\right)$$

$$= \min(0.61;\ 0.81;\ 2.44;\ 1.0) = 0.61$$

$$F_{b,Rd,p} = \frac{2.1 \times 0.61 \times 410 \times 20 \times 12}{1.25} \times 10^{-3} = 100.8 \text{ kN}$$

Bearing on the supporting column:

Since the hollow section wall is 12.5 mm thick and S355, clearly the end plate is critical.

Therefore can be assumed that:

$F_{b,Rd,2} > F_{b,Rd,p}$

$F_{b,Rd} = \min(F_{b,Rd,p};\ F_{b,Rd,2}) = 100.8$ kN

$0.8F_{v,Rd} = 0.8 \times 76.1 = 60.9$ kN

$\therefore\ F_{b,Rd} = 100.8$ kN > 60.9 kN

$\therefore\ F_{Rd} = 0.8nF_{v,Rd} = 0.8 \times 12 \times 76.1 = 731$ kN

$\therefore\ V_{Ed} = 550$ kN < 731 kN ∴ **O.K.**

Title	*Example 5 – Partial depth end plate – Beam to hollow section column using Blind Bolts*	*Sheet 6 of 11*

Check 10: Supporting column – Shear and bearing

Local shear and bearing resistance of the hollow section column wall

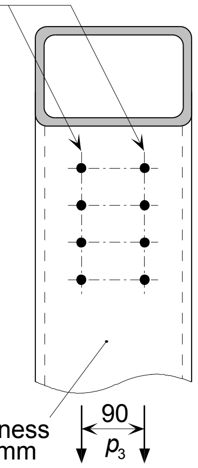
(i) Shear:

Basic requirement: $\frac{V_{Ed}}{2} \leq V_{Rd,min}$

$$V_{Rd,min} = \min\left(\frac{A_v f_{y,2}}{\sqrt{3}\gamma_{M0}};\quad \frac{A_{v,net} f_{u,2}}{\sqrt{3}\gamma_{M2}}\right)$$

406 × 178 × 74 UKB, S275

Critical sections



wall thickness t_2 = 12.5 mm

90 p_3

$\frac{V_{Ed}}{2}$ $\frac{V_{Ed}}{2}$ = 170 kN

Shear area of gross section: $A_v = t_2\,(e_t + (n_1 - 1)p_1 + e_b)$

$$e_b = \min\left(e_{1,b};\frac{p_3}{2};\quad 5d\right)$$

Since the connection is not near the bottom of the column $e_{1,b}$ is not applicable.

$$e_b = \min\left(\frac{p_3}{2};\quad 5d\right) = \min\left(\frac{90}{2};\quad 5\times 20\right) = 45 \text{ mm}$$

$$e_t = \min\left(e_{1,t};\quad 5d\right)$$

Since the connection is not near the top of the column $e_{1,t}$ is not applicable.

e_t = 5 × 20 = 100 mm

$\therefore A_v$ = 12.5 × (100 + (4 – 1) × 70 + 45) = 4438 mm²

t_2 < 16 mm, hence $f_{y,2}$ = 355 N/mm²

Therefore the resistance of the gross section is:

$$\therefore \frac{A_v f_{y,2}}{\sqrt{3}\gamma_{M0}} = \frac{4438\times 355}{\sqrt{3}\times 1.0}\times 10^{-3} = 910 \text{ kN}$$

Shear area of net section: $A_{v,net} = A_v - n_1 d_0 t_2$

$\therefore A_{v,net}$ = 4438 – 4 × 22 × 12.5 = 3338 mm²

Therefore the resistance of the net section is:

$$\therefore \frac{A_{v,net} f_{u,2}}{\sqrt{3}\gamma_{M2}} = \frac{3338\times 470}{\sqrt{3}\times 1.1}\times 10^{-3} = 823 \text{ kN}$$

$f_{u,2}$ for S355 from Table A.3 of EN 10210

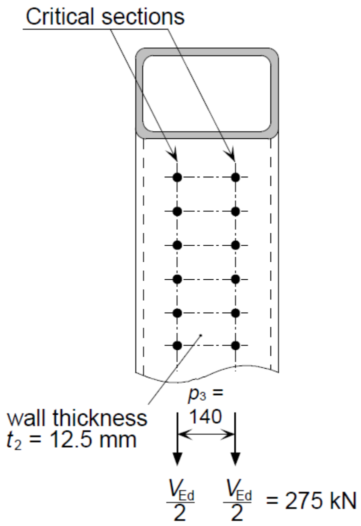
$\therefore V_{Rd,min} = \min(910;\quad 823) = 823$ kN

$\frac{V_{Ed}}{2}$ = 170 kN < 823 kN

∴ **O.K.**

Title	*Example 5 – Partial depth end plate – Beam to hollow section column using Blind Bolts*	Sheet 7 of 11

533 × 210 × 92 UKB, S275



Shear area of gross section: $A_v = t_2\,(e_t + (n_1 - 1)p_1 + e_b)$

$e_b \quad = \min\left(e_{1,b};\ \frac{p_3}{2};\ \ 5d\right)$

Since the connection is not near the bottom of the column $e_{1,b}$ is not applicable.

$e_b \quad = \min\left(\frac{p_3}{2};\ \ 5d\right) = \min\left(\frac{140}{2};\ \ 5 \times 20\right) = 70$ mm

$e_t \quad = \min\left(e_{1,t};\ \ 5d\right)$

Since the connection is not near the top of column $e_{1,t}$ is not applicable.

$e_t \quad = 5 \times 20 = 100$ mm

$\therefore A_v \quad = 12.5 \times (100 + (6 - 1) \times 70 + 70) = 6500$ mm^2

Therefore the resistance of the gross section is:

$$\therefore \frac{A_v f_{y,2}}{\sqrt{3}\gamma_{M0}} = \frac{6500 \times 355}{\sqrt{3} \times 1.0} \times 10^{-3} = 1332 \text{ kN}$$

Shear area of net section: $A_{v,net} = A_v - n_1 d_0 t_2$

$\therefore A_{v,net} \quad = 6500 - 6 \times 22 \times 12.5 = 4850$ mm^2

Therefore the resistance of the net section is:

$$\therefore \frac{A_{v,net} f_{u,2}}{\sqrt{3}\gamma_{M2}} = \frac{4850 \times 470}{\sqrt{3} \times 1.1} \times 10^{-3} = 1196 \text{ kN}$$

$\therefore V_{Rd,min} = \min\left(1332;\ \ 1196\right) = 1196$ kN

$\frac{V_{Ed}}{2} \quad = 275 \text{ kN} < 1196 \text{ kN}$ **∴ O.K.**

(ii) Bearing resistance

Bearing resistance in the column wall will not be critical when compared to the bearing resistance in the end plates (see Check 8).

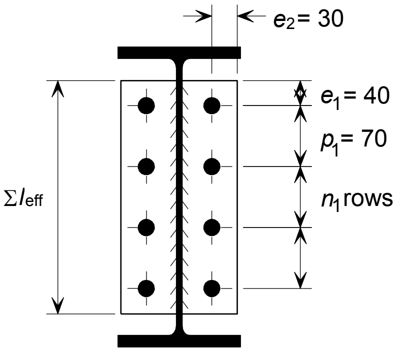
Title	*Example 5 – Partial depth end plate – Beam to hollow section column using Blind Bolts*	*Sheet 8 of 11*

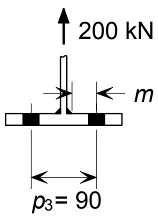
Check 11: Tying resistance – Plate and bolts

Resistance of end plate

Basic requirement: $F_{Ed} \leq \min(F_{Rd,u,1};\ F_{Rd,u,2};\ F_{Rdu,3})$

406 × 178 × 74 UKB, S275





Mode 1:

$$F_{Rd,u,1} = \frac{(8n - 2e_w)M_{pl,1,Rd,u}}{2mn - e_w(m+n)}$$

Σl_{eff} is the effective length of the equivalent T-stub = h_p = 290 mm

$$M_{pl,1,Rd,u} = \frac{0.25\Sigma l_{eff} t_p^{\ 2} f_{u,p}}{\gamma_{M,u}} = \frac{0.25 \times 290 \times 10^2 \times 410}{1.1} \times 10^{-6} = 2.7 \text{ kNm}$$

$a\sqrt{2}$ is the weld leg length = 6 mm

$$m = \frac{p_3 - t_{w,b1} - 2 \times 0.8 \times a\sqrt{2}}{2} = \frac{90 - 9.5 - 2 \times 0.8 \times 6}{2} = 35.5 \text{ mm}$$

$$e_w = \frac{d_w}{4} = \frac{37}{4} = 9.25 \text{ mm}$$

d_w from Table G.66

$$n = \min(e_2;\ 1.25m) = \min(30;\ 1.25 \times 35.5) = 30 \text{ mm}$$

$$\therefore F_{Rd,u,1} = \frac{(8 \times 30 - 2 \times 9.25) \times 2.7 \times 10^6}{2 \times 35.5 \times 30 - 9.25 \times (35.5 + 30)} \times 10^{-3} = 392 \text{ kN}$$

Mode 2:

$$F_{Rd,u,2} = \frac{2M_{pl,2,Rd,u} + n\Sigma F_{t,Rd,u}}{m+n}$$

$M_{pl,2,Rd,u}$ = $M_{pl,1,Rd,u}$ = 2.7 kNm

$F_{t,Rd,u}$ = 65.7 kN — Table G.63

$$F_{Rd,u,2} = \frac{2 \times 2.7 \times 10^6 + 30 \times 8 \times 65.7 \times 10^3}{35.5 + 30} \times 10^{-3} = 323 \text{ kN}$$

Mode 3:

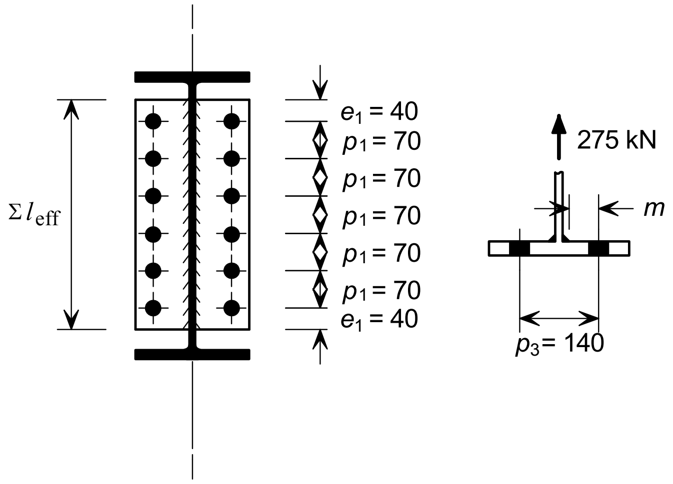
$F_{Rd,u,3}$ = $\Sigma F_{t,Rd,u}$ = 8 × 65.7 = 526 kN

$$\min(F_{Rd,u,1},\ F_{Rd,u,2},\ F_{Rd,u,3}) = \min(392,\ 323;\ 526) = 323 \text{ kN}$$

F_{Ed} = 200 kN < 323 kN — **∴ O.K.**

Title	*Example 5 – Partial depth end plate – Beam to hollow section column using Blind Bolts*	*Sheet 9 of 11*

533 × 210 × 92 UKB, S275



Mode 1:

$F_{Rd,u,1} = \dfrac{(8n - 2e_w)M_{pl,1,Rd,u}}{2mn - e_w(m+n)}$

Σl_{eff} is the effective length of the equivalent T-stub = h_p = 430 mm

$M_{pl,1,Rd,u} = \dfrac{0.25\Sigma l_{eff} t_p^{\ 2} f_{u,p}}{\gamma_{M,u}} = \dfrac{0.25 \times 430 \times 12^2 \times 410}{1.1} \times 10^{-6}$ = 5.77 kNm

$a\sqrt{2}$ is the weld leg length = 6 mm

$m = \dfrac{p_3 - t_{w,b1} - 2 \times 0.8 \times a\sqrt{2}}{2} = \dfrac{140 - 10.1 - 2 \times 0.8 \times 6}{2}$ = 60.15 mm

$e_w = \dfrac{d_w}{4} = \dfrac{37}{4}$ = 9.25 mm — Table G.69

$n = \min(e_2;\ 1.25m) = \min(30;\ 1.25 \times 60.15)$ = 30 mm

$\therefore F_{Rd,u,1} = \dfrac{(8 \times 30 - 2 \times 9.25) \times 5.77 \times 10^6}{2 \times 60.15 \times 30 - 9.25 \times (60.15 + 30)} \times 10^{-3}$ = 461 kN

Mode 2:

$F_{Rd,u,2} = \dfrac{2M_{pl,2,Rd,u} + n\Sigma F_{t,Rd,u}}{m+n}$

$M_{pl,2,Rd,u} = M_{pl,1,Rd,u}$ = 5.77 kNm

$F_{t,Rd,u}$ = 65.7 kN — Table G.63

$F_{Rd,u,2} = \dfrac{2 \times 5.77 \times 10^6 + 30 \times 12 \times 65.7 \times 10^3}{60.15 + 30} \times 10^{-3}$ = 390 kN

Mode 3:

$F_{Rd,u,3} = \Sigma F_{t,Rd,u}$ = 12 × 65.7 = 788 kN

$\min(F_{Rd,u,1},\ F_{Rd,u,2},\ F_{Rd,u,3}) = \min(461,\ 390;\ 788)$ = 390 kN

F_{Ed} = 275 kN < 390 kN — **∴ O.K.**

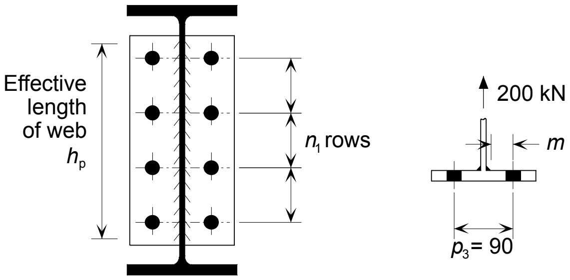
Title	Example 5 – Partial depth end plate – Beam to hollow section column using Blind Bolts	Sheet 10 of 11

Check 12: Tying resistance – Supported beam web

Resistance of the beam web

Basic requirement: $F_{Ed} \leq F_{Rd}$

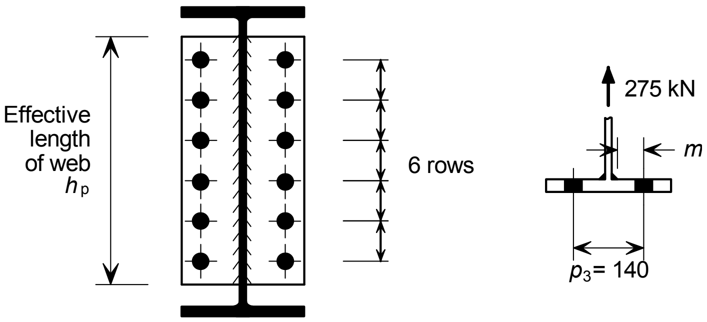
406 × 178 × 74 UKB, S275



$$F_{Rd} = \frac{t_{w,b1} h_p f_{u,b1}}{\gamma_{M,u}} = \frac{9.5 \times 290 \times 410}{1.1} \times 10^{-3} = 1027 \text{ kN}$$

F_{Ed} = 200 kN < 1027 kN ∴ **O.K.**

533 × 210 × 92 UKB, S275



$$F_{Rd} = \frac{t_{w,b1} h_p f_{u,b1}}{\gamma_{M,u}} = \frac{10.1 \times 430 \times 410}{1.1} \times 10^{-3} = 1619 \text{ kN}$$

F_{Ed} = 275 kN < 1619 kN ∴ **O.K.**

Check 13: Tying resistance – Welds

Basic requirement: $a \leq 0.40\ t_{w,b1}$

406 × 178 × 74 UKB, S275

Throat thickness:

$$a = \frac{6}{\sqrt{2}} = 4.24 \text{ mm}$$

$0.40\ t_{w,b1} = 0.40 \times 9.5 = 3.8$ mm

a = 4.24 mm ≥ 3.8 mm ∴ **O.K.**

533 × 210 × 92 UKB, S275

Throat thickness:

$$a = \frac{6}{\sqrt{2}} = 4.24 \text{ mm}$$

$0.40\ t_{w,b1} = 0.40 \times 10.1 = 4.04$ mm

a = 4.24 mm ≥ 4.04 mm ∴ **O.K.**

Title	*Example 5 – Partial depth end plate – Beam to hollow section column using Blind Bolts*	Sheet 11 of 11

Check 15: Tying resistance – Supporting column wall

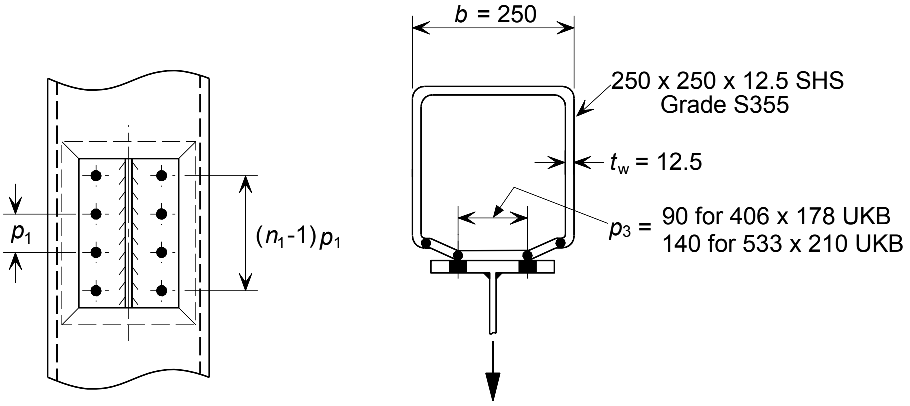
Resistance of hollow section wall

Basic requirement: $F_{Ed} \leq F_{Rd}$

$$F_{Rd} = \frac{8M_{pl,Rd,u}}{(1-\beta_1)}\left(\eta_1 + 1.5(1-\beta_1)^{0.5} \times (1-\gamma_1)^{0.5}\right)$$

$$M_{pl,Rd,u} = \frac{f_{u,c}\, t_2{}^2}{4\gamma_{M,u}}$$

406 × 178 × 74 UKB, S275



$$M_{pl,Rd,u} = \frac{470 \times 12.5^2}{4 \times 1.1} \times 10^{-3} = 16.7 \text{ kNm/mm}$$

$f_{u,c}$ for S355 from Table A.3 of EN 10210-1

$$\beta_1 = \frac{p_3}{b-3t_2} = \frac{90}{250-3\times12.5} = 0.424$$

$$\gamma_1 = \frac{d_0}{b-3t_2} = \frac{22}{250-3\times12.5} = 0.104$$

$$\eta_1 = \frac{(n_1-1)p_1 - \frac{n_1}{2}d_0}{b-3t_2} = \frac{(4-1)\times70 - \frac{4}{2}\times22}{250-3\times12.5} = 0.781$$

$$F_{Rd,u} = \frac{8\times16.7}{(1-0.424)}\times\left(0.781 + 1.5\times(1-0.424)^{0.5}\times(1-0.104)^{0.5}\right) = 431 \text{ kN}$$

F_{Ed} = 200 kN < 431 kN ∴ **O.K.**

533 × 210 × 92 UKB, S275

$$M_{pl,Rd,u} = \frac{470 \times 12.5^2}{4 \times 1.1} \times 10^{-3} = 16.7 \text{ kNm/mm}$$

$$\beta_1 = \frac{p_3}{b-3t_2} = \frac{140}{250-3\times12.5} = 0.659$$

$$\gamma_1 = \frac{d_0}{b-3t_2} = \frac{22}{250-3\times12.5} = 0.104$$

$$\eta_1 = \frac{(n_1-1)p_1 - \frac{n_1}{2}d_0}{b-3t_2} = \frac{(6-1)\times70 - \frac{6}{2}\times22}{250-3\times12.5} = 1.34$$

$$F_{Rd,u} = \frac{8\times16.7}{(1-0.659)}\times\left(1.34 + 1.5\times(1-0.659)^{0.5}\times(1-0.104)^{0.5}\right) = 850 \text{ kN}$$

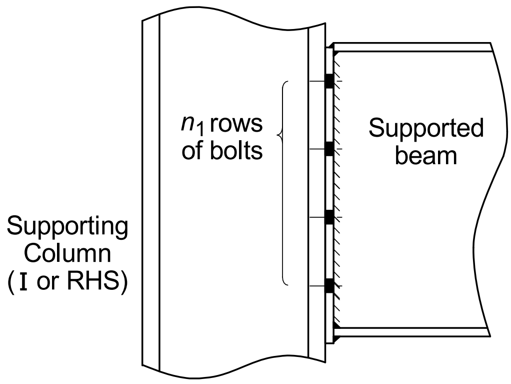
F_{Ed} = 275 kN < 850 kN ∴ **O.K.**

4.7 DESIGN PROCEDURES FOR FULL DEPTH END PLATES

The design model used in this publication is in accordance with traditional UK design practice and is based on the simply supported beam end reaction.

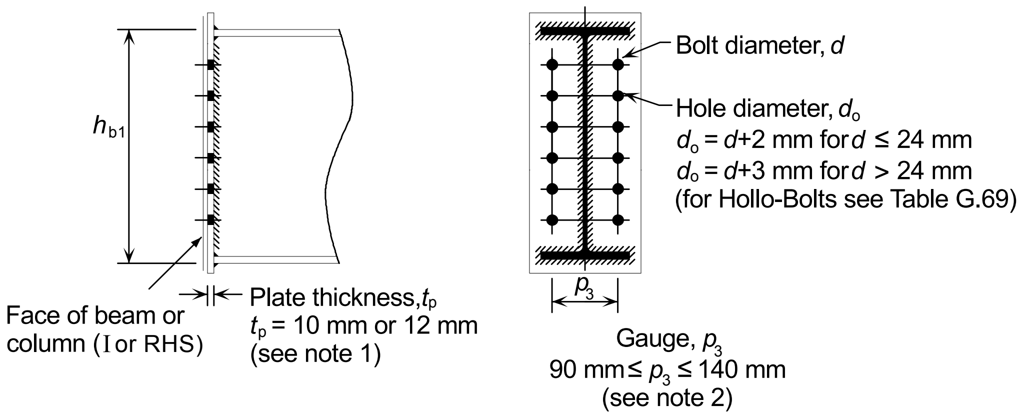
The design procedure applies to beams connected to a column flange, a column web, a supporting beam web or to a hollow section column.

The checks can be modified to cover end plates welded to one flange only, though this situation is not covered in the publication.



Check	1	Recommended detailing practice		
Check	2	Supported beam	–	Welds
Check	3	*Not applicable*		
Check	4	Supported beam	–	Web in shear
Check	5	*Not applicable*		
Check	6	*Not applicable*		
Check	7	*Not applicable*		
Check	8	Connection	–	Bolt group
Check	9	*Not applicable*		
Check	10	Supporting beam/column	–	Shear and bearing
Check	11	Tying resistance	–	Plate and bolts
Check	12	Tying resistance	–	Supported beam web
Check	13	Tying resistance	–	Welds
Check	14	Tying resistance	–	Supporting column web (UKC or UKB)
Check	15	Tying resistance	–	Supporting column wall (Hollow Section)
Check	16	*Not applicable*		

CHECK 1	Recommended detailing practice



Notes:

(1) In order to satisfy rotational requirements for pinned connections according to BS EN 1993-1-8, the maximum end plate thickness is 12 mm for S275 fittings. For shallow beams, the use of 10 mm thick end plates is recommended. Increasing the end plate thickness above this value may lead to an increased moment resistance which does not satisfy BS EN 1993-1-8 requirements for pinned connections.

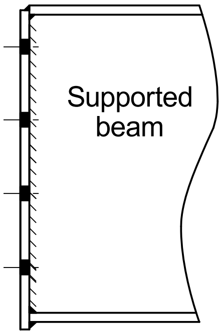
(2) Reducing the gauge will increase the tying resistance and the connection stiffness. Care must be taken to ensure that conditions in BS EN 1993-1-8, 5.2 are satisfied.

(3) The effective lengths calculated in Check 11 are valid provided $p_1 < 4m + 1.25e$

(4) The tying resistance assumes that a plastic distribution of bolt forces is possible. For this to be appropriate, the end plate thickness, t_p, must be limited to $t_p \leq \frac{d}{1.9}\sqrt{\frac{f_{ub}}{f_{y,p}}}$

This requirement is satisfied with M20, 8.8 bolts and 12 mm S275 plates. Although thicker plates will also satisfy this requirement, they are not recommended for the reasons explained above.

CHECK 2	Supported beam – Welds



Resistance of web fillet welds connecting end plate to beam web:

Basic requirement:

$a \geq 0.40\ t_{w,b1}$ for S275 supported beam

$a \geq 0.48\ t_{w,b1}$ for S355 supported beam

where:

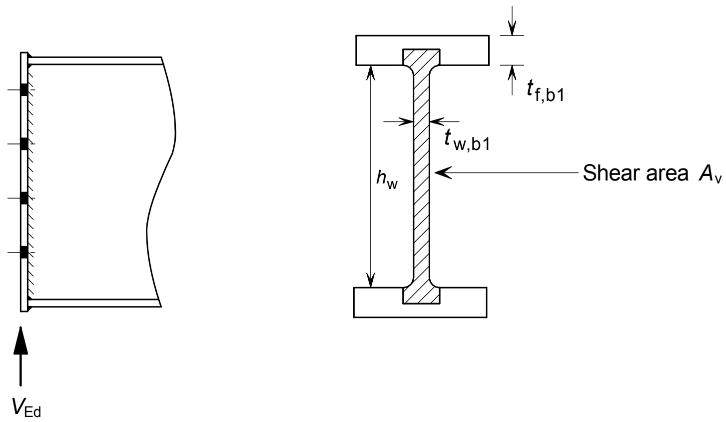
a is the effective weld throat thickness = $0.7s$ (normally)

s is the weld leg length

$t_{w,b1}$ is the web thickness of the supported beam

See Appendix C for more details about the weld requirements.

CHECK 4 — Supported beam – Web in shear



Resistance of the beam web (complete section is effective)

Basic requirement:

$V_{Ed} \leq V_{c,Rd}$

$V_{c,Rd}$ is the design shear resistance of the supported beam connected to the end plate

$$= A_v \frac{f_{y,b1}}{\sqrt{3}\gamma_{M0}}$$

where:

A_v is the shear area

$= A_{b1} - 2b_{b1}t_{f,b1} + (t_{w,b1} + 2r_{b1})t_{f,b1}$
but not less than $\eta\, h_w\, t_{w,b1}$

h_w $= (h - 2t_{f,b1})$

A_{b1} is the gross area of the supported beam

$t_{f,b1}$ is the thickness of the supported beam flange

$t_{w,b1}$ is the thickness of the supported beam web

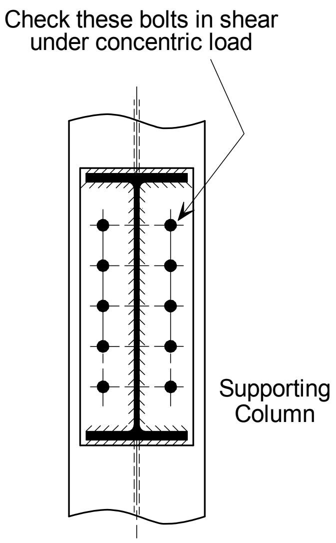
b_{b1} is the width of the supported beam

d_{b1} is the straight part of the supported beam web

r_{b1} is the root radius of the supported beam

η is given by BS EN 1993-1-5 but may be conservatively taken as 1.0

CHECK 8	**Connection – Bolt group**



Shear and bearing resistance of the bolt group connecting the end plate to the supporting beam or column.

Basic requirement:

$V_{Ed} \le F_{Rd}$

F_{Rd} is the resistance of the bolt group

If $F_{b,Rd} \le 0.8F_{v,Rd}$* then $F_{Rd} = nF_{b,Rd}$

If $F_{b,Rd} > 0.8F_{v,Rd}$* then $F_{Rd} = 0.8nF_{v,Rd}$ *

* The reduction factor 0.8 allows for the presence of tension in the bolts[24].

Shear resistance

$F_{v,Rd}$ is the shear resistance of one bolt

$$= \frac{\alpha_v f_{ub} A}{\gamma_{M2}}$$

Bearing resistance:

$F_{b,Rd}$ is the minimum of the bearing resistance on the end plate and the bearing resistance on the supporting member per bolt

$$= \min\left(F_{b,Rd,p};\quad F_{b,Rd,2}\right)$$

Bearing on the end plate:

$$F_{b,Rd,p} = \frac{k_{1,p}\alpha_{b,p} f_{u,p} d t_p}{\gamma_{M2}}$$

Bearing on the supporting member:

$$F_{b,Rd,2} = \frac{k_{1,2}\alpha_{b,2} f_{u,2} d t_2}{\gamma_{M2}}$$

where:

α_v = 0.6 for 8.8 bolts

A is the tensile stress area of the bolt, A_s

d_0 is the diameter of the holes

t_p is the thickness of the plate

t_2 is the thickness of the supporting member

$$\alpha_{b,p} = \min\left(\frac{p_1}{3d_0} - \frac{1}{4};\quad \frac{f_{ub}}{f_{u,p}};\quad 1.0\right)$$

$$\alpha_{b,2} = \min\left(\frac{p_1}{3d_0} - \frac{1}{4};\quad \frac{f_{ub}}{f_{u,2}};\quad 1.0\right)$$

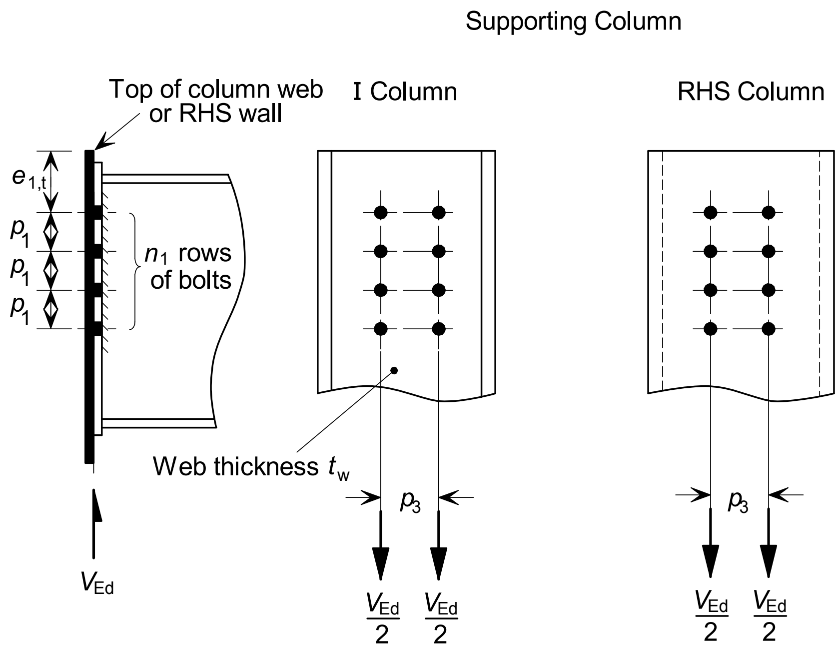
$$k_{1,p} = \min\left(2.8\frac{e_2}{d_0} - 1.7;\quad 2.5\right)$$

$$k_{1,2} = \min\left(2.8\frac{e_{2,b}}{d_0} - 1.7;\quad 2.5\right)$$

If supporting member is a beam or a column web then $k_{1,2}$ = 2.5

γ_{M2} is the partial factor for resistance of bolts (γ_{M2} = 1.25 as given in the National Annex)

CHECK 10	Supporting beam/column – Shear and bearing (with one supported beam)



Local shear resistance of supporting column web for one supported beam

Shear:

Basic requirement:

$V_{Ed}/2 \leq V_{Rd,min}$

$V_{Rd,min}$ is the local shear resistance of supporting member

$$= \min\left(\frac{A_v f_{y,2}}{\sqrt{3}\gamma_{M0}};\ \frac{A_{v,net} f_{u,2}}{\sqrt{3}\gamma_{M2}}\right)$$

A_v is the shear area of supporting member

$= (e_t + (n_1 - 1)\ p_1 + e_b)\ t_2$

$A_{v,net}$ is the net shear area of supporting member

$= A_v - n_1 d_0 t_2$

where:

e_t = min ($e_{1,t}$; $5d$)

e_b = min ($p_3/2$; $5d$)

d_0 is the diameter of the holes

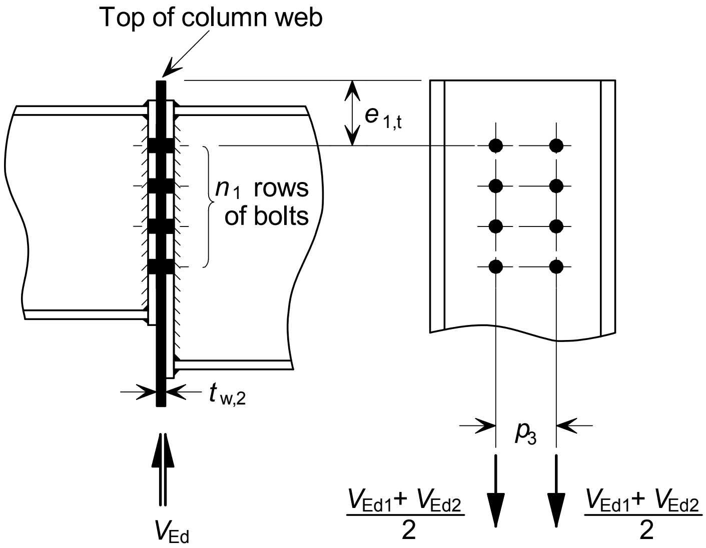
d is the diameter of the bolt

n_1 is the number of bolt rows

t_2 is the thickness of the supporting member

CHECK 10 (continued)	**Supporting beam/column – Shear and bearing** (with two supported beams)

Supporting I Column



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

Shear:

Basic requirement:

$$\frac{V_{Ed,1}}{2} + \frac{V_{Ed,2}}{2} \leq V_{Rd,min}$$

$V_{Rd,min}$ is the local shear resistance of supporting member

$$= \min\left(\frac{A_v f_{y,2}}{\sqrt{3}\gamma_{M0}}; \frac{A_{v,net} f_{u,2}}{\sqrt{3}\gamma_{M2}}\right)$$

Bearing:

Basic requirement:

$$\frac{V_{Ed,1}}{2n_1} + \frac{V_{Ed,2}}{2n_1} \leq F_{b,Rd}$$

$F_{b,Rd}$ is the bearing resistance of a single bolt. See Check 8 for equations to calculate this value.

where:

A_v is the shear area of the supporting member
$= (e_t + (n_1 - 1)p_1 + e_b)t_{w,2}$

$A_{v,net}$ is the net shear area of the supporting member
$= A_v - n_1 d_0 t_{w,2}$

e_t = min ($e_{1,t}$; $5d$)

e_b = min ($e_{1,b}$; $p_3/2$; p_1 ; $5d$)

$t_{w,2}$ is the thickness of the supporting element (column web or beam web)

d_0 is the diameter of the holes

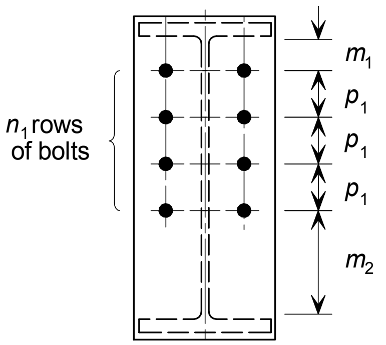
d is the diameter of the bolt

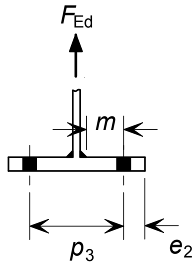
n_1 is the number of bolt rows

Note:

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

CHECK 11	Tying resistance – Plate and bolts





Resistance of end plate.

Basic requirement:

$F_{Ed} \quad \leq \quad F_{Rd}$

F_{Rd} = tying resistance of end plate

$= \min\left(F_{Rd,1}\,;\quad F_{Rd,2}\,;\quad F_{Rd,3}\right)$

$$F_{Rd,1} = \frac{\left(8n - 2e_w\right)M_{pl,Rd,u}}{2mn - e_w\left(m+n\right)}$$

$$M_{pl,Rd,u} = \frac{0.25\Sigma\ell_{eff}t_p^{\,2}f_{u,p}}{\gamma_{M,u}}$$

$$F_{Rd,2} = \frac{2M_{pl,Rd,u} + n\sum F_{t,Rd}}{m+n}$$

$$F_{Rd,3} = \sum F_{t,Rd}$$

$$F_{t,Rd} = \frac{k_2 f_{ub} A_s}{\gamma_{M,u}}$$ for ordinary bolts.

(For Flowdrill, Hollo-Bolt and Blind Bolt resistances the value of $F_{t,Rd,u}$ should be taken from Tables G.59, G.61 and G.63 respctively)

where:

$\ell_{eff} = \ell_{eff,t} + (n\text{-}1)p_1 + \ell_{eff,b}$

$$\ell_{eff,t} = \max\left(\alpha_t m - \left(\frac{4m + 1.25e_2}{2}\right);\quad \left(\frac{4m + 1.25e_2}{2}\right)\right)$$

$$\ell_{eff,b} = \max\left(\alpha_b m - \left(\frac{4m + 1.25e_2}{2}\right);\quad \left(\frac{4m + 1.25e_2}{2}\right)\right)$$

α_t and α_b are from Figure 7 for the top and bottom flanges based on λ_1 and λ_2 for the respective flanges

t_p is the end plate thickness

k_2 = 0.63 for countersunk bolts

= 0.9 otherwise

A_s is the tensile stress area of the bolt

$$m = \frac{p_3 - t_{w,b1} - 2 \times 0.8 \times a\sqrt{2}}{2}$$

$t_{w,b1}$ is the web thickness of the supported beam

a is the weld throat thickness

$n = e_{min}$ but $n \leq 1.25m$

$e_{min} = \min\left(e_2;\ e_{2,c}\right)$

$e_{2,c}$ is the edge distance on the column flange

$$e_w = \frac{d_w}{4}$$

d_w is the diameter of the washer or the width across points of the bolt or nut as relevant

The effective lengths, ℓ_{eff}, given above, are appropriate as long as p_1 is not large compared to other connection geometry. The expressions given above are appropriate if $p_1 < 4m + 1.25e_2$. If p_1 is larger than this limit, P398[19] should be consulted and the effective lengths worked out from first principles.

CHECK 11 (continued)	Tying resistance – Plate and bolts

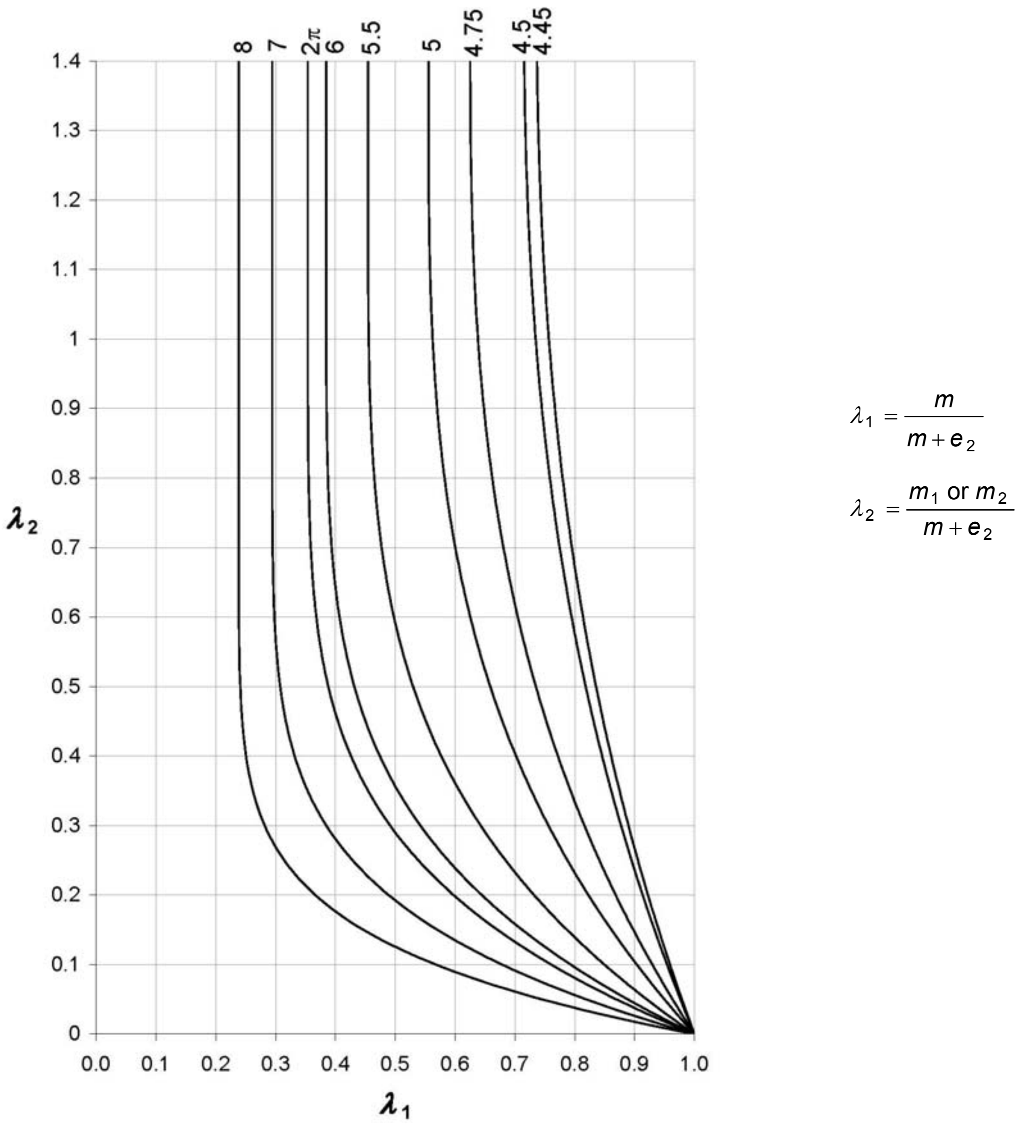
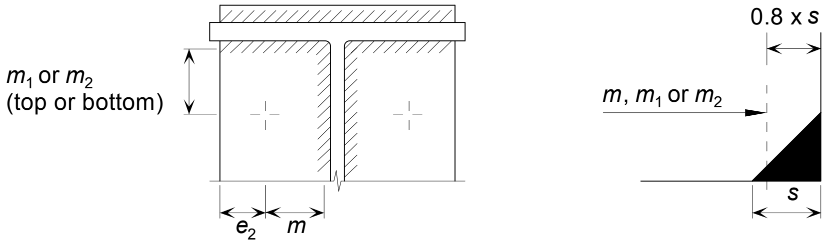


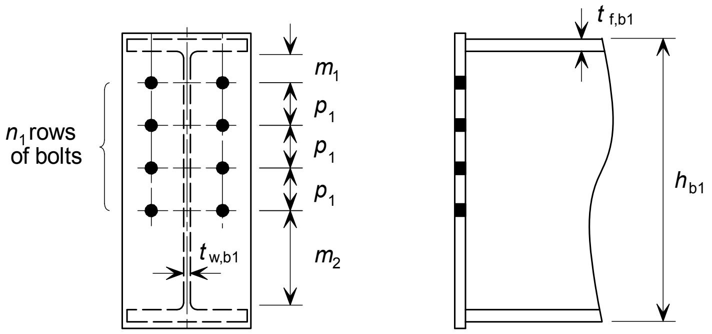
Figure 7 Values of α based on connection geometry



Setting out for Figure 7

Mathematical expressions to determine α are provided in P398[19].

CHECK 12	Tying resistance – supported beam web



Resistance of supported beam web

Basic requirement:

$F_{Ed} \leq F_{t,Rd,u}$

$$F_{t,Rd,u} = \frac{t_{w,b1}\left(h_{b1} - 2t_{f,b1}\right)f_{u,b1}}{\gamma_{Mu}}$$

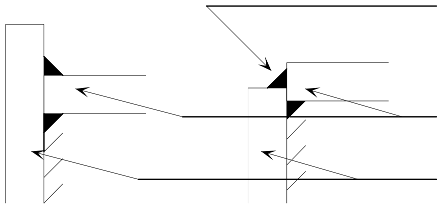
where:

$t_{w,b1}$ is the web thickness of the supported beam

h_{b1} is the height of the supported beam

$t_{f,b1}$ is the flange thickness of the supported beam

CHECK 13	Tying resistance – Welds



Flange weld tension resistance

Basic requirement:

$F_{Ed} / 2 \quad \leq \quad F_{Rd,flangeweld}$

$F_{Rd,flangeweld} = 2 f_{vw,d}\ a\ (b - 2s)$

$$f_{vw,d} = \frac{f_u / \sqrt{3}}{\beta_w \gamma_{Mu}}$$

where:

a	is the weld throat	
	= 0.7s	
s	is the leg length	
b	is the width of the supported beam or the width of the end plate, whichever is smaller	
β_w	= 0.85	for S275
	= 0.9	for S355

Web weld tension resistance

The weld size specified in Check 2 will be adequate for the web weld in tying.

See Appendix C for more details of weld requirements.

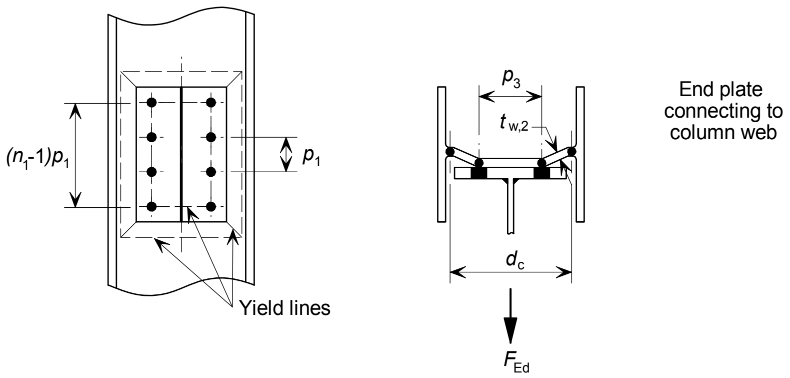
Note 1: In this tying model for flush end plates it is recommended that the flange welds be designed for half the tying force. Some designers might find this to be unnecessarily conservative and it is acknowledged that a more efficient design may be feasible. However due to the lack of tests to confirm this, the more conservative approach has been chosen in this publication.

Note 2: The design method given in Check 11 relies on the development of yield line patterns near the flange, which requires that the end plate be continuous with the flange. The designer must ensure that this restraint is made effective by providing adequate weld details. Adequate flange weld details include those shown in the figure. Other details may be possible.

Note 3: Research is underway which may lead to a less conservative model.

Note 4: As an alternative to calculation, a full strength weld to the flange may be provided, following the guidance in Appendix C.

CHECK 14	Tying resistance – supporting column web (UKC or UKB)



Resistance of the column web

Basic requirement:

$F_{Ed} \leq F_{Rd,u}$

$$F_{Rd,u} = \frac{8M_{pl,Rd,u}}{1-\beta_1}\left[\eta_1 + 1.5(1-\beta_1)^{0.5} \times (1-\gamma_1)^{0.5}\right]$$

$$M_{pl,Rd,u} = \frac{f_{u,2}t_{w,2}^{\ 2}}{4\gamma_{M,u}}$$

The factor 1.5 in the equation for $F_{Rd,u}$ includes an allowance for the axial compression in the column.

where:

$$\eta_1 = \frac{(n_1-1)p_1 - \frac{n_1}{2}d_0}{d_c}$$

$$\beta_1 = \frac{p_3}{d_2}$$

$$\gamma_1 = \frac{d_0}{d_2}$$

d_2 is the depth of column between fillets

d_0 is the diameter of hole

$t_{w,2}$ is the thickness of the column web

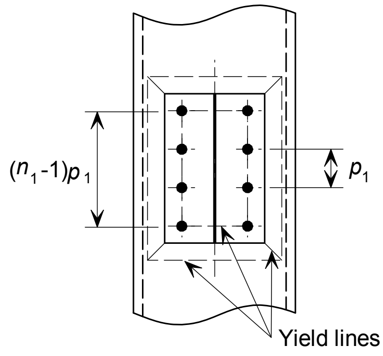
n_1 is the number of bolt rows

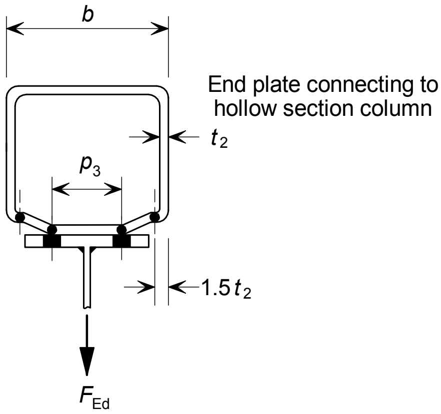
Note:

This check is required for either single-sided connections to the column web or unequally loaded double-sided connections to the column web.

If the beam is connected to a column flange, the tying resistance of the column flange may be assessed using the procedures given in Reference 16.

CHECK 15	Tying resistance – supporting column wall (hollow section)





Resistance of hollow section wall with axial compression in the column.

Basic requirement:

$F_{\mathrm{Ed}} \leq F_{\mathrm{Rd,u}}$

$$F_{\mathrm{Rd,u}} = \frac{8M_{\mathrm{pl,Rd,u}}}{1-\beta_1}\left[\eta_1 + 1.5\left(1-\beta_1\right)^{0.5}\left(1-\gamma_1\right)^{0.5}\right]$$

$$M_{\mathrm{pl,Rd,u}} = \frac{f_{\mathrm{u,2}}\,t_2^{\,2}}{4\gamma_{\mathrm{M,u}}}$$

The factor 1.5 in the equation for $F_{\mathrm{Rd,u}}$ includes an allowance for the axial compression in the column.

where:

$$\eta_1 = \frac{\left(n_1-1\right)p_1 - \frac{n_1}{2}d_0}{\left(b-3t_2\right)}$$

$$\beta_1 = \frac{p_3}{\left(b-3t_2\right)}$$

$$\gamma_1 = \frac{d_0}{\left(b-3t_2\right)}$$

t_2 is the thickness of the RHS wall

d_0 is the diameter of hole in RHS (the bolt diameter for Flowdrill or the hole diameter given from Table G.69 of the yellow pages for Hollo-Bolts)

n_1 is the number of rows of bolts

See Appendix B for more details of prying effects.

4.8 WORKED EXAMPLES WITH FULL DEPTH END PLATES

The worked examples show design calculations for typical standard connections. Each example firstly demonstrates the use of the resistance 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.

Checks 11 to 15 deal with tying resistance. The tying force should be determined from BS EN 1991-1-7 based on the Class of building.

Example 1

Example 1 covers design checks for a one sided beam to column connection using a flush end plate.

Example 2

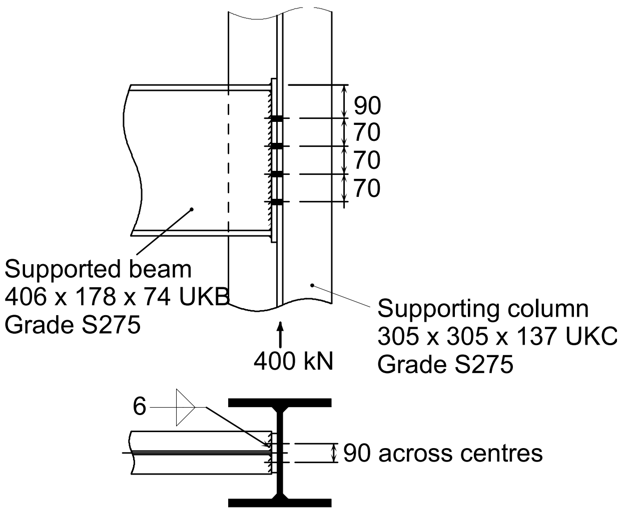
Example 2 demonstrates the additional design checks required when a beam to column web connection must be verified.

SCI Steel Knowledge CALCULATION SHEET BCSA	Job	*Joints in Steel Construction – Simple Joints*		*Sheet 1 of 5*
	Title	*Example 1 – Full depth end plate – Beam to column*		
	Client	*Connections Group*		
	Calcs by *CZT*	Checked by *ENM*	Date *Sept 2011*	

DESIGN EXAMPLE 1

Check the following beam to column joint for the design forces shown.

Yellow pages are used for the initial selection of the connection detail.

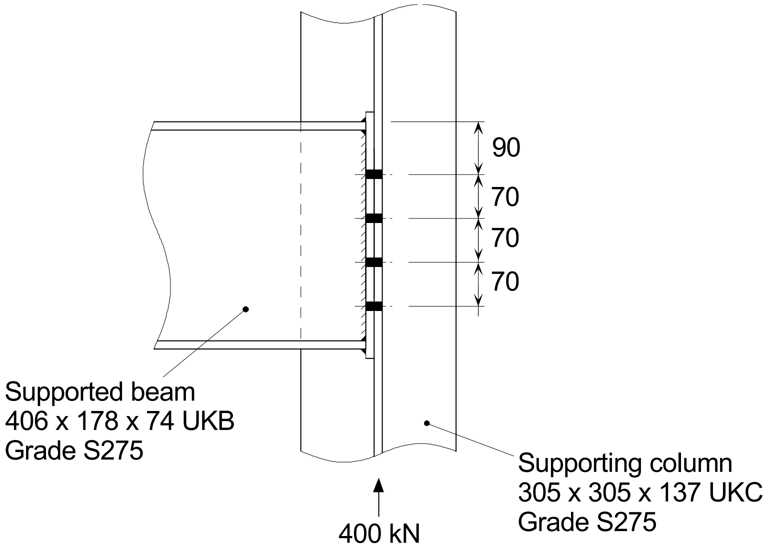


Design Information:

Bolts:	M20 8.8 @ 90 c/c	Table G.11
End plates:	150 × 10	
Web welds:	6 mm leg length fillet weld	
Flange welds:	6 mm leg length fillet weld (assumed)	
Material:	All S275	

Title Example 1 – Full depth end plate – Beam to Column *Sheet 2 of 5*

JOINT DESIGN USING RESISTANCE TABLES



406 × 178 × 74 UKB, S275

End plate:	150 × 10
Web welds:	6 mm leg length fillet weld
Flange welds:	6 mm leg length fillet weld (assumed)
Bolts:	M20 8.8
	@ 90 cross centres
	4 rows of bolts

From Table G.11 in Yellow pages

Connection shear resistance = 602 kN > 400 kN

Web thickness of supporting column = 13.8 mm

Minimum support thickness = 5.7 mm < 13.8 mm

Joint is adequate

Title Example 1 – Full depth end plate – Beam to Column *Sheet 3 of 5*

SUMMARY OF FULL DESIGN CHECKS FOR EXAMPLE 1

Sheet No.	CHECK		406 UKB (S275)		305 UKC (S275)	
			Resistance	Design force	Resistance	Design force
4	***Check 1*** Recommended detailing practice		All recommendations adopted			
4	***Check 2*** Supported beam Welds	Resistance of fillet welds (kN)	Full strength welds provided to web			
	Check 3		Not applicable			
4	***Check 4*** Supported beam Web in shear	Shear resistance (kN)	664	400	Not applicable	
	Checks 5, 6, 7	-	Not applicable			
4	***Check 8*** Supporting beam Bolt group	Bolt group resistance (kN)	602	400	Not applicable	
	Check 9		Not applicable			
5	***Check 10*** Supporting beam Shear and bearing	Shear and bearing resistance (kN)	Not applicable		749	400

CONNECTION DESIGN FOLLOWING THE DESIGN PROCEDURES

Check 1: Recommended detailing practice

End plate: 150 × 10 mm thick

Web bolts: 20 mm diameter at 90 mm cross centres

Check 2: Supported beam – Welds

Resistance of fillet welds to connecting end plate to beam web.

Web welds are designed as full strength

Basic requirement: $a \geq 0.40\ t_{w,b1}$ for S275 supported beam

$0.40 \times 9.5 = 3.8$ mm

$a = 4.24$ mm ≥ 3.8 mm **∴ OK**

Check 4: Supported beam – Web in shear

Shear resistance of beam web at the end plate

Basic requirement: $V_{Ed} \leq V_{c,Rd}$

$$V_{c,Rd} = A_v \frac{f_{y,b1}}{\sqrt{3}\gamma_{M0}}$$

Shear area of beam web:

$A_v = A_{b1} - 2b_{b1}t_{f,b1} + (t_{w,b1} + 2r_{b1})\ t_{f,b1}$

$A_v = 9450 - 2 \times 179.5 \times 16 + (9.5 + 2 \times 10.2) \times 16 = 4184\ \text{mm}^2$

$t_{f,b1} = 16$ mm, hence $f_{y,b1} = 275\ \text{N/mm}^2$

$$\therefore\ V_{c,Rd} = 4184 \times \frac{275}{\sqrt{3} \times 1.0} \times 10^{-3} = 664\ \text{kN}$$

$V_{Ed} = 400$ kN $<$ 664 kN **∴OK**

Check 8: Connection – Bolt group

Basic requirement: $V_{Ed} \leq F_{Rd}$

Shear resistance of a single bolt:

$$F_{v,Rd} = \frac{\alpha_v f_{ub} A}{\gamma_{M2}}$$

For M20 8.8 bolts:

$$F_{v,Rd} = \frac{0.6 \times 800 \times 245}{1.25} \times 10^{-3} = 94\ \text{kN}$$

Bearing resistance of a single bolt:

$F_{b,Rd} = \min(F_{b,Rd,p};\ F_{b,Rd,2})$

$$F_{b,Rd,p} = \frac{k_{1,p}\alpha_{b,p} f_{u,p} d t_p}{\gamma_{M2}}$$

$$F_{b,Rd,2} = \frac{k_{1,2}\alpha_{b,2} f_{u,2} d t_2}{\gamma_{M2}}$$

Bearing on the end plate:

$$k_{1,p} = \min\left(2.8\frac{e_2}{d_0} - 1.7;\quad 2.5\right) = \min\left(2.8 \times \frac{30}{22} - 1.7;\quad 2.5\right) = \min(2.12;\quad 2.5)$$

$\therefore\ k_{1,p} = 2.12$

Title Example 1 – Full depth end plate – Beam to column | Sheet 5 of 5

$\alpha_{b,p} = \min\left(\dfrac{p_1}{3d_0} - \dfrac{1}{4};\ \dfrac{f_{ub}}{f_{u,p}};\ 1.0\right) = \min\left(\dfrac{70}{3\times 22} - 0.25;\ \dfrac{800}{410};\ 1.0\right) = \min(0.81;\ 1.95;\ 1.0)$

$\therefore\ \alpha_{b,p} = 0.81$

$F_{b,Rd,p} = \dfrac{2.12\times 0.81\times 410\times 20\times 10}{1.25}\times 10^{-3} = 113$ kN

Bearing on the supporting column:

The thickness of the web of the supporting column is thicker than the end plate, the edge and end distances and bolt spacings are equal or larger than those in the end plate and the yield strength of the supporting column is the same as the yield strength in the end plate. Therefore the bearing resistance of the supporting column will be greater than that obtained for the end plate.

$0.8\ F_{v,Rd} = 0.8 \times 94 = 75.2$ kN

$\therefore\ F_{b,Rd} = 113$ kN

$0.8F_{v,Rd} < F_{b,Rd}$

therefore

$F_{Rd} = 0.8\ n\ F_{v,Rd}$

$\therefore\ F_{Rd} = 0.8 \times 94 \times 8 = 602$ kN

$\therefore\ V_{Ed} = 400$ kN $<$ 602 kN $\qquad \therefore$ **OK**

Check 10: Supporting column – Shear and bearing

Local shear resistance of column web supporting the beam:

Basic requirement: $V_{Ed}/2 \leq V_{Rd,min}$

Shear resistance of supporting beam web:

$V_{Rd,min} = \min\left(\dfrac{A_v f_{y,2}}{\sqrt{3}\gamma_{M0}};\ \dfrac{A_{v,net} f_{u,2}}{\sqrt{3}\gamma_{M2}}\right)$

Shear area of gross section:

$A_v = (e_t + (n_1 - 1) \times p_1 + e_b) \times t_2$

$e_t = \min(e_{1,t};\ 5d) = 5 \times 20 = 100$ mm

Since the connection is not near the top of the column, $e_{1,t}$ is not applicable

$e_b = \min(p_3/2;\ 5d) = \min(90/2, 5 \times 20) = 45$ mm

$A_v = (100 + (4 - 1) \times 70 + 45) \times 13.8 = 4899$ mm^2

$t_{f,c} = 21.7$ mm, hence $f_{y,b1} = 265$ N/mm^2

Therefore the shear resistance of the gross section is:

$\dfrac{A_v f_{y,2}}{\sqrt{3}\gamma_{M0}} = \dfrac{4899\times 265}{\sqrt{3}\times 1.0}\times 10^{-3} = 749$ kN

Shear area of net section:

$A_{v,net} = A_v - n_1 d_0 t_2$

$A_{v,net} = 4899 - 4 \times 22 \times 13.8 = 3685$ mm^2

Therefore the shear resistance of the net section is:

$\dfrac{A_{v,net} f_{u,2}}{\sqrt{3}\gamma_{M2}} = \dfrac{3685\times 410}{\sqrt{3}\times 1.1}\times 10^{-3} = 793$ kN

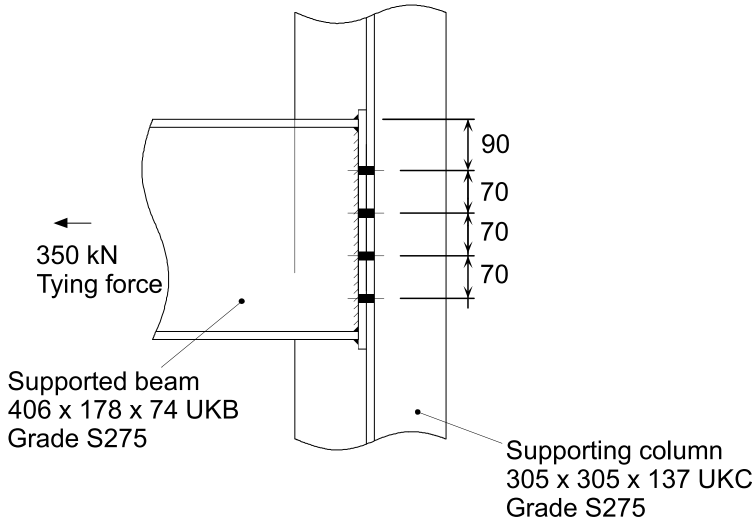
$\therefore\ V_{Rd,min} = \min(749;\ 793) = 749$ kN

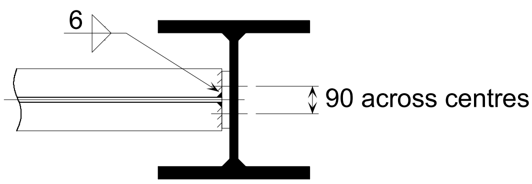
$V_{Ed}/2 = 400/2 = 200$ kN $<$ 749 kN $\qquad \therefore$ **OK**

SCI Steel Knowledge **CALCULATION SHEET** BCSA	Job	*Joints in Steel Construction – Simple Joints*	*Sheet 1 of 6*
	Title	*Example 2 – Full depth end plate – Beam to column – Tying resistance*	
	Client	*Connections Group*	
	Calcs by *CZT*	Checked by *ENM*	Date *Sept 2011*

DESIGN EXAMPLE 2

Check the following beam to column connection for the tying force shown.



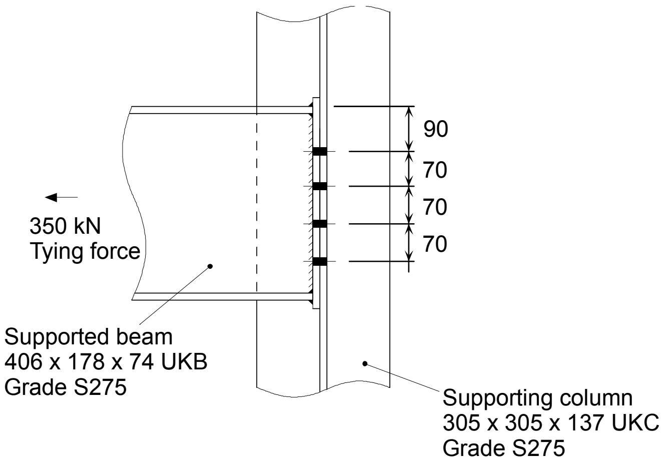


Design Information:

Bolts:	M20 8.8 @ 90 c/c
End plates:	150 × 10
Web welds:	6 mm fillet
Flange welds:	6 mm fillet assumed – see check 13
Material:	All S275

Title Example 2 – Full depth end plate – Beam to Column *Sheet 2 of 6*

CONNECTION DESIGN USING RESISTANCE TABLES



End plate: 150 × 10

Bolts: M20 8.8 @ at 90 c/c

4 rows of bolts

Weld: 6 mm fillet web weld

6 mm fillet flange weld assumed – see check 13

From Resistance Table G.11

Connection tying resistance = 515 kN

Tie force = 350 kN < 515 kN

The beam side of the connection is adequate

Notes:

(1) The tying resistance of the connection given in the tables in the yellow pages is the least of the values obtained from Checks 11 & 12.

(2) The flange weld size must be determined by calculation, or a full strength weld provided.

(3) The resistance of the column web must also be checked as shown in Check 14.

Title Example 2 – Full depth end plate – Beam to Column *Sheet 3 of 6*

SUMMARY OF FULL DESIGN CHECKS FOR EXAMPLE 2

Sheet No.	CHECK	406 UKB (S275)		305 UKC (S275)	
		Resistance	Design force	Resistance	Design force
4	***Check 11*** Tying resistance Plate and bolts (kN)	517	350	Not applicable	
5	***Check 12*** Tying resistance Supported beam web (kN)	1348	350	Not applicable	
5	***Check 13*** Tying resistance Welds (kN)	293	175	Not applicable	
6	***Check 14*** Tying resistance Supporting column web (kN)	Not applicable		404	350

Title *Example 2 – Full depth end plate – Beam to column –Tying resistance*	*Sheet 4 of 6*

CONNECTION DESIGN FOLLOWING THE DESIGN PROCEDURES

Check 11: Tying resistance – Plate and bolts

Resistance of end plate

Basic requirement: $F_{\mathrm{Ed}} \leq \min (F_{\mathrm{Rd,u,1}}; F_{\mathrm{Rd,u,2}}; F_{\mathrm{Rd,u,3}})$

Mode 1:

$$F_{\mathrm{Rd,u,1}} = \frac{\left(8n - 2e_{\mathrm{w}}\right) M_{\mathrm{pl,Rd,u}}}{2mn - e_{\mathrm{w}}\left(m + n\right)}$$

Force assumed in flange = 350 / 2 = 175 kN
Length of weld around flange is approximately 2 × 180 = 360 mm
Load / mm = 175 / 360 = 0.49 kN/mm
From Table G.64, a 6 mm fillet weld will be adequate (1.15 kN/mm) — Table G.64
Therefore, assuming a 6 mm fillet weld to the flange,

$m_{1,\mathrm{t}} = 90 - t_{\mathrm{f,b1}} - 0.8 \times s = 90 - 16 - 0.8 \times 6 = 69.2$ mm

$$m = \frac{p_3 - t_{\mathrm{w,b1}} - 2 \times 0.8 \times a\sqrt{2}}{2} = \frac{90 - 9.5 - 2 \times 0.8 \times 4.2 \times \sqrt{2}}{2} = 35.5 \text{ mm}$$

$\ell_{\mathrm{eff}} = \ell_{\mathrm{eff,t}} + (n_1 - 1)p_1 + \ell_{\mathrm{eff,b}}$

$$\ell_{\mathrm{eff,t}} = \max\left(\alpha_{\mathrm{t}} m - \left(\frac{4m + 1.25e_2}{2}\right); \left(\frac{4m + 1.25e_2}{2}\right)\right)$$

$$\lambda_{1,\mathrm{t}} = \frac{m}{m + e_2} = \frac{35.5}{35.5 + 30} = 0.542$$

$$\lambda_{2,\mathrm{t}} = \frac{m_{1,\mathrm{t}}}{m + e_1} = \frac{69.2}{35.5 + 30} = 1.06$$

From Figure 7, $\alpha_{\mathrm{t}} = 5.2$ — Figure 7

$$\ell_{\mathrm{eff,t}} = \max\left(5.2 \times 35.5 - \left(\frac{4 \times 35.5 + 1.25 \times 30}{2}\right); \left(\frac{4 \times 35.5 + 1.25 \times 30}{2}\right)\right) = \max(95;\ 90)$$

$\therefore \ell_{\mathrm{eff,t}} = 95$ mm

$$\ell_{\mathrm{eff,b}} = \max\left(\alpha_{\mathrm{b}} m - \left(\frac{4m + 1.25e_2}{2}\right); \left(\frac{4m + 1.25e_2}{2}\right)\right)$$

$m_{1,\mathrm{b}} = h_{\mathrm{b1}} - 90 - (n_1 - 1)\, p_1 - 0.8\, a\sqrt{2} = 412.8 - 90 - (4 - 1)\, 70 - 0.8 \times 4.2 \times \sqrt{2} = 108$ mm

$$\lambda_{1,\mathrm{b}} = \frac{m}{m + e_2} = \frac{35.5}{35.5 + 30} = 0.542$$

$$\lambda_{2,\mathrm{b}} = \frac{m_{1,\mathrm{b}}}{m + e_1} = \frac{108}{35.5 + 30} = 1.65$$

From Figure 7, $\alpha_{\mathrm{b}} = 5$ — Figure 7

$$\therefore \ell_{\mathrm{eff,b}} = \max\left(5 \times 35.5 - \left(\frac{4 \times 35.5 + 1.25 \times 30}{2}\right); \left(\frac{4 \times 35.5 + 1.25 \times 30}{2}\right)\right)$$

$= \max(88;\ 90) = 90$ mm

Title *Example 2 – Full depth end plate – Beam to column –Tying resistance*	*Sheet 5 of 6*
$\therefore \ell_{eff}$ = 95 + (4 – 1) × 70 + 90 = 395 mm	
$M_{pl,Rd,u} = \dfrac{0.25\sum \ell_{eff,1} t_p^2 f_{u,p}}{\gamma_{M,u}} = \dfrac{0.25 \times 395 \times 10^2 \times 410}{1.1} \times 10^{-6} = 3.68$ kNm	
Assuming no washer is used, d_w is taken as the width across points from Table G.66	Table G.66
$e_w = \dfrac{d_w}{4} = \dfrac{33}{4} = 8.25$ mm	
$n = e_{min}$ but $\leq$ $1.25m$	
e_{min} = min (e_2, $e_{2,c}$) but $e_{2,c}$ is not relevant when the connection is to the web	
$e_{min} = e_2$ = 30 mm ; $1.25m$ = 44 mm	
n = 30 mm	
$F_{Rd,u,1} = \dfrac{(8 \times 30 - 2 \times 8.25) \times 3.68 \times 10^6}{2 \times 35.5 \times 30 - 8.25 \times (35.5 + 30)} \times 10^{-3} = 517$ kN	
Mode 2:	
$F_{Rd,u,2} = \dfrac{2M_{pl,Rd} + n\Sigma F_{t,Rd,u}}{m+n}$	
$F_{t,Rd,u} = \dfrac{k_2 f_{ub} A_s}{\gamma_{M,u}} = \dfrac{0.9 \times 800 \times 245}{1.1} \times 10^{-3} = 160$ kN	
$F_{Rd,u,2} = \left(\dfrac{2 \times 3.68 \times 10^6 + 30 \times 8 \times 160 \times 10^3}{35.5 + 30}\right) \times 10^{-3} = 699$ kN	
Mode 3:	
$F_{Rd,u,3} = \Sigma F_{t,Rd,u} = 8 \times 160 = 1280$ kN	
min ($F_{Rd,u,1}$; $F_{Rd,u,2}$; $F_{Rd,u,3}$) = min (517; 699; 1280) = 517 kN	
F_{Ed} = 350 kN < 517 kN	**∴O.K.**
Check 12: Tying resistance – Supported beam web	
Resistance of the beam web.	
Basic requirement: $F_{Ed} \leq F_{Rd,u}$	
$F_{Rd,u} = \dfrac{t_{w,b1}(h_{b1} - 2t_{f,b1})f_{u,b1}}{\gamma_{Mu}} = \dfrac{9.5 \times (412.8 - 2 \times 16) \times 410}{1.1} \times 10^{-3} = 1348$ kN	
F_{tie} = 350 kN < 1348 kN	**∴ OK**
Check 13: Tying resistance – Welds	
For the flush end plate the flange welds are designed for the applied tying force, and the web welds are designed for the applied shear force. In this example, the end plate is welded on both sides of the beam flange.	
The web welds are full strength, as verified in Example 1, Check 2	**∴ OK**

Title Example 2 – Full depth end plate – Beam to column –Tying resistance | Sheet 6 of 6

For the flange welds, 6 mm fillet welds were assumed, therefore $a = \frac{6}{\sqrt{2}} = 4.24$ mm

For S275, $\beta = 0.85$

$$f_{\mathrm{vw,d}} = \frac{410/\sqrt{3}}{0.85 \times 1.1} = 253 \text{ N/mm}^2$$

$$F_{\mathrm{Rd,flangeweld}} = 2\, f_{\mathrm{vw,d}}\, a\, (b_{\mathrm{p}} - 2s) = 2 \times 253 \times 4.2 \times (150 - 2 \times 6) \times 10^{-3} = 293 \text{ kN}$$

$$\frac{F_{\mathrm{Ed}}}{2} = \frac{350}{2} = 175 \text{ kN} < 293 \text{ kN}$$

∴ **OK**

Check 14: Tying resistance – Supporting column web

Basic requirement: $F_{\mathrm{Ed}} < F_{\mathrm{Rd}}$

$$F_{\mathrm{Rd,u}} = \frac{8M_{\mathrm{pl,Rd,u}}}{(1-\beta_1)}\left(\eta_1 + 1.5(1-\beta_1)^{0.5} \times (1-\gamma_1)^{0.5}\right)$$

$$M_{\mathrm{pl,Rd,u}} = \frac{f_{\mathrm{u,c}} t_{\mathrm{w,c}}^{\ 2}}{4\,\gamma_{\mathrm{M,u}}} = \frac{410 \times 13.8^2}{4 \times 1.1} \times 10^{-3} = 17.7 \text{ kNm/mm}$$

$$\beta_1 = \frac{p_3}{d_{\mathrm{c}}} = \frac{90}{246.7} = 0.365$$

$$\gamma_1 = \frac{d_0}{d_{\mathrm{c}}} = \frac{22}{246.7} = 0.089$$

$$\eta_1 = \frac{(n_1 - 1)p_1 - \frac{n_1}{2} d_0}{d_c} = \frac{(4-1) \times 70 - \frac{4}{2} \times 22}{246.7} = 0.673$$

$$F_{\mathrm{Rd,u}} = \frac{8 \times 17.7}{(1-0.365)} \times \left(0.673 + 1.5 \times (1-0.365)^{0.5} \times (1-0.089)^{0.5}\right) = 404 \text{ kN}$$

$F_{\mathrm{Ed}} = 350$ kN < 404 kN

∴ **OK**

5 FIN PLATES

5.1 INTRODUCTION

The fin plate connection consists of a length of plate welded in the workshop to the supporting member, to which the supported beam web is bolted on site (see Figure 5.1)

This connection is popular, as it can be one of the quickest connections to erect and overcomes the problem of shared bolts in two-sided 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 rotational capacity, a series of tests[26],[27] were undertaken to verify the design model, and thus to establish the design procedure. However, for beams deeper than 610 mm, caution must be exercised. In these cases further tests[27] have shown that additional geometrical precautions are needed, as described later in this Section.

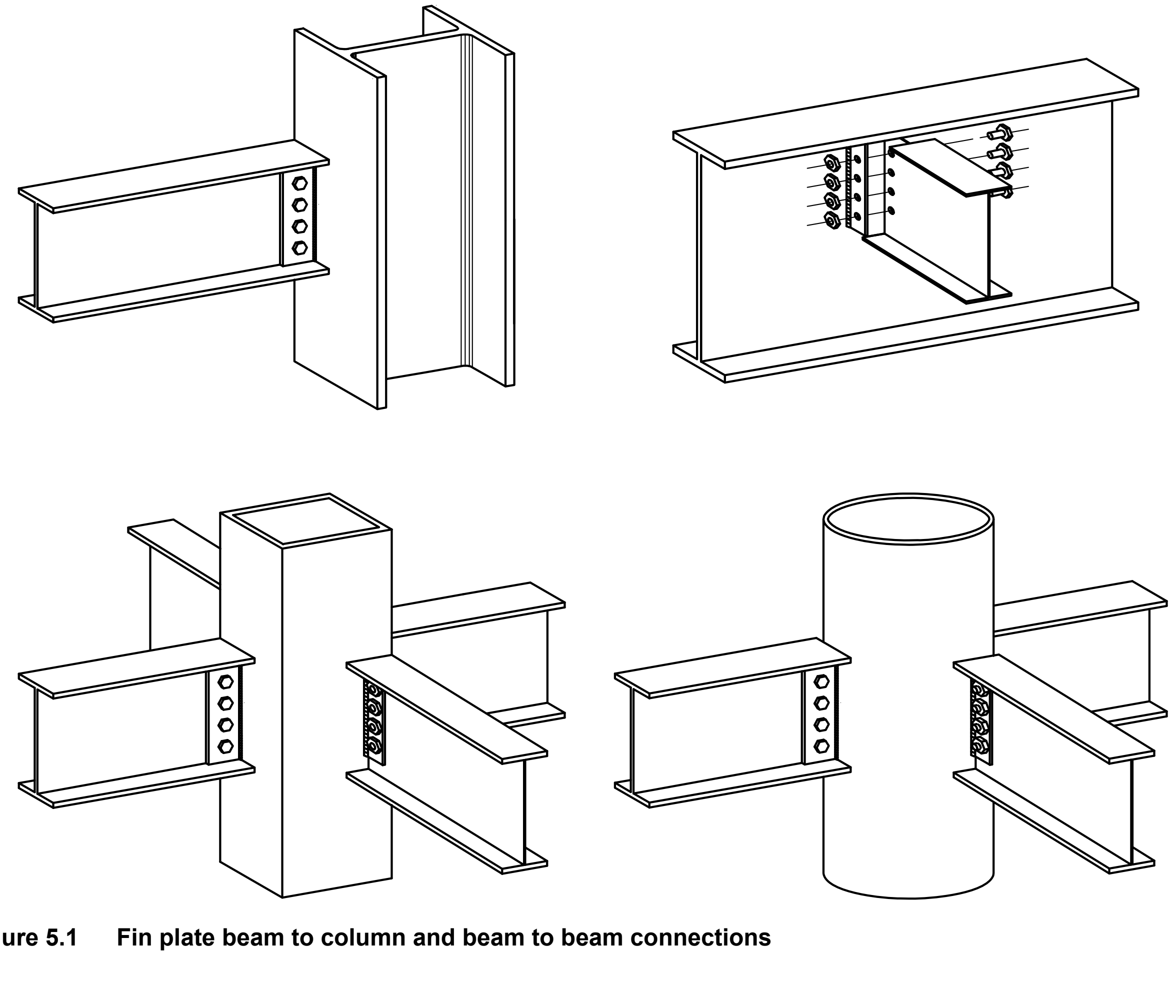


Figure 5.1 Fin plate beam to column and beam to beam connections

Rotational capacity

Fin plate connections derive their rotational capacity from:

- hole distortions in the fin plate and/or the beam web
- shear deformation of the bolts.

In the previous publication P212, components were selected such that bearing resistance of the bolts was the critical design check, thus ensuring adequate rotational capacity. With increased bearing resistance calculated according to BS EN 1993-1-8[1], bearing may no longer be the critical design check. The recommended detailing rules are most important, since it is only for such connections that adequate rotational capacity has been demonstrated by test.

5.2 PRACTICAL CONSIDERATIONS

A fin plate is usually made by cropping and punching. With 10 mm thick plates in S275 steel, 8 mm fillet welds to the supporting member will guard against any possibility of weld failure.

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 centre line of the supporting member and the fin plate offset as shown in Figure 5.2. On site it is not always evident which side of the fin plate the beam should be connected to and so a consistent system for setting out should be established during detailing. Alternatively, the contact face of the fin plate should be marked.

Like the end plate joint, there is little facility for site adjustment, and care must be taken with continuous runs of beams.

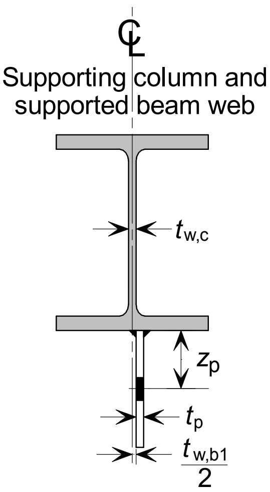


Figure 5.2 Setting out for fin plate

Short and long fin plates

Fin plates may be classified as short or long as follows:

Short, $\frac{t_p}{z_p} \geq 0.15$ Long, $\frac{t_p}{z_p} < 0.15$

Where 'z_p' is the distance between the face of the support and the first line of bolts.

If a short fin plate is used for connections to column webs, erection can be difficult. In these situations it is common to remove one half of the bottom flange of the beam, to allow the beam to be lowered into position, as shown in Figure 5.3.

Long fin plates may be used, in which case the fin plate must be checked for lateral torsional buckling.

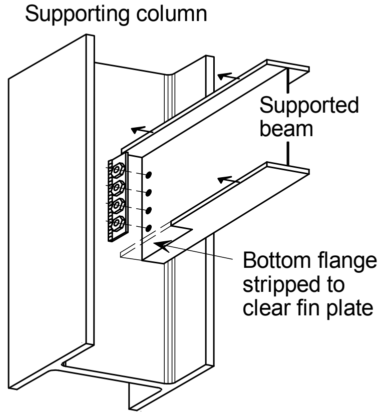


Figure 5.3 Detail to facilitate erection

If the beam is laterally unrestrained, long fin plates can behave in an extremely complex way. Long fin plates should not be used with beams that are laterally unrestrained in the permanent condition unless satisfactory performance is demonstrated by tests.

5.3 RECOMMENDED GEOMETRY

The recommendations which follow establish details that are intended to deliver the necessary flexibility and rotational capacity for the joint to behave as nominally pinned.

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

(1) the fin plate is positioned close to the top flange, in order to provide positional restraint
(2) the fin plate depth is at least 0.6 times the supported beam depth, in order to provide the beam with adequate torsional restraint
(3) the thickness of the fin plate or the beam web is: $\leq 0.42d$ (for S355 steel) or
$\leq 0.50d$ (for S275 steel)
(4) Property class 8.8 bolts are used, non-preloaded, in clearance holes
(5) all end and edge distances on the plate and the beam web are at least $2d$
(6) full strength fillet welds are provided (see Check 8)

The first two requirements ensure that in those cases where the beam is laterally unrestrained, the beam can be designed with an effective length of $1.0L$.

The last four requirements ensure the connection provides the necessary rotational capacity.

These requirements, together with the standard geometry presented in Section 2, have been used to create the standard connections shown in Figure 5.4.

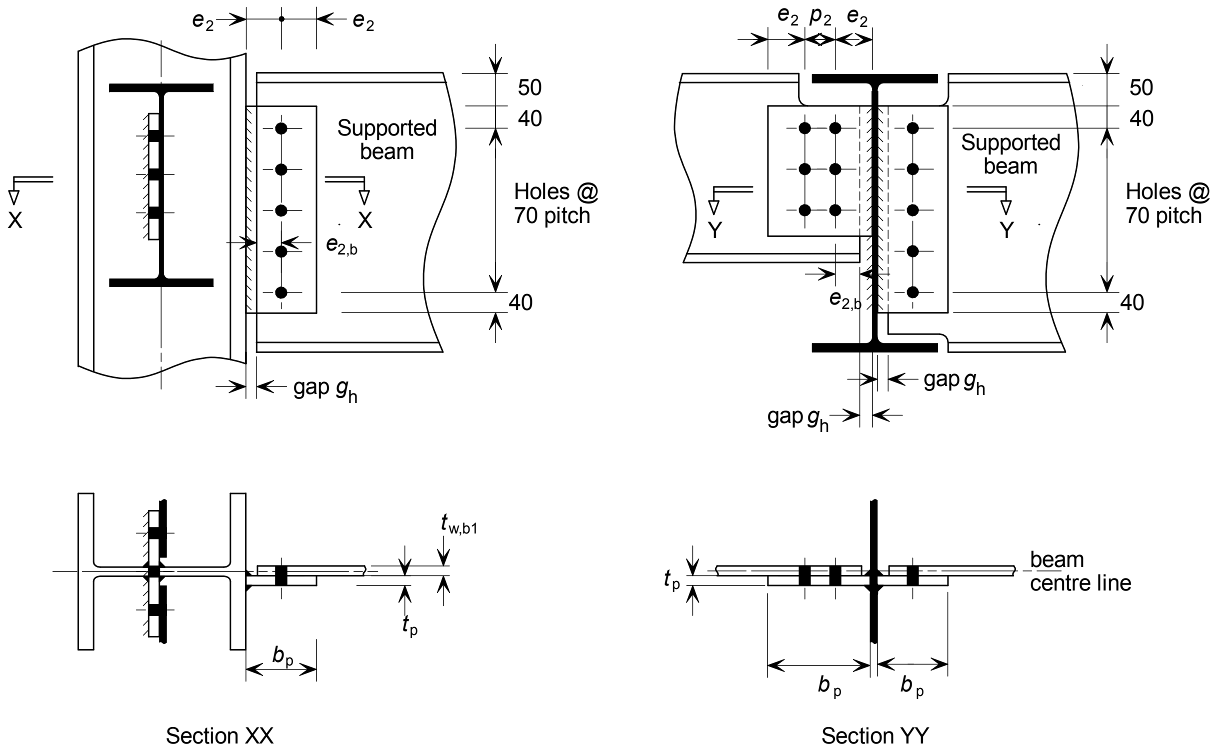


Figure 5.4 Standard fin plate connection

Table 5.1 Standard fin plate connection details

Supported beam nominal depth	Vertical bolt lines n_2	Recommended fin plate size	Horizontal bolt spacing, e_2/e_2 or $e_2/p_2/e_2$	Gap, g_h
mm		mm	mm	mm
≤ 610	1	100 × 10	50/50	10
> 610*	1	120 × 10	60/60	20
≤ 610	2	160 × 10	50/60/50	10
> 610*	2	180 × 10	60/60/60	20

Bolts:	M20 8.8 in 22 mm diameter holes
Plate:	S275 steel, minimum length $0.6h_{b1}$ where h_{b1} is the depth of the supported beam
Weld:	Two 8 mm fillets for 10 mm thick plates

* For beams over 610 mm nominal depth the span to depth ratio of beam should not exceed 20 and the vertical distance between extreme bolts should not exceed 530 mm

Fin plate connections outside these geometric recommendations, or with different grade of plate or property class of bolt, may not behave as nominally pinned.

5.4 DESIGN

The full design procedure is presented in Section 5.5.

With a single vertical line of bolts, the connection shear resistance will be in the range of 25% to 50% of the beam shear resistance. Using two vertical lines of bolts does increase the resistance, but as the eccentricity of the load also increases, the benefit does not double and the best that can be achieved is around 75% of the beam shear resistance.

Support flexibility

For the design of the fin plate, weld and bolts, it could be considered that the line of action for the load is either on the bolt line or where the fin plate is supported.

If the support is flexible, the line of action is considered to be at the weld line. In this case the bolt group should be designed for the shear force and the bending moment $V_{Ed}z$.

If the support is rigid, the line of action is considered to be at the bolt line. In this case the weld should be designed for the shear force and the bending moment $V_{Ed}z$.

In this publication, both situations are covered. The bolt groups are checked for the shear force and the bending moment, whilst the welds are sized to be full strength.

Stiffening

It is possible to improve the performance of a long fin plate by providing some stiffening. This can prevent lateral torsional buckling of the plates. Possible arrangements are shown in Figure 5.5 and Figure 5.6.

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.

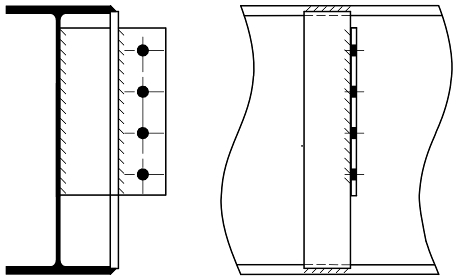


Figure 5.5 Possible stiffening arrangement for long fin plates to supporting beam

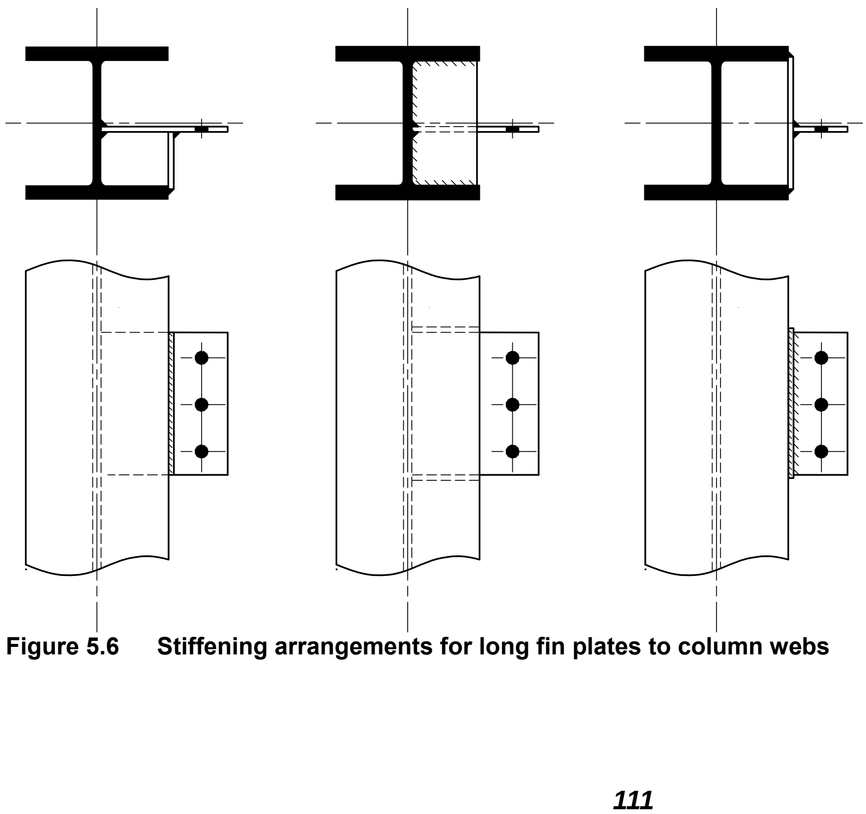


Figure 5.6 Stiffening arrangements for long fin plates to column webs

Structural integrity

As noted in Section 1.2, BS EN 1991-1-7[4] defines horizontal and vertical tying, used to ensure a minimum level of robustness and to prevent disproportionate collapse in the event of an accidental action affecting the structure. When these tying forces are carried by the primary structure, which is often the case, the connection must be designed to carry an axial tension, which varies in magnitude with the Class of the building.

Fin plate connections detailed in accordance with Figure 5.4 will be found to have tying resistances which generally exceed their shear resistance, and in this respect no further checks will normally be required for the plate, weld or beam. Local checks to the column, particularly in one-sided beam to column web connections or connections to hollow section columns, must be carried out (see Checks 14 to 16).

In checking the appropriateness of a connection to resist tying forces, it should be noted that the tying forces are ignored in checks for vertical reactions and similarly, vertical forces are ignored in checks for tying forces, so tying forces and vertical forces are never considered acting simultaneously.

Worked examples

Four worked examples are provided in Section 5.6 to illustrate the full set of design checks of Section 5.5.

Resistance tables

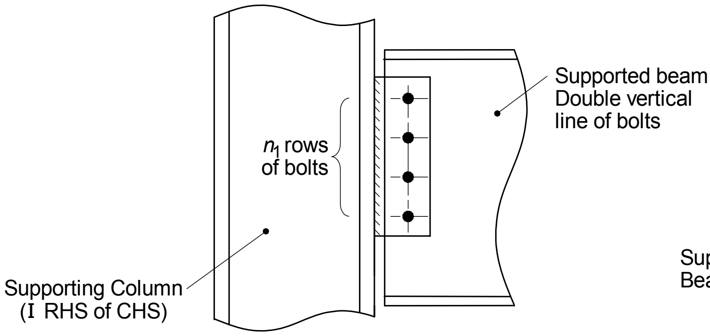
Resistance tables for fin plate connections with one and two vertical rows of bolts, using grade S275 and S355 beams, detailed in accordance with the standard geometry presented in Figure 5.4, are included in the yellow pages.

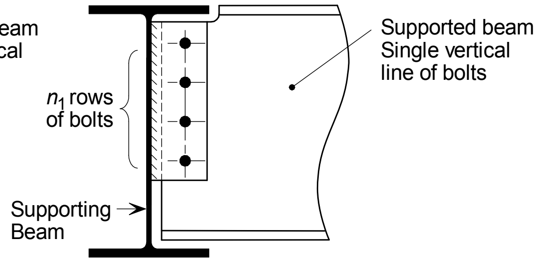
5.5 DESIGN PROCEDURES

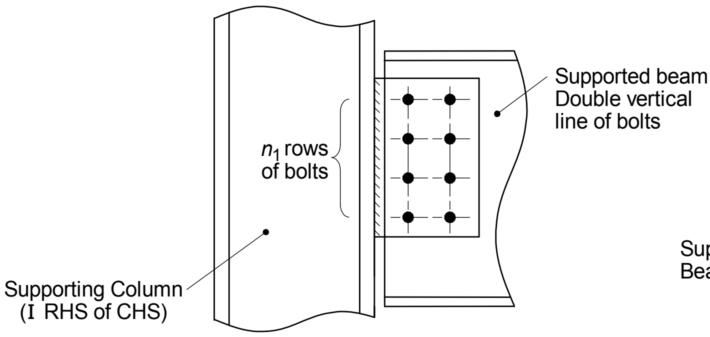
The design procedures apply to beams connected either to the column flange, the column web, the supporting beam web, or to hollow section columns.

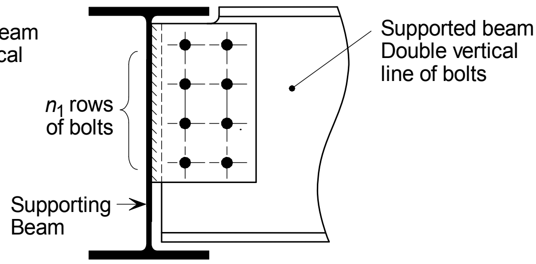
For connections to the column web the c_w/t_w ratio of the column should be limited to 40ε for I sections, SHS and RHS. d/t should be limited to $70\varepsilon^2$ for CHS, where:

$$\varepsilon = \sqrt{\frac{235}{f_{y,2}}}$$



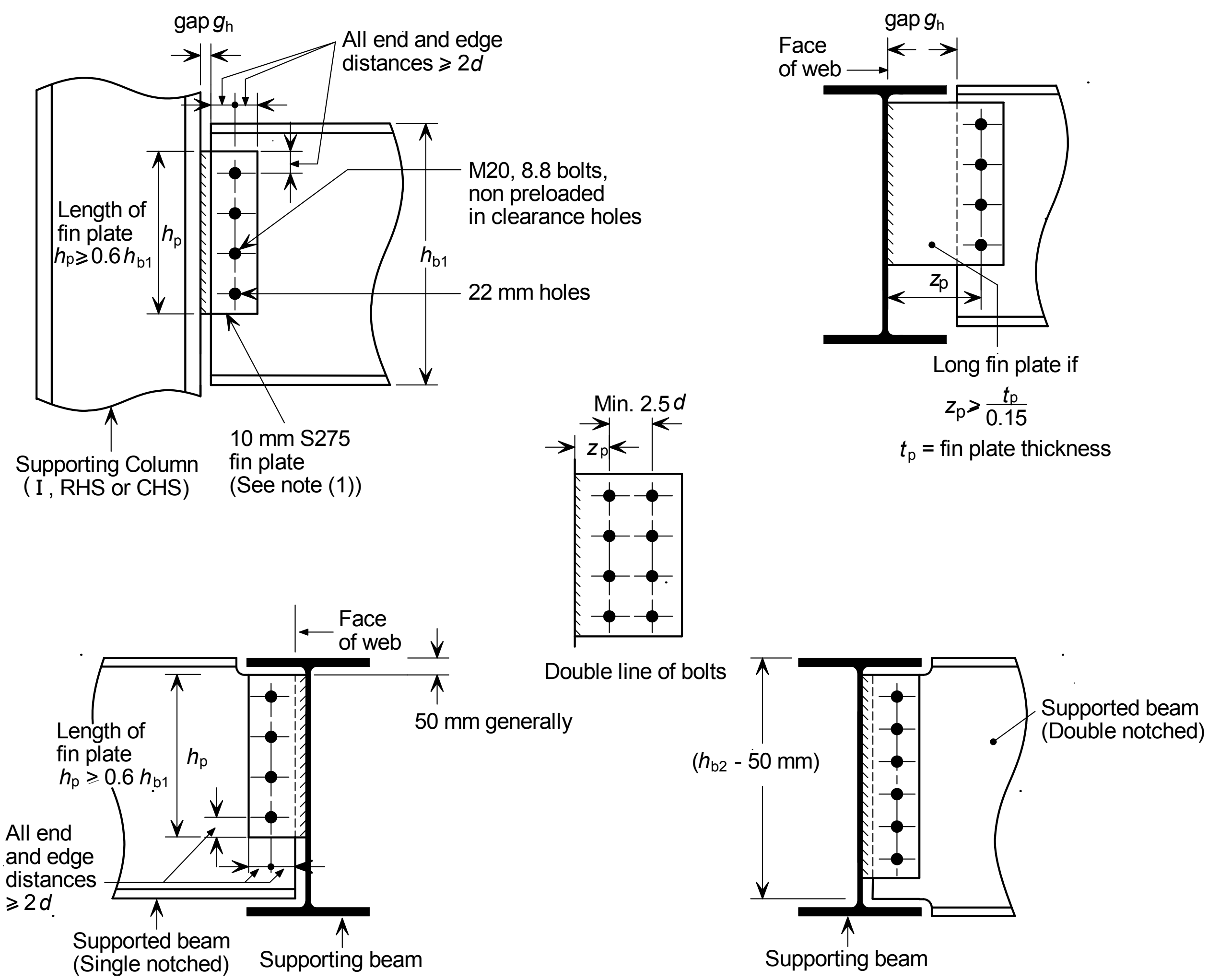






Check 1	Recommended detailing practice		
Check 2	Supported beam	–	Bolt group
Check 3	Supported beam	–	Fin plate
Check 4	Supported beam	–	Web in shear
Check 5	Supported beam	–	Resistance at a notch
Check 6	Supported beam	–	Local stability of notched beam
Check 7	Unrestrained supported beam	–	Overall stability of notched beam
Check 8	Supporting beam/column	–	Welds
Check 9	*Not applicable*		
Check 10	Supporting beam/column	–	Shear and bearing
Check 11	Tying resistance	–	Plate and bolts
Check 12	Tying resistance	–	Supported beam web
Check 13	Tying resistance	–	Welds
Check 14	Tying resistance	–	Supporting column web (UKC or UKB)
Check 15	Tying resistance	–	Supporting column wall (RHS)
Check 16	Tying resistance	–	Supporting column wall (CHS)

CHECK 1	Recommended detailing practice



Notes:

(1) The fin plate is generally positioned close to the top flange of the beam to provide adequate positional restraint. Its height should be at least 0.6 h_{b1} to give adequate "nominal torsional restraint".

(2) For supported beams exceeding a nominal depth of 610 mm, the design method given here may only be used when the following three conditions are all met:

- Supported beam Span/Depth ≤ 20
- Gap $g_h \geq$ 20 mm
- Vertical distance between extreme bolts $(n_1\text{-}1)p_1 \leq$ 530 mm

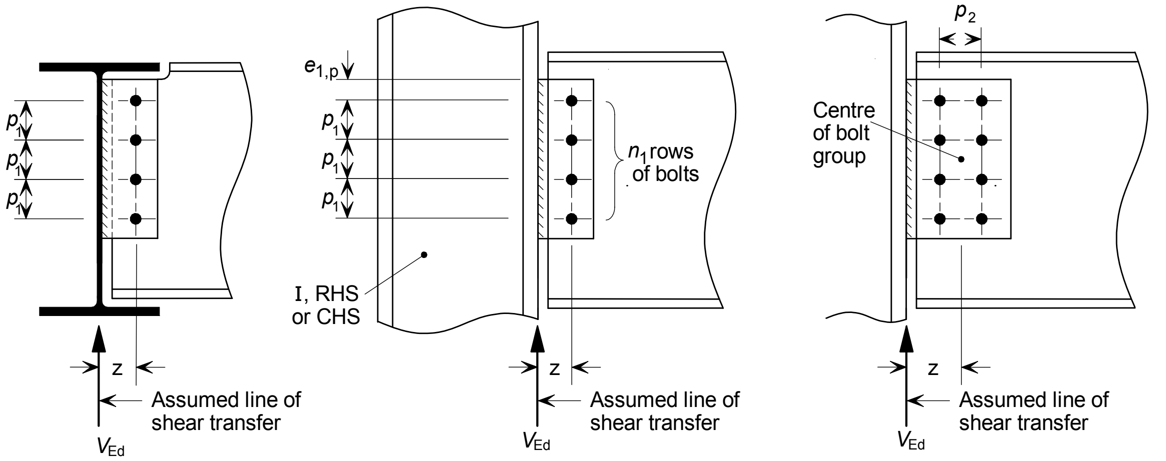
(3) Bolt spacing and edge distances should comply with the requirements of BS EN 1993-1-8.

(4) Detailing is similar for long fin plates (i.e. the fin plate thickness t_p is less than $0.15z_p$) except the gap g_h will be considerably greater.

In beam to column flanges of I sections connections designed for a tie force of 75 kN, the connection must have at least 2 No. M20, 8.8 bolts and the fin plate thickness ≥ 6 mm.

For greater tie forces, Checks 11 to 16, as appropriate, should be carried out.

CHECK 2	Supported beam – Bolt group



Shear and bearing resistance of bolt group on fin plate and supported beam web (taking account of eccentricity "z").

Bolt shear

Basic requirement:

$V_{Ed} \leq V_{Rd}$

$$V_{Rd} = \frac{nF_{v,Rd}}{\sqrt{(1+\alpha n)^2 + (\beta n)^2}}$$

$$F_{v,Rd} = \frac{\alpha_v f_{ub} A}{\gamma_{M2}}$$

The expression for V_{Rd} is taken from Access-steel resource SN017[28]

where:

n is the total number of bolts

$= n_1 \times n_2$

For a single vertical line of bolts ($n_2 = 1$)

$\alpha = 0$

$$\beta = \frac{6z}{n_1(n_1+1)p_1}$$

For two vertical lines of bolts ($n_2 = 2$)

$$\alpha = \frac{zp_2}{2I}$$

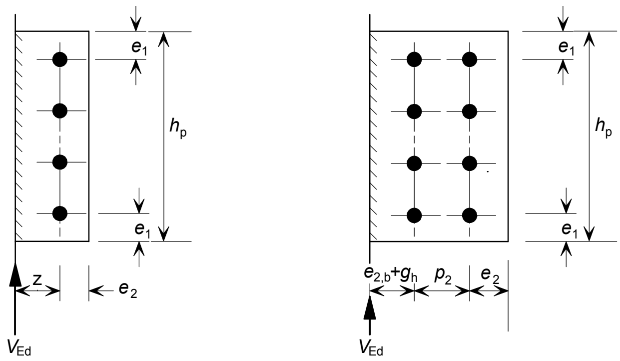
$$\beta = \frac{zp_1}{2I}(n_1 - 1)$$

$$I = \frac{n_1}{2}p_2^2 + \frac{1}{6}n_1(n_1^2 - 1)p_1^2$$

α_v = 0.6 for 8.8 bolts

γ_{M2} is the partial factor for resistance of bolts (γ_{M2} = 1.25 as given in the UK National Annex to BS EN 1993-1-8)

CHECK 2 **(continued)**	**Supported beam – Bolt group**



Bolt bearing in the fin plate

Basic requirement:

$V_{Ed} \leq V_{Rd}$

$$V_{Rd} = \frac{n}{\sqrt{\left(\frac{1+\alpha n}{F_{b,ver,Rd}}\right)^2 + \left(\frac{\beta n}{F_{b,hor,Rd}}\right)^2}}$$

$F_{b,Rd}$ is the bearing resistance of a single bolt defined as:

$$F_{b,Rd} = \frac{k_1 \alpha_b f_{u,p} d t_p}{\gamma_{M2}}$$

$F_{b,ver,Rd}$ is the vertical bearing resistance of a single bolt on the fin plate

$F_{b,hor,Rd}$ is the horizontal bearing resistance of a single bolt on the fin plate

where:

α, β and n are as defined in Check 2(i)

d is the diameter of the bolt

For $F_{b,ver,Rd}$:

$$k_1 = \min\left(2.8\frac{e_2}{d_0} - 1.7;\quad 1.4\frac{p_2}{d_0} - 1.7;\quad 2.5\right)$$

$$\alpha_b = \min\left(\frac{e_1}{3d_0};\quad \frac{p_1}{3d_0} - \frac{1}{4};\quad \frac{f_{ub}}{f_{u,p}};\quad 1.0\right)$$

For $F_{b,hor,Rd}$:

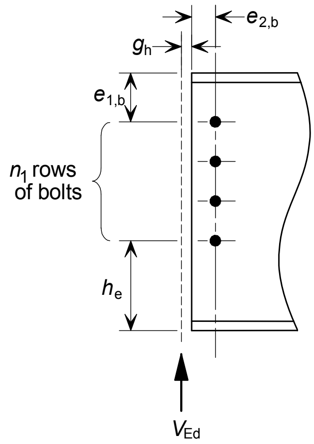
$$k_1 = \min\left(2.8\frac{e_1}{d_0} - 1.7;\quad 1.4\frac{p_1}{d_0} - 1.7;\quad 2.5\right)$$

$$\alpha_b = \min\left(\frac{e_2}{3d_0};\quad \frac{p_2}{3d_0} - \frac{1}{4};\quad \frac{f_{ub}}{f_{u,p}};\quad 1.0\right)$$

γ_{M2} is the partial factor for resistance of bolts (γ_{M2} = 1.25 as given in the UK NA to BS EN 1993-1-8)

When calculating k_1 and α_b for connections with a single vertical row of bolts (n_2 = 1), terms involving p_2 are ignored.

CHECK 2 (continued)	Supported beam – Bolt group Beam web in bearing



Bolt bearing in supported beam web

Basic requirement:

$V_{Ed} \leq V_{Rd}$

$$V_{Rd} = \frac{n}{\sqrt{\left(\frac{1+\alpha n}{F_{b,ver,Rd}}\right)^2 + \left(\frac{\beta n}{F_{b,hor,Rd}}\right)^2}}$$

$F_{b,Rd}$ is the bearing resistance of a single bolt

$$= \frac{k_1 \alpha_b f_{u,b1} d t_{w,b1}}{\gamma_{M2}}$$

$F_{b,ver,Rd}$ is the vertical bearing resistance of a single bolt on the supported beam web

$F_{b,hor,Rd}$ is the horizontal bearing resistance of a single bolt on the supported beam web

where:

α, β and n are as defined in Check 2(i)

d is the diameter of the bolt

For $F_{b,ver,Rd}$:

$$k_1 = \min\left(2.8\frac{e_{2,b}}{d_0} - 1.7; 1.4\frac{p_2}{d_0} - 1.7; 2.5\right)$$

$$\alpha_b = \min\left(\frac{e_{1,b}}{3d_0}; \frac{p_1}{3d_0} - \frac{1}{4}; \frac{f_{ub}}{f_{u,b1}}; 1.0\right)$$

For $F_{b,hor,Rd}$:

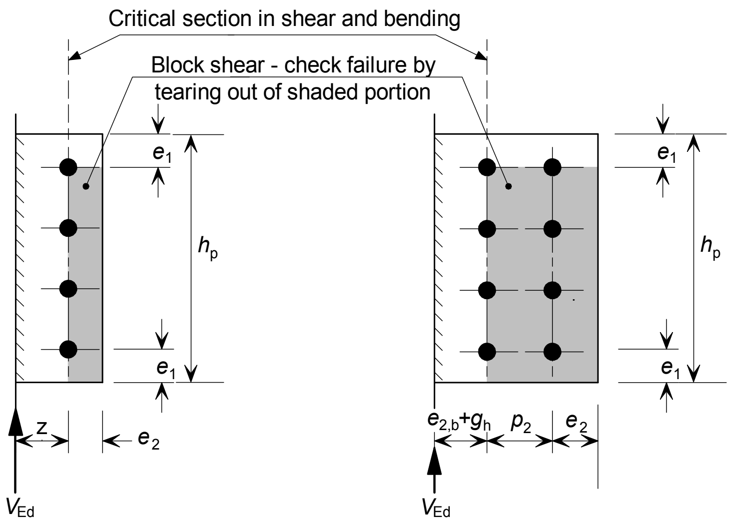
$$k_1 = \min\left(2.8\frac{e_{1,b}}{d_0} - 1.7; 1.4\frac{p_1}{d_0} - 1.7; 2.5\right)$$

$$\alpha_b = \min\left(\frac{e_{2,b}}{3d_0}; \frac{p_2}{3d_0} - \frac{1}{4}; \frac{f_{ub}}{f_{u,b1}}; 1.0\right)$$

γ_{M2} is the partial factor for resistance of bolts (γ_{M2} = 1.25 as given in the UK National Annex to BS EN 1993-1-8)

When calculating k_1 and α_b for connections with a single vertical row of bolts (n_2 = 1), terms involving p_2 are ignored.

CHECK 3	**Supported beam – Fin plate**



Shear and bending resistance of the fin plate:

Shear

Basic requirement:

$V_{Ed} \leq V_{Rd,min}$

$V_{Rd,min}$ is the shear resistance of the fin plate, calculated as the smaller of the gross section shear resistance $V_{Rd,g}$, net section shear resistance, $V_{Rd,n}$ and block shear resistance, $V_{Rd,b}$.

Fin plate in shear: gross section

$$V_{Rd,g} = \frac{h_p t_p}{1.27} \frac{f_{y,p}}{\sqrt{3}\,\gamma_{M0}}$$

The coefficient 1.27 takes into account the reduction in the shear resistance of the cross section due to the nominal moment in the connection.

Fin plate in shear: net section

$$V_{Rd,n} = A_{v,net} \frac{f_{u,p}}{\sqrt{3}\gamma_{M2}}$$

Fin plate in shear: block shear

$$V_{Rd,b} = \frac{0.5 f_{u,p} A_{nt}}{\gamma_{M2}} + \frac{f_{y,p}}{\sqrt{3}} \frac{A_{nv}}{\gamma_{M0}}$$

where:

$A_{v,net} = t_p\,(h_p - n_1 d_0)$

A_{nt} is the net area subjected to tension

For a single vertical line of bolts (n_2 = 1):

$$A_{nt} = t_p\,(e_2 - \frac{d_0}{2})$$

For two vertical lines of bolts (n_2 = 2):

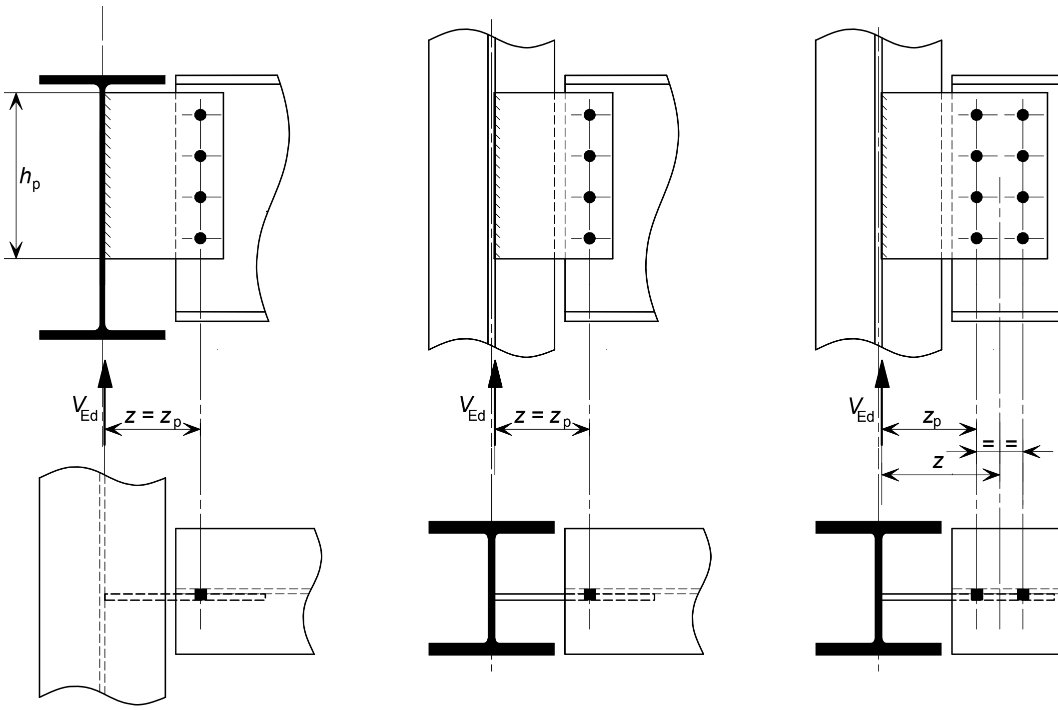
$$A_{nt} = t_p\,(p_2 + e_2 - 3\frac{d_0}{2})$$

A_{nv} is net area subjected to shear

$= t_p\,(h_p - e_1 - (n_1 - 0.5) d_0)$

γ_{M2} is the partial factor for the resistance of net sections (γ_{M2} = 1.1 as given in the UK National Annex to BS EN 1993-1-1)

CHECK 3 (continued)	**Supported beam – Fin plate**



Bending

Basic requirement:

$V_{Ed} \leq V_{Rd}$

If $h_p \geq 2.73\ z$ then $V_{Rd} = \infty$

Else $V_{Rd} = \dfrac{W_{el,p}}{z} \dfrac{f_{y,p}}{\gamma_{M0}}$

Lateral torsional buckling

Basic requirement:

$V_{Ed} \leq V_{Rd}$

If $z_p > t_p/0.15$ (long fin plate)

then $V_{Rd} = \min\left(\dfrac{W_{el,p}}{z} \dfrac{\chi_{LT} f_{y,p}}{0.6\gamma_{M1}};\ \dfrac{W_{el,p}}{z} \dfrac{f_{y,p}}{\gamma_{M0}}\right)$

The 0.6 factor in the expression for V_{Rd} accounts for the triangular shape of the assumed bending moment diagram in the fin plate.

If $z_p \leq t_p/0.15$ (short fin plate)

then $V_{Rd} = \dfrac{W_{el,p}}{z} \dfrac{f_{y,p}}{\gamma_{M0}}$

where:

$W_{el,p} = \dfrac{t_p h_p^2}{6}$

χ_{LT} is the reduction factor for lateral torsional buckling of the fin plate obtained from Table 5.2 and based on the slenderness of the fin plate, given for S275 by

$$\bar{\lambda}_{LT} = \frac{2.8}{86.8}\left(\frac{z_p \times h_p}{1.5 t_p^2}\right)^{1/2}$$

γ_{M2} is the partial factor for resistance of net sections (γ_{M2} = 1.1 as given in the UK National Annex)

$z_p = e_{2,b} + g_h$

Note:

Long fin plates should not be used with unrestrained beams without experimental evidence to justify the design.

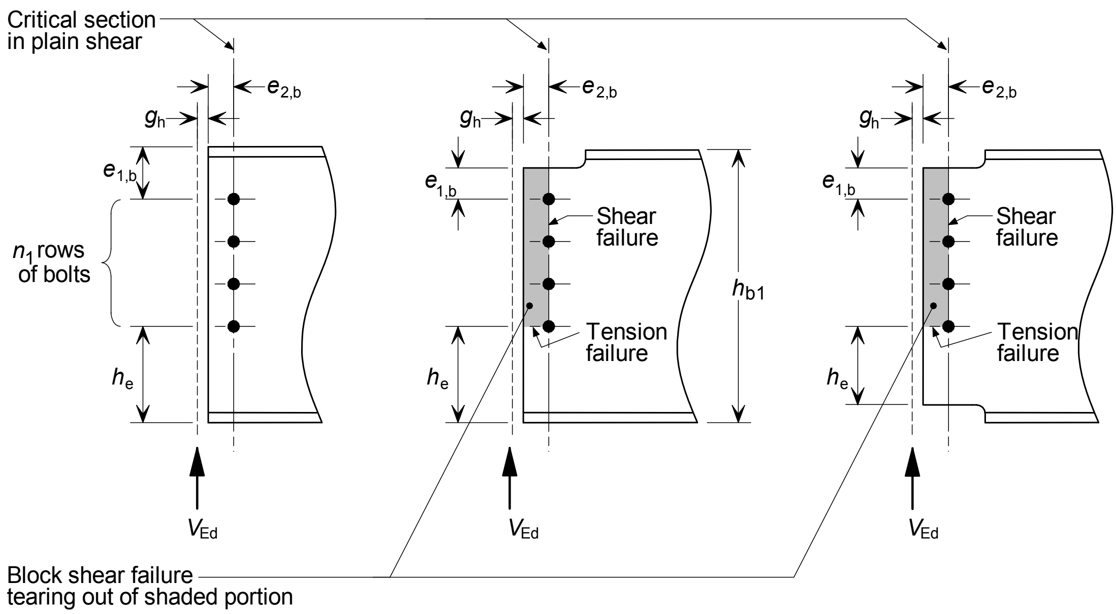
CHECK 3 **(continued)**	**Supported beam – Fin plate**

Table 5.2 Reduction factor for lateral torsional buckling, χ_{LT}

$\bar{\lambda}_{LT}$	Reduction factor, χ_{LT}	$\bar{\lambda}_{LT}$	Reduction factor, χ_{LT}
0.25	0.96	1.50	0.28
0.30	0.92	1.55	0.26
0.35	0.89	1.60	0.25
0.40	0.85	1.65	0.24
0.45	0.81	1.70	0.23
0.50	0.78	1.75	0.22
0.55	0.74	1.80	0.21
0.60	0.71	1.85	0.20
0.65	0.68	1.90	0.19
0.70	0.64	1.95	0.18
0.75	0.61	2.00	0.18
0.80	0.58	2.05	0.17
0.85	0.55	2.10	0.16
0.90	0.52	2.15	0.16
0.95	0.49	2.20	0.15
1.00	0.47	2.25	0.15
1.05	0.44	2.30	0.14
1.10	0.42	2.35	0.13
1.15	0.40	2.40	0.13
1.20	0.38	2.45	0.13
1.25	0.36	2.50	0.12
1.30	0.34	2.55	0.12
1.35	0.32	2.60	0.11
1.40	0.31	2.65	0.11
1.45	0.29	2.70	0.11

The values in Table 5.2 have been calculated using expression 6.56 of BS EN 1993-1-1 with an imperfection factor α_{LT} of 0.76

CHECK 4	Supported beam – Web in shear



Shear and bending resistance of the supported beam:

Shear

Basic requirement:

$V_{Ed} \leq V_{Rd,min}$

$V_{Rd,min}$ is the shear resistance of the supported beam web

= smaller of the gross section shear resistance $V_{Rd,g}$, net section shear resistance, $V_{Rd,n}$ and block shear resistance, $V_{Rd,b}$

Beam web in shear: gross section

$$V_{Rd,g} = V_{pl,Rd} = A_v \frac{f_{y,b1}}{\sqrt{3}\,\gamma_{M0}}$$

Beam web in shear: net section

$$V_{Rd,n} = A_{v,net} \frac{f_{u,b1}}{\sqrt{3}\gamma_{M2}}$$

Beam web in shear: block shear (applicable to notched beams only)

$$V_{Rd,b} = \frac{0.5 f_{u,b1} A_{nt}}{\gamma_{M2}} + \frac{f_{y,b1} A_{nv}}{\sqrt{3}\gamma_{M0}}$$

where:

$A_{v,net} = A_v - n_1 d_0 t_{w,b1}$

Unnotched beams:

$A_v = A_g - 2 b_{b1} t_{f,b1} + (t_{w,b1} + 2 r_{b1}) t_{f,b1}$ but $\not<$ $h_{w,b1} t_{w,b1}$

Single notched beams:

$$A_v = A_{Tee} - b_{b1} t_{f,b1} + (t_{w,b1} + 2 r_{b1}) \frac{t_{f,b1}}{2}$$

A_{Tee} is the area of the Tee section

Double notched beams:

$A_v = 0.9 t_{w,b1} (h_{b1} - d_{nt} - d_{nb})$

For a single vertical line of bolts (n_2 = 1)

$$A_{nt} = t_{w,b1} \left(e_{2,b} - \frac{d_0}{2} \right)$$

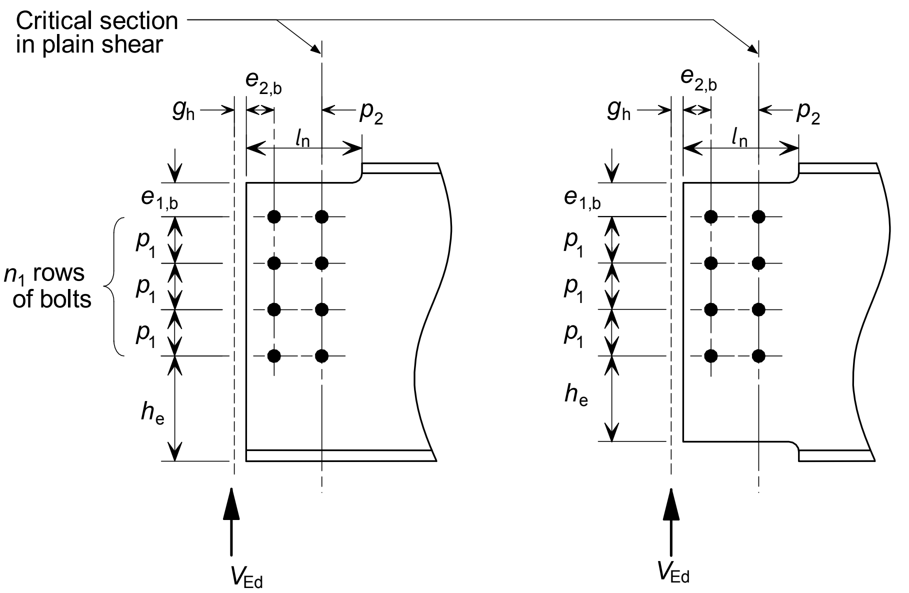
For two vertical lines of bolts (n_2 = 2)

$$A_{nt} = t_{w,b1} \left(p_2 + e_{2,b} - 3\frac{d_0}{2} \right)$$

$$A_{nv} = t_{w,b1} \left(e_{1,b} + (n_1 - 1) p_1 - (n_1 - 0.5) d_0 \right)$$

γ_{M2} is the partial factor for the resistance of net sections (γ_{M2} = 1.1 as given in the UK National Annex to BS EN 1993-1-1)

CHECK 4 (continued)	**Supported beam – Web in shear** (notched beams)



Shear and bending interaction for two lines of bolts, if the notch length $l_n > (e_{2,b} + p_2)$

Note:

Although shear and bending should be checked at both bolt lines, the check is generally only applied in connections with two rows of bolts ($n_2 = 2$), assuming that unless the geometry is extreme, it will not be critical in connections with only one row of bolts. The check is only applicable if the notch extends past the second bolt row (i.e. $l_n > (e_{2,b} + p_2)$). The beam should be checked at the end of the notch - see Check 5.

Basic requirement:

$V_{Ed}(g_h + e_{2,b} + p_2) \leq M_{c,Rd}$

$M_{c,Rd}$ is the moment resistance of the notched beam at the connection in the presence of shear

For a single notched beam

For low shear (i.e. $V_{Ed} \leq 0.5V_{pl,N,Rd}$)

$$M_{c,Rd} = \frac{f_{y,b1}W_{el,N}}{\gamma_{M0}}$$

For high shear (i.e. $V_{Ed} > 0.5V_{pl,N,Rd}$)

$$M_{c,Rd} = \frac{f_{y,b1}W_{el,N}}{\gamma_{M0}}\left(1-\left(\frac{2V_{Ed}}{V_{pl,N,Rd}}-1\right)^2\right)$$

For a double notched beam

For low shear (i.e. $V_{Ed} \leq 0.5V_{pl,DN,Rd}$)

$$M_{c,Rd} = \frac{f_{y,b1}t_{w,b1}}{6\gamma_{M0}}\left(e_{1,b}+(n_1-1)p_1+h_e\right)^2$$

For high shear (i.e. $V_{Ed} > 0.5V_{pl,DN,Rd}$)

$$M_{c,Rd} = \frac{f_{y,b1}t_{w,b1}}{6\gamma_{M0}}\left(e_{1,b}+(n_1-1)p_1+h_e\right)^2\left(1-\left(\frac{2V_{Ed}}{V_{pl,DN,Rd}}-1\right)^2\right)$$

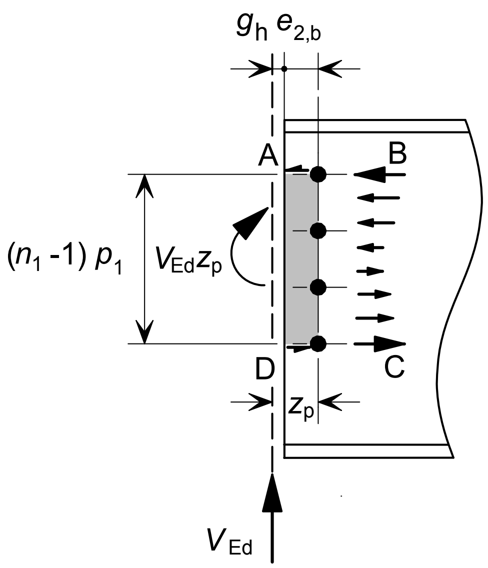
where:

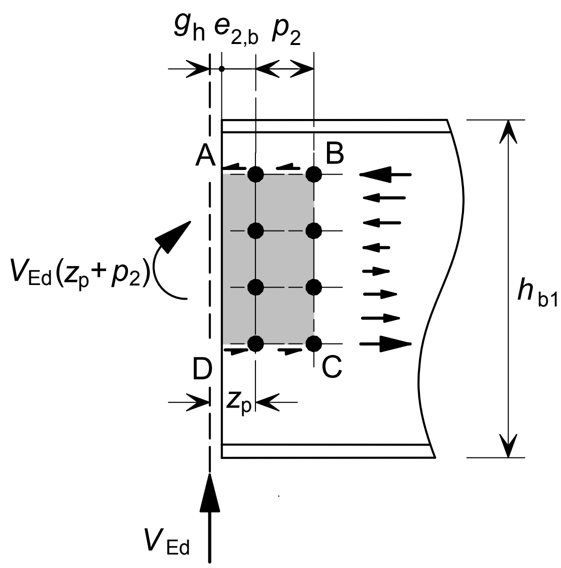
$W_{el,N}$ is the elastic section modulus of the gross tee section at the notch

$V_{pl,N,Rd}$ is the shear resistance of the single notched beam at the critical section (see figure)
= $\min(V_{Rd,g};\ V_{Rd,n})$ (see Check 4 (i))

$V_{pl,DN,Rd}$ is the shear resistance of the double notched beam at the critical section (see figure)
= $\min(V_{Rd,g};\ V_{Rd,n})$ (see Check 4 (i))

CHECK 4 (continued) — Supported beam – Resistance at the connection





Shear and bending interaction of the beam web:

For short fin plates (i.e. $z_p \le t_p/0.15$) background studies have demonstrated that the resistance of the web does not need to be checked.

For long fin plates (i.e. $z_p > t_p/0.15$) it is necessary to ensure that the section ABCD shown above can resist a moment $V_{Ed} z_p$ for single line of bolts or $V_{Ed}\left(z_p + p_2\right)$ for a double line of bolts. Sections AB and CD are considered to be in shear and section BC in bending.

Basic requirement:

For a single vertical line of bolts ($n_2 = 1$):

$V_{Ed}\, z_p \le M_{c,BC,Rd} + V_{pl,AB,Rd}\,(n_1 - 1)\, p_1$

For two vertical lines of bolts ($n_2 = 2$):

$V_{Ed}\left(z_p + p_2\right) \le M_{c,BC,Rd} + V_{pl,AB,Rd}\,(n_1 - 1)\, p_1$

$M_{c,BC,Rd}$ is the moment resistance of the beam web BC

For low shear (i.e. $V_{BC,Ed} \le 0.5 V_{pl,BC,Rd}$)

$$M_{c,BC,Rd} = \frac{f_{y,b1} t_{w,b1}}{6\gamma_{M0}} \left((n_1 - 1)p_1\right)^2$$

For high shear (i.e. $V_{BC,Ed} > 0.5 V_{pl,BC,Rd}$)

$$M_{c,BC,Rd} = \frac{f_{y,b1} t_{w,b1}}{6\gamma_{M0}} \left((n_1 - 1)p_1\right)^2 \left(1 - \left(\frac{2V_{BC,Ed}}{V_{pl,BC,Rd}} - 1\right)^2\right)$$

$V_{pl,AB,Rd}$ is the shear resistance of the beam web AB

$V_{pl,BC,Rd}$ is the shear resistance of the beam web BC

For a single vertical line of bolts ($n_2 = 1$):

$$V_{pl,AB,Rd} = \frac{t_{w,b1} e_{2,b} \times f_{y,b1}}{\sqrt{3}\gamma_{M0}}$$

For two vertical lines of bolts ($n_2 = 2$):

$$V_{pl,AB,Rd} = \frac{t_{w,b1}\left(e_{2,b} + p_2\right) \times f_{y,b1}}{\sqrt{3}\gamma_{M0}}$$

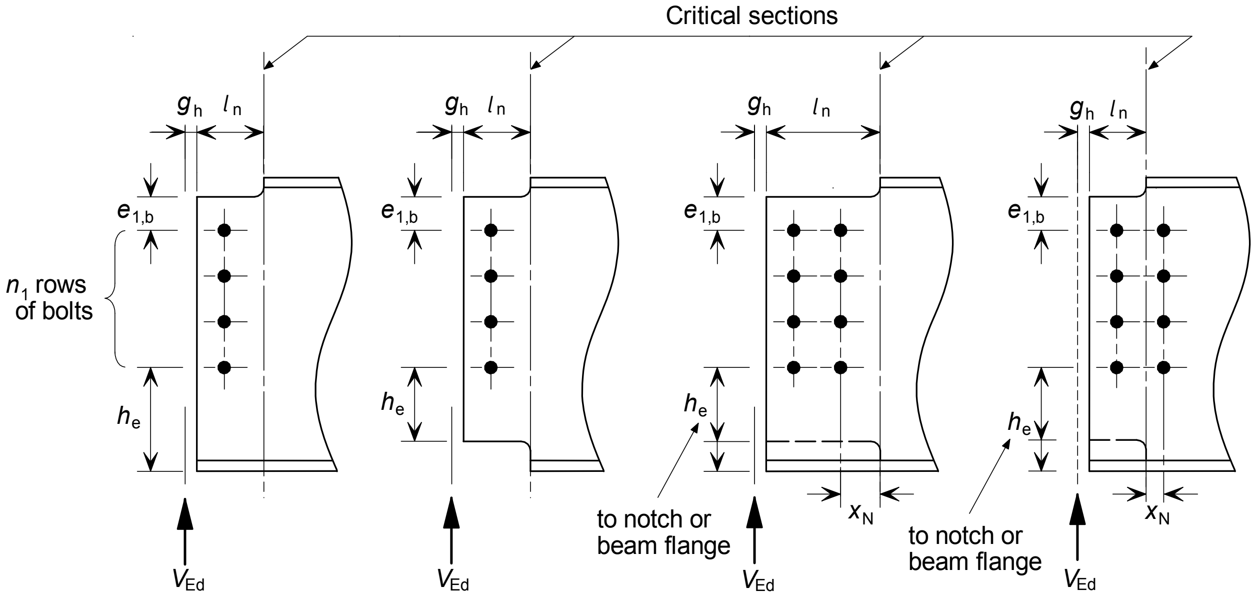
$V_{BC,Ed}$ is the shear force on the beam web BC

$$= V_{Ed} \frac{(n_1 - 1)p_1}{h_{b1}}$$

h_{b1} is the height of the beam

$$V_{pl,BC,Rd} = \frac{t_{w,b1}(n_1 - 1)p_1 \times f_{y,b1}}{\sqrt{3}\gamma_{M0}}$$

CHECK 5	Supported beam – Resistance at a notch



Shear and bending interaction at the notch:

Basic requirement:

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

$V_{Ed}\ (g_h + l_n) \leq M_{v,N,Rd}$

$M_{v,N,Rd}$ is the moment resistance of the beam at the notch in the presence of shear

For single notched beam:

For low shear (i.e. $V_{Ed} \leq 0.5V_{pl,N,Rd}$)

$$M_{v,N,Rd} = \frac{f_{y,b1}W_{el,N}}{\gamma_{M0}}$$

For high shear (i.e. $V_{Ed} > 0.5V_{pl,N,Rd}$)

$$M_{v,N,Rd} = \frac{f_{y,b1}W_{el,N}}{\gamma_{M0}}\left(1-\left(\frac{2V_{Ed}}{V_{pl,N,Rd}}-1\right)^2\right)$$

For double notched beam if $x_N \geq 2d$:

For low shear (i.e. $V_{Ed} \leq 0.5V_{pl,DN,Rd}$)

$$M_{v,DN,Rd} = \frac{f_{y,b1}t_{w,b1}}{6\gamma_{M0}}\left(e_{1,b}+(n_1-1)p_1+h_e\right)^2$$

For high shear (i.e. $V_{Ed} > 0.5V_{pl,DN,Rd}$)

$$M_{v,DN,Rd} = \frac{f_{y,b1}t_{w,b1}}{6\gamma_{M0}}\left(e_{1,b}+(n_1-1)p_1+h_e\right)^2\left(1-\left(\frac{2V_{Ed}}{V_{pl,DN,Rd}}-1\right)^2\right)$$

For double bolt lines if $x_N < 2d$:

$\max\left(V_{Ed}\ (g_h + l_n);\ V_{Ed}\ (g_h + e_{2,b} + p_2)\right) \leq M_{v,N,Rd}$

where:

$W_{el,N}$ is the elastic section modulus of the gross tee section at the notch

$V_{pl,N,Rd}$ is the shear resistance at the notch for single notched beams

$$= \frac{A_{v,N}f_{y,b1}}{\sqrt{3}\,\gamma_{M0}}$$

$$A_{v,N} = A_{Tee} - b_{b1}t_{f,b1} + (t_{w,b1} + 2r_{b1})\frac{t_{f,b1}}{2}$$

$V_{pl,DN,Rd}$ is the shear resistance at the notch for double notched beams

$$= \frac{A_{v,DN}f_{yb}}{\sqrt{3}\,\gamma_{M0}}$$

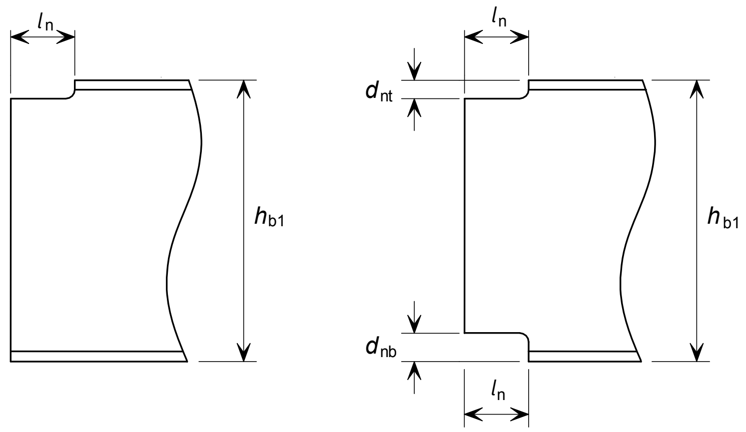
$A_{v,DN}$ $= 0.9\ t_{w,b1}\ (e_{1,b} + (n_1 - 1)\ p_1 + h_e)$

$t_{w,b1}$ is the thickness of the beam web

t_p is the thickness of the fin plate

$M_{v,N,Rd}$ $= M_{c,Rd}$ from 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:

Basic requirement:

For one flange notched [21], [22].

$d_{nt} \leq h_{b1} / 2$ and:

$l_n \leq h_{b1}$ for $h_{b1} / t_{w,b1} \leq$ 54.3 (S275 steel)

$l_n \leq \dfrac{160000\, h_{b1}}{(h_{b1} / t_{w,b1})^3}$ for $h_{b1} / t_{w,b1} >$ 54.3 (S275 steel)

$l_n \leq h_{b1}$ for $h_{b1} / t_{w,b1} \leq$ 48.0 (S355 steel)

$l_n \leq \dfrac{110000\, h_{b1}}{(h_{b1} / t_{w,b1})^3}$ for $h_{b1} / t_{w,b1} >$ 48.0 (S355 steel)

For both flanges notched [21].

$max\ (d_{nt}, d_{nb}) \leq h_{b1} / 5$ and:

$l_n \leq h_{b1}$ for $h_{b1} / t_{w,b1} \leq$ 54.3 (S275 steel)

$l_n \leq \dfrac{160000\, h_{b1}}{(h_{b1} / t_{w,b1})^3}$ for $h_{b1} / t_{w,b1} >$ 54.3 (S275 steel)

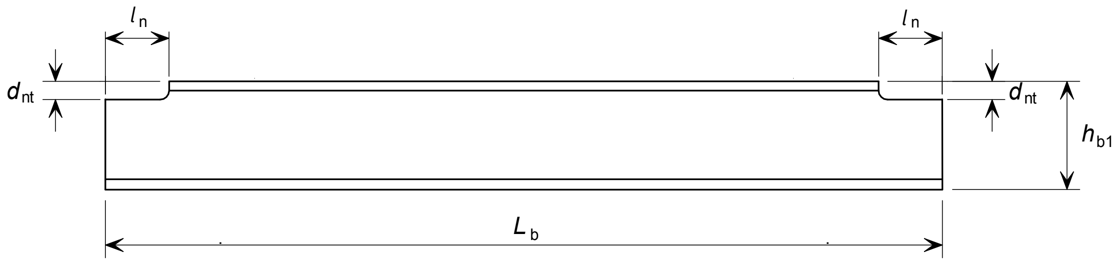
$l_n \leq h_{b1}$ for $h_{b1} / t_{w,b1} \leq$ 48.0 (S355 steel)

$l_n \leq \dfrac{110000\, h_{b1}}{(h_{b1} / t_{w,b1})^3}$ for $h_{b1} / t_{w,b1} >$ 48.0 (S355 steel)

$t_{w,b1}$ is the web thickness of the supported beam

Where the notch length l_n exceeds these limits, either suitable stiffening should be provided or the notch should be checked to references 21, 22 and 27.

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.

(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 l_n and/or notch depth d_{nt} are different at each end, then the larger value for l_n and d_{nt} should be used.

(3) Beams should be checked for lateral torsional buckling to BS EN 1993-1-1, clause 6.3.2 using an effective length in the calculation of M_{cr}, the elastic critical buckling moment for lateral torsional buckling.

(4) The solution below gives an effective length (L_E) based on references 29, 30 and 31. It is only valid for l_n / L_b < 0.15 and d_{nt} / h_{b1} < 0.2. Beams with notches outside these limits should be checked as tee sections, or stiffened.

Basic requirement:

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

$$K = K_0 / \lambda_b$$

$$\lambda_b = \frac{UVL_b}{i_z}$$

where:

X, U, V and i_z are for the un-notched I beam section and are defined in P363 [23]

Conservatively:

U = 0.9 and

V = 1.0

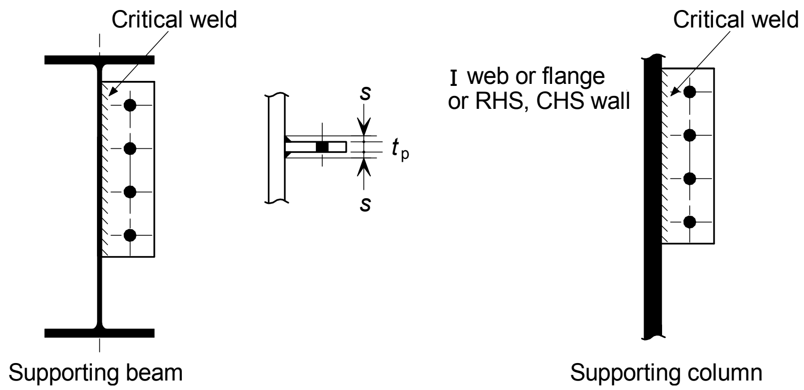
For $\lambda_b < 30$ $K_0 = 1.1\, g_0 \times$ but $\leq 1.1K_{max}$

For $\lambda_b \geq 30$ $K_0 = g_0 \times$ but $\leq K_{max}$

g_o and K_{max} are as follows:

$\frac{l_n}{L_b}$	g_0	K_{max}	
		UKB section	**UKC 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:

$a \geq 0.5t_p$ for S275 fin plate

$a \geq 0.6t_p$ for S355 fin plate

where:

a is the effective weld throat thickness $= 0.7s$ (normally)

s is the leg length

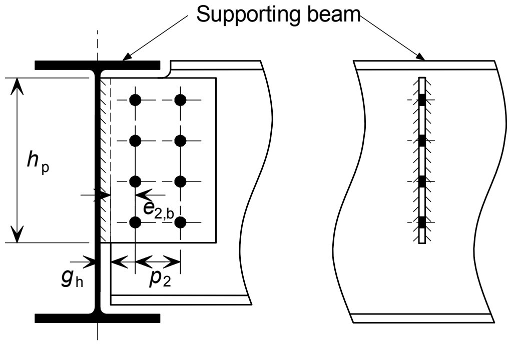
t_p is the thickness of the fin plate

Note:

This check ensures that the weld is not the weakest part of the connection.

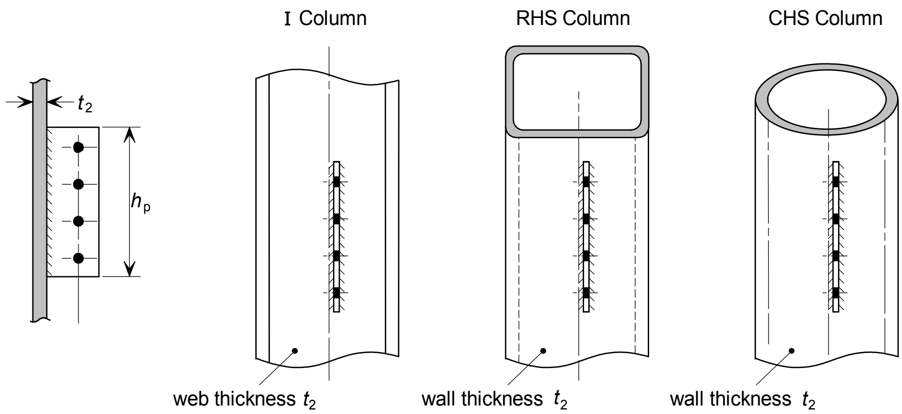
See Appendix C for more details about the weld requirements.

CHECK 10	Supporting beam/column – Shear and bearing (with one supported beam)



$$z = e_{2,b} + g_h \quad \text{for single row of bolts}$$

$$z = e_{2,b} + g_h + \frac{p_2}{2} \quad \text{for two rows of bolts}$$



Local shear and punching shear resistance of beam web, web of I column or wall of RHS or CHS column supporting one beam

Local shear

Basic requirement:

$$\frac{V_{Ed}}{2} \leq F_{Rd}$$

$$F_{Rd} = A_v \frac{f_{y,2}}{\sqrt{3}\gamma_{M0}}$$

where:

$A_v = h_p t_2$

t_2 is the thickness of the supporting column web or beam web

h_p is the height of the fin plate

Punching shear

Provided that the requirement given below is satisfied, yielding of the fin plate occurs before punching shear failure of the supporting member.

Basic requirement:

$$t_p \leq t_2 \frac{f_{u,2}}{f_{y,p}\gamma_{M2}} \quad \text{(conservative)}$$

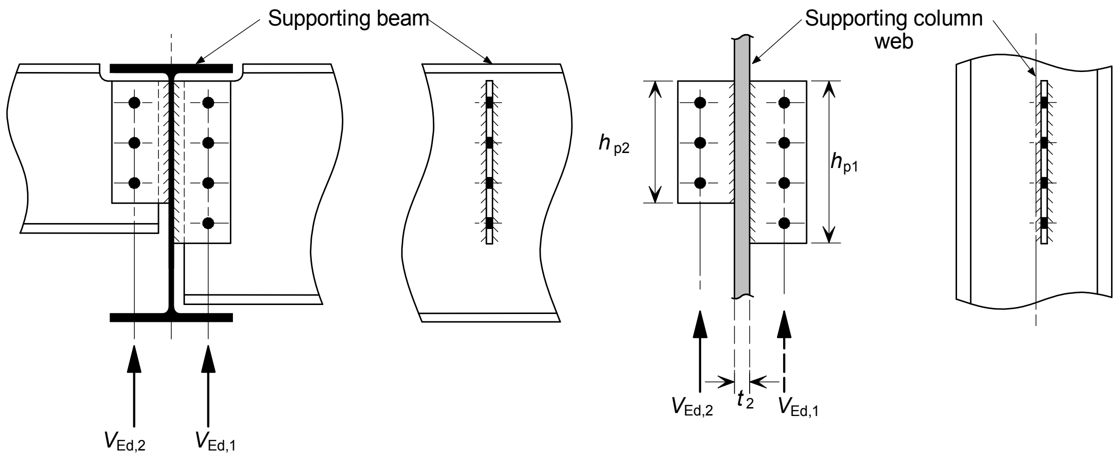
$$t_p \leq t_2 \frac{f_{u,2}\, t_p h_p^2}{V_{Ed}\, 6 z \gamma_{M2}} \quad \text{(rigorous)}$$

If the requirement is not satisfied, strengthening will be necessary.

CHECK 10 (continued)	**Supporting beam/column – Shear and bearing** (with two supported beams)

Note:

The supporting beam web or column web must be checked for the combined shear force applied by both fin plates, $V_{Ed,tot}$, over the length of the smaller fin plate.



Local shear and punching shear resistance of supporting beam web or column web supporting two beams:

Basic requirement:

$$\frac{V_{Ed,tot}}{2} \leq F_{Rd}$$

$$V_{Ed,tot} = \left(\frac{V_{Ed,1}}{h_{p,1}} + \frac{V_{Ed,2}}{h_{p,2}}\right) h_{p,min}$$

$$F_{Rd} = \frac{A_v f_{y,2}}{\sqrt{3}\gamma_{M0}}$$

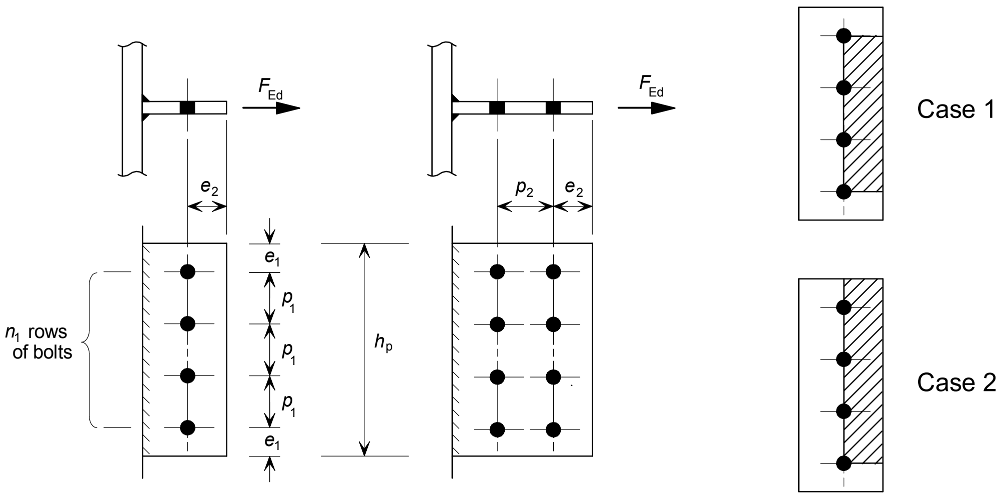
where:

A_v = $h_{p,min}\, t_2$

$h_{p,min}$ = $\min\left(h_{p,1}\,;\quad h_{p,2}\right)$

$f_{y,2}$ = yield strength of the supporting member

CHECK 11	Structural integrity – Plate and bolts



Single line of bolts   Double line of bolts

Note:

To resist a tie force of 75 kN, the connection must have at least 2 M20, 8.8 bolts and the fin plate thickness ≥ 6 mm.

Structural integrity – tension and bearing resistance of the fin plate:

Tension resistance of fin plate

Basic requirement: $F_{Ed} \leq F_{Rd,u}$

$F_{Rd,u}$ is the smaller of the block tearing tension resistance, $F_{Rd,b}$ and the net section tension resistance, $F_{Rd,n}$

$$F_{Rd,b} = \frac{f_{u,p} A_{nt}}{\gamma_{Mu}} + \frac{f_{y,p}}{\sqrt{3}} \frac{A_{nv}}{\gamma_{M0}}$$

$$F_{Rd,n} = \frac{0.9 A_{net,p} f_{u,p}}{\gamma_{Mu}}$$

Bolt shear

Basic requirement: $F_{Ed} \leq F_{Rd,u}$

$F_{Rd,u} = n\, F_{v,u}$

$$F_{v,u} = \frac{\alpha_v f_{ub} A}{\gamma_{Mu}}$$

Bolt bearing in fin plate

Basic requirement: $F_{Ed} \leq F_{Rd,u}$

$F_{Rd,u} = n\, F_{b,hor,Rd,u}$

$$F_{b,hor,Rd,u} = \frac{k_1 \alpha_b f_{u,p} d t_p}{\gamma_{Mu}}$$

where:

For Case 1:

$A_{nt} = t_p((n_1 - 1)p_1 - (n_1 - 1)\, d_0)$

For one vertical line of bolts ($n_2 = 1$):

$A_{nv} = 2t_p\left(e_2 - d_0/2\right)$

For two vertical lines of bolts ($n_2 = 2$):

$A_{nv} = 2t_p\left(p_2 + e_2 - 3d_0/2\right)$

For Case 2:

$A_{nt} = t_p((n_1 - 1)p_1 - (n_1 - 0.5)\, d_0 + e_1)$

For one vertical line of bolts ($n_2 = 1$):

$A_{nv} = t_p\left(e_2 - d_0/2\right)$

For two vertical lines of bolts ($n_2 = 2$):

$A_{nv} = t_p\left(p_2 + e_2 - 3d_0/2\right)$

$A_{net,p} = t_p(h_p - d_0 n_1)$

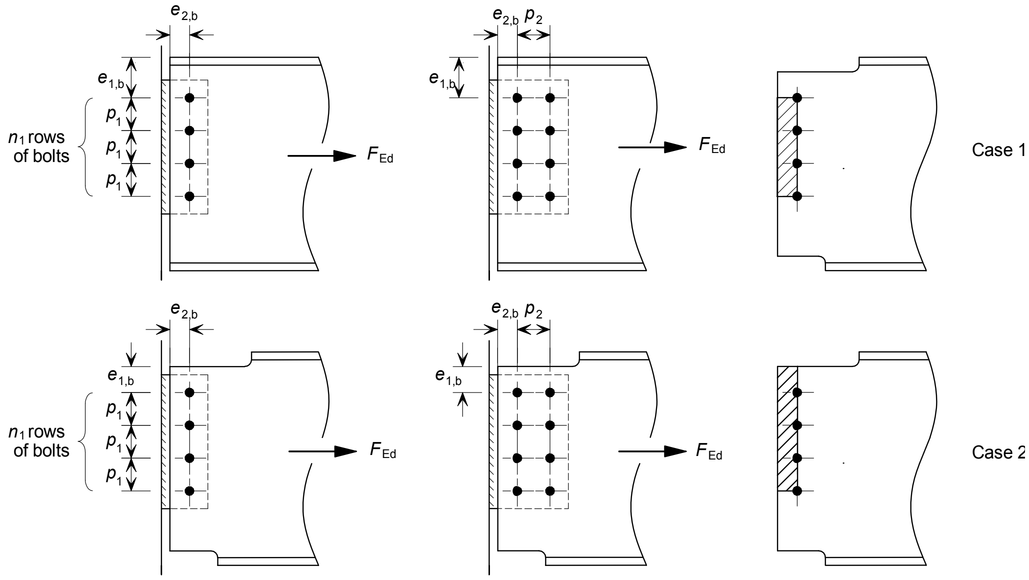
$\alpha_v = 0.6$ for 8.8

$$\alpha_b = \min\left(\frac{e_2}{3d_0};\ \frac{p_2}{3d_0} - \frac{1}{4};\ \frac{f_{ub}}{f_{u,p}};\ 1.0\right)$$

$$k_1 = \min\left(2.8\frac{e_1}{d_0} - 1.7;\quad 1.4\frac{p_1}{d_0} - 1.7;\quad 2.5\right)$$

When calculating k_1 and α_b for connections with a single vertical row of bolts ($n_2 = 1$), terms involving p_2 are ignored.

CHECK 12	Structural integrity – Supported beam web



Structural integrity – tension and bearing resistance of beam web

Tension resistance of beam web

Basic requirement:

$F_{\mathrm{Ed}} \leq F_{\mathrm{t,Rd,u}}$

$F_{\mathrm{t,Rd,u}}$ is the smaller of the block tearing tension resistance, $F_{\mathrm{Rd,b}}$ and the net section tension resistance, $F_{\mathrm{Rd,n}}$

$$F_{\mathrm{Rd,b}} = \frac{A_{\mathrm{nt}} f_{\mathrm{u,b1}}}{\gamma_{\mathrm{Mu}}} + \frac{A_{\mathrm{nv}} f_{\mathrm{y,b1}}}{\sqrt{3}\gamma_{\mathrm{M0}}}$$

$$F_{\mathrm{Rd,n}} = 0.9\, A_{\mathrm{net,wb}} \frac{f_{\mathrm{u,b1}}}{\gamma_{\mathrm{Mu}}}$$

Bolt bearing in beam web

Basic requirement:

$F_{\mathrm{Ed}} \leq F_{\mathrm{Rd,u}}$

$F_{\mathrm{Rd,u}} = n\, F_{\mathrm{b,hor,Rd,u}}$

$$F_{\mathrm{b,hor,Rd,u}} = \frac{k_1 \alpha_{\mathrm{b}} f_{\mathrm{u,b1}} d t_{\mathrm{w,b1}}}{\gamma_{\mathrm{Mu}}}$$

where:

for Case 1:

$A_{\mathrm{nt}} = t_{\mathrm{w,b1}}\ ((n_1 - 1)\, p_1 - (n_1 - 1) d_0\ )$

For a single vertical line of bolts (n_2 = 1):

$A_{\mathrm{nv}} = 2 t_{\mathrm{w,b1}} \left(e_{2,\mathrm{b}} - d_0 / 2\right)$

For two vertical lines of bolts (n_2 = 2)

$A_{\mathrm{nv}} = 2 t_{\mathrm{w,b1}} \left(p_2 + e_{2,\mathrm{b}} - 3 d_0 / 2\right)$

For case 2:

$A_{\mathrm{nt}} = t_{\mathrm{w,b1}}\ ((n_1 - 1)\, p_1 - (n_1 - 0.5) d_0 + e_{1,\mathrm{b}})$

For a single vertical line of bolts (n_2 = 1):

$A_{\mathrm{nv}} = t_{\mathrm{w,b1}} + \left(e_{2,\mathrm{b}} - d_0/2\right)$

For two vertical lines of bolts (n_2 = 2)

$A_{\mathrm{nv}} = t_{\mathrm{w,b1}} + \left(p_2 + e_{2,\mathrm{b}} - 3 d_0/2\right)$

$A_{\mathrm{net,wb}} = t_{\mathrm{w,b1}}\ (h_{\mathrm{wb}} - d_0\, n_1)$

$h_{\mathrm{wb}} = h_{\mathrm{p}}$ (conservatively)

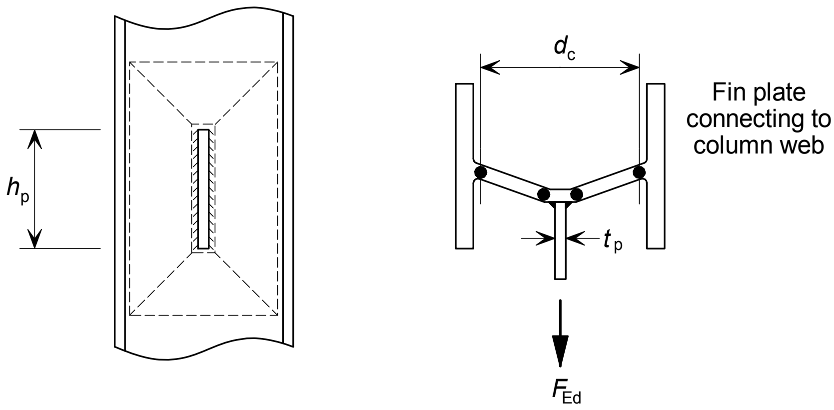
$$\alpha_{\mathrm{b}} = \min\left(\frac{e_{2,\mathrm{b}}}{3 d_0};\ \frac{p_2}{3 d_0} - \frac{1}{4};\ \frac{f_{\mathrm{ub}}}{f_{\mathrm{u,b1}}};\ 1.0\right)$$

$$k_1 = \min\left(1.4 \frac{p_1}{d_0} - 1.7;\quad 2.5\right)$$

When calculating α_{b} for connections with a single vertical row of bolts (n_2 = 1), terms involving p_2 are ignored.

CHECK 13	Structural integrity – welds
The weld size specified in Check 8 will be adequate for tying resistance. See Appendix C for more details about the weld requirements.	

CHECK 14	Structural integrity – supporting column (UKC or UKB)



Structural integrity – tying resistance of rolled column web, in the presence of axial compression in the column

Basic requirement:

$F_{Ed} \leq F_{Rd,u}$

$$F_{Rd,u} = \frac{8M_{pl,Rd,u}}{\gamma_{Mu}\left(1-\beta_1\right)}\left(\eta_1 + 1.5\left(1-\beta_1\right)^{0.5}\right)$$

$$M_{pl,Rd,u} = \frac{1}{4} f_{u,2} t_2^{\,2}$$

The 1.5 factor in the equation for F_{Rd} includes an allowance for the axial compression in the column

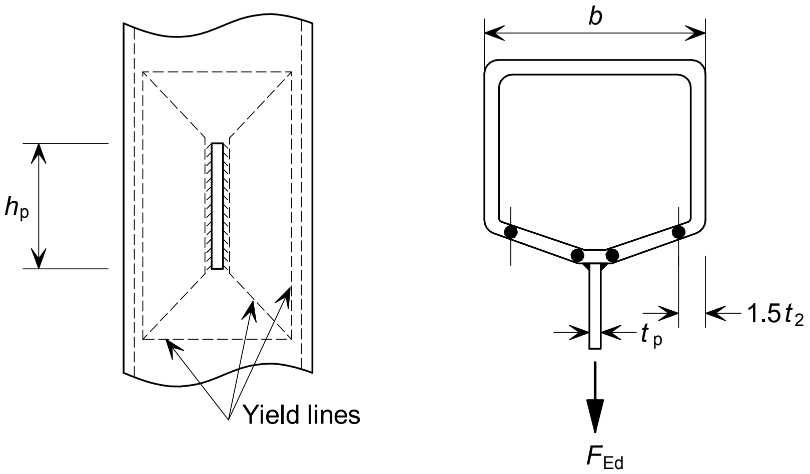
where:

t_2		is the web thickness of the column
η_1	=	$\dfrac{h_p}{d_c}$
β_1	=	$\dfrac{t_p + 2s}{d_c}$
d_c		is the depth of straight portion of the column web
a		is the weld throat = 0.7s
s		is the leg length of fillet weld

Note:

This check is required for either single-sided connections to a column web or unequally loaded double-sided connections to a column web.

CHECK 15	**Structural integrity – supporting column** **(Hollow section)**



Structural integrity – tying resistance of the hollow section wall, with axial compression in the column

Basic requirement:

$F_{Ed} \leq F_{Rd,u}$

$$F_{Rd,u} = \frac{8M_{pl,Rd,u}}{\gamma_{Mu}(1-\beta)}\left(\eta + 1.5(1-\beta)^{0.5}\right)$$

$$M_{pl,Rd,u} = \frac{1}{4} f_{u,2} t_2^{\,2}$$

The 1.5 factor in the equation for $F_{Rd,u}$ includes an allowance for the axial compression in the column

where:

t_2 is the thickness of the hollow section column

$\eta = \dfrac{h_p}{b - 3t_2}$

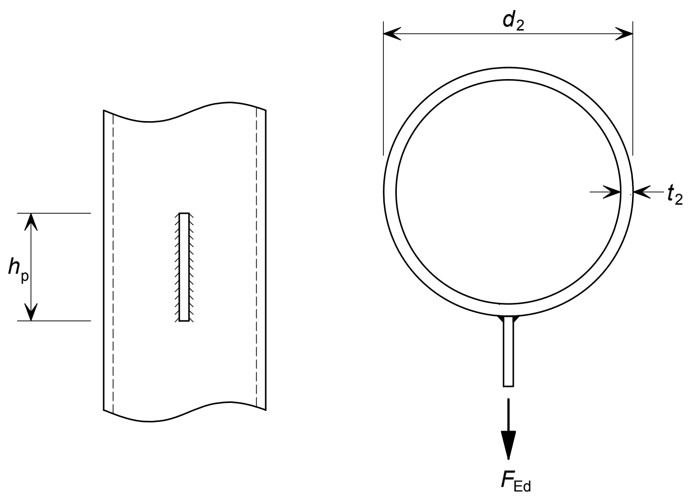
$\beta = \dfrac{t_p + 2s}{b - 3t_2}$

b is the overall width of the hollow section column wall to which the connection is made

a is the weld throat = $0.7s$

s is the leg length of fillet weld

CHECK 16	Structural integrity – supporting column wall (CHS)



Structural integrity – tying resistance of the CHS wall, with axial force in the column

Basic requirement:

$F_{Ed} \leq F_{Rd}$

$$F_{Rd} = \frac{5 f_{u,2} t_2^{\ 2} \left(1 + 0.25\eta\right) 0.67}{\gamma_{Mu}}$$

The 0.67 factor in the equation for F_{Rd} includes an allowance for the axial force in the column. It is recommended that this factor is applied in all cases (either compression or tension in the column).

where:

t_2 is the thickness of the CHS column

$\eta = \dfrac{h_p}{d_2}$

d_2 is the diameter of the CHS column

5.6 WORKED EXAMPLES

The worked examples show design calculations for typical standard connections. Each example demonstrates firstly the use of the resistance tables (yellow pages) and then the 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.

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 tying resistance. The magnitude of the tie force depends on the class of building. When the tie force is 75 kN, no checks are required, as every standard detail will accommodate this force. Calculations will be required for larger forces.

Example 1

This example covers the design checks for a two sided beam to beam connection with fin plates welded to the web of the supporting beam. A fin plate with a single line of vertical bolts is employed on one side and a fin plate 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 carried by the connection.

Example 3

Example 3 is a beam to hollow section column connection using fin plates welded to the column wall. The beam sizes and vertical reactions are as in Example 1, so only checks which are different from those in the first example are shown.

Example 4

Example 4 shows the same beam connections as in Example 3 but uses fin plates welded to a CHS column.

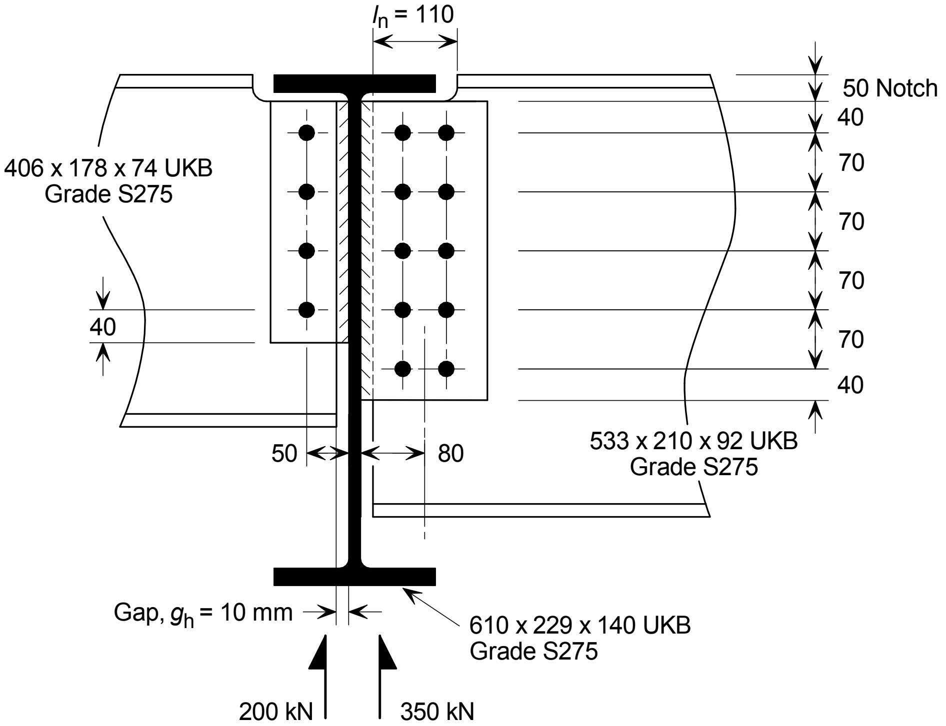
SCI Steel Knowledge **CALCULATION SHEET** BCSA	Job	*Joints in Steel Construction – Simple Joints*	*Sheet 1 of 15*
	Title	*Example 1 – Fin plates – Beam to beam*	
	Client	*Connections Group*	
	Calcs by *CZT*	Checked by *ENM*	Date *Sept 2011*

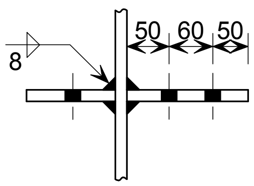
DESIGN EXAMPLE 1

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

Yellow pages are used for the initial selection of the connection detail.

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 × 229 × 92 UKB in order to demonstrate the design checks required for this configuration.





Fin Plates: 100 × 10 (Single line) 160 × 10 (Double line) — Table G.18 and G.19

Design information:

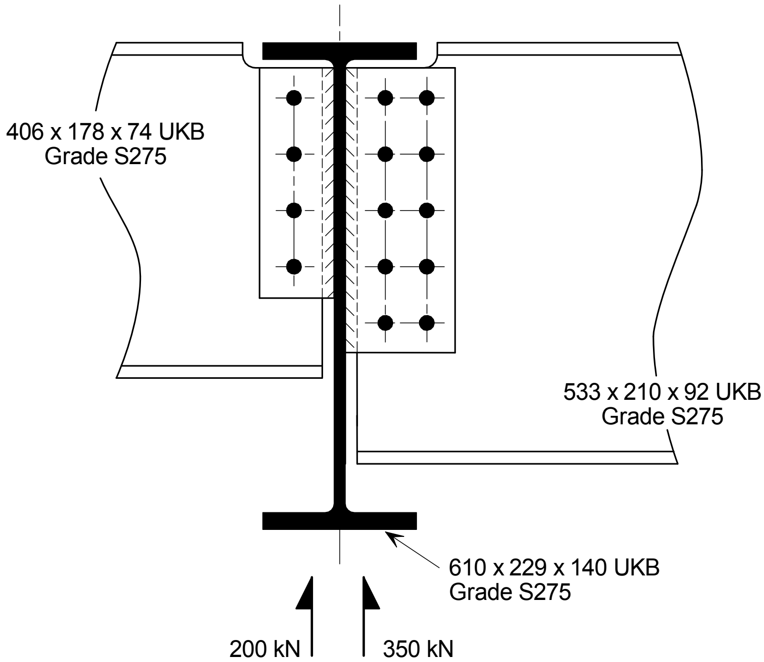
Bolts: M20, 8.8

Welds: 8 mm leg length fillet weld

Material: All S275

Title Example 1 – Fin Plates – Beam to Beam	Sheet 2 of 15

CONNECTION DESIGN USING RESISTANCE TABLES



406 × 178 × 74 UKB, S275

Bolts M20 8.8

From Resistance Table G.18

Connection shear resistance (single notch)
= 286 kN > 200 kN

Maximum notch length
= 311 mm > 120 mm

533 × 210 × 92 UKB, S275

Bolts M20 8.8

From Resistance Table G.19

Connection shear resistance (single notch)
= 450 kN > 350 kN

Maximum notch length
= 359 mm > 120 mm

Web thickness of supporting beam = 13.1 mm
Minimum support thickness based on the maximum shears of 286 kN and 450 kN
= (3.4 + 4.4) = 7.8 mm < 13.1 mm ∴ **O.K.**
The connection is adequate.

Title *Example 1 – Fin Plates – Beam to Beam* *Sheet 3 of 15*

SUMMARY OF FULL DESIGN CHECKS FOR EXAMPLE 1

Sheet No.	CHECK		406 UB (S275)		533 UB (S275)		610 UB (S275)	
			Resistance	Design force	Resistance	Design force	Resistance	Design force
4	***Check 1*** Recommended detailing practice		All recommendations adopted					
4	***Check 2*** Supported beam Bolt group	Shear (kN)	288	200	577	350	Not applicable	
		Fin plate in bearing (kN)	332	200	574	350		
		Beam web in bearing (kN)	291	200	565	350		
8	***Check 3*** Supported beam Fin plate	Shear	347	200	450	350	Not applicable	
		Bending resistance (kN)	∞	200	∞	350		
		LTB of fin plate (kN)	771	200	743	350		
10	***Check 4*** Supported beam Web in shear	Shear (kN)	312	200	481	350	Not applicable	
		Bending resistance (kNm)	NA	NA	164	38.5		
		Long fin plate (kN)	NA	NA	NA	NA		
13	***Check 5*** Supported beam Resistance at a notch	Bending resistance (kNm)	89	24	164	42	Not applicable	
14	***Check 6*** Supported beam Local stability of notched beam	Notch length (mm)	413	110	533	120	Not applicable	
			Notch length l_n < specified limits					
	Check 7 Supported beam Overall stability	–	Not applicable				Not applicable	
14	***Check 8*** Supporting beam/column – Welds	Weld throat, *a* (mm)	5.7	5.0	5.7	5.0	Not applicable	
	Check 9	–	Not applicable				Not applicable	
15	***Check 10*** Supporting beam Shear and bearing	Shear resistance (kN)	Not applicable				603	241

Title *Example 1 – Fin plates – Beam to beam* | *Sheet 4 of 15*

CONNECTION DESIGN FOLLOWING THE DESIGN PROCEDURES

Check 1: Recommended detailing practice

Fin plate thickness: t_p = 10 mm $\leq 0.5d$ (d = 20 mm diameter)

Height of fin plate: h_p = 290 mm (> $0.6h_{b1}$ for 406 UKB)

= 360 mm (> $0.6h_{b1}$ for 533 UKB)

Bolts: M20 8.8

Check 2: Supported beam – Bolt group

Bolt shear

Basic requirement: $V_{Ed} \leq V_{Rd}$

$$V_{Rd} = \frac{nF_{v,Rd}}{\sqrt{(1+\alpha n)^2 + (\beta n)^2}}$$

$$F_{v,Rd} = \frac{\alpha_v f_{ub} A}{\gamma_{M2}}$$

For M20 8.8 bolts:

$$F_{v,Rd} = \frac{0.6 \times 800 \times 245}{1.25} \times 10^{-3} = 94 \text{ kN}$$

406 × 178 × 74 UKB, S275

For a single vertical line of bolt (i.e. n_2 = 1 and $n = n_1$):

α = 0

$$\beta = \frac{6z}{n_1(n_1+1)p_1} = \frac{6 \times 50}{4 \times (4+1) \times 70} = 0.21$$

$$\therefore V_{Rd} = \frac{4 \times 94}{\sqrt{(1+0 \times 4)^2 + (0.21 \times 4)^2}} = 288 \text{ kN}$$

V_{Ed} = 200 kN < 288 kN ∴ **O.K.**

533 × 210 × 92 UKB, S275

For a double vertical line of bolt (i.e. n_2 = 2 and $n = 2n_1$):

$$\alpha = \frac{zp_2}{2I}$$

$$I = \frac{n_1}{2}p_2^{\,2} + \frac{1}{6}n_1\left(n_1^{\,2}-1\right)p_1^{\,2} = \frac{5}{2} \times 60^2 + \frac{1}{6} \times 5 \times \left(5^2 - 1\right) \times 70^2 = 107000 \text{ mm}^2$$

z = 50 + 0.5 × 60 = 80 mm

$$\therefore \alpha = \frac{80 \times 60}{2 \times 107000} = 0.02$$

$$\beta = \frac{zp_1}{2I}\left(n_1 - 1\right) = \frac{80 \times 70}{2 \times 107000} \times (5-1) = 0.11$$

$$\therefore V_{Rd} = \frac{10 \times 94}{\sqrt{(1+0.02 \times 10)^2 + (0.11 \times 10)^2}} = 577 \text{ kN}$$ ∴ **O.K.**

V_{Ed} = 350 kN < 577 kN

Title *Example 1 – Fin plates – Beam to beam*	*Sheet 5 of 15*

Fin plate in bearing

Basic requirement: $V_{Ed} \leq V_{Rd}$

$$V_{Rd} = \frac{n}{\sqrt{\left(\frac{1+\alpha n}{F_{b,ver,Rd}}\right)^2 + \left(\frac{\beta n}{F_{b,hor,Rd}}\right)^2}}$$

406 × 178 × 74 UKB, S275

$\alpha = 0$ and $\beta = 0.21$, as above

The vertical bearing resistance of a single bolt:

$$F_{b,ver,Rd} = \frac{k_1 \alpha_b f_{up} d t_p}{\gamma_{M2}}$$

$$k_1 = \min\left(2.8\frac{e_2}{d_0} - 1.7;\quad 2.5\right) = \min\left(2.8 \times \frac{50}{22} - 1.7;\quad 2.5\right)$$

$$= \min(4.66;\quad 2.5) = 2.5$$

$$\alpha_b = \min\left(\frac{e_1}{3d_0};\ \frac{p_1}{3d_0} - \frac{1}{4};\ \frac{f_{ub}}{f_{u,p}};\ 1.0\right) = \min\left(\frac{40}{3 \times 22};\ \frac{70}{3 \times 22} - 0.25;\ \frac{800}{410};\ 1.0\right)$$

$$= \min(0.61;\quad 0.81;\quad 1.95;\quad 1.0) = 0.61$$

$$F_{b,ver,Rd} = \frac{2.5 \times 0.61 \times 410 \times 20 \times 10}{1.25} \times 10^{-3} = 100 \text{ kN}$$

The horizontal bearing resistance for a single bolt:

$$F_{b,hor,Rd} = \frac{k_1 \alpha_b f_{up} d t_p}{\gamma_{M2}}$$

$$k_1 = \min\left(2.8\frac{e_1}{d_0} - 1.7;\quad 1.4\frac{p_1}{d_0} - 1.7;\quad 2.5\right) = \min\left(2.8\frac{40}{22} - 1.7;\quad 1.4\frac{70}{22} - 1.7;\quad 2.5\right)$$

$$= \min(3.39;\quad 2.75;\quad 2.5) = 2.5$$

$$\alpha_b = \min\left(\frac{e_2}{3d_0};\quad \frac{f_{ub}}{f_{up}};\quad 1.0\right) = \min\left(\frac{50}{3 \times 22};\quad \frac{800}{410};\quad 1.0\right)$$

$$= \min(0.76;\quad 1.95;\quad 1.0) = 0.76$$

$$F_{b,hor,Rd} = \frac{2.5 \times 0.76 \times 410 \times 20 \times 10}{1.25} \times 10^{-3} = 125 \text{ kN}$$

$$\therefore V_{Rd} = \frac{4}{\sqrt{\left(\frac{1+0 \times 4}{100}\right)^2 + \left(\frac{0.21 \times 4}{125}\right)^2}} = 332 \text{ kN}$$

$\therefore V_{Ed} = 200$ kN $<$ 332 kN — **∴ O.K.**

533 × 210 × 92 UKB, S275

$\alpha = 0.02$ and $\beta = 0.11$, as above

The vertical bearing resistance for a single bolt:

$$F_{b,ver,Rd} = \frac{k_1 \alpha_b f_{up} d t_p}{\gamma_{M2}}$$

$$k_1 = \min\left(2.8\frac{e_2}{d_0} - 1.7;\ 1.4\frac{p_2}{d_0} - 1.7;\ 2.5\right) = \min\left(2.8\frac{50}{22} - 1.7;\ 1.4\frac{60}{22} - 1.7;\ 2.5\right)$$

$$= \min(4.66;\quad 2.12;\quad 2.5) = 2.12$$

$$\alpha_b = \min\left(\frac{e_1}{3d_0};\ \frac{p_1}{3d_0}-\frac{1}{4};\ \frac{f_{ub}}{f_{u,p}};\ 1.0\right) = \min\left(\frac{40}{3\times22};\ \frac{70}{3\times22}-0.25;\ \frac{800}{410};\ 1.0\right)$$

$$\alpha_b = \min(0.61;\ \ 0.81;\ \ 1.95;\ \ 1.0) = 0.61$$

$$\therefore F_{b,ver,Rd} = \frac{2.12\times0.61\times410\times20\times10}{1.25}\times10^{-3} = 85\text{ kN}$$

The horizontal bearing resistance for a single bolt is:

$$F_{b,hor,Rd} = \frac{k_1\alpha_b f_{up} d t_p}{\gamma_{M2}}$$

$$k_1 = \min\left(2.8\frac{e_1}{d_0}-1.7;\ 1.4\frac{p_1}{d_0}-1.7;\ 2.5\right) = \min\left(2.8\times\frac{40}{22}-1.7;\ 1.4\times\frac{70}{22}-1.7;\ 2.5\right)$$

$$= \min(3.39;\ \ 2.75;\ \ 2.5) = 2.5$$

$$\alpha_b = \min\left(\frac{e_2}{3d_0};\ \frac{p_2}{3d_0}-0.25;\ \frac{f_{ub}}{f_{u,p}};\ 1.0\right) = \min\left(\frac{50}{3\times22};\ \frac{60}{3\times22}-0.25;\ \frac{800}{410};\ 1.0\right)$$

$$= \min(0.76;\ \ 0.66;\ \ 1.95;\ \ 1.0) = 0.66$$

$$\therefore F_{b,hor,Rd} = \frac{2.5\times0.66\times410\times20\times10}{1.25}\times10^{-3} = 108\text{ kN}$$

$$\therefore V_{Rd} = \frac{10}{\sqrt{\left(\frac{1+0.02\times10}{85}\right)^2+\left(\frac{0.11\times10}{108}\right)^2}} = 574\text{ kN}$$

$\therefore V_{Ed} = 350$ kN < 574 kN **∴ O.K.**

Beam web in bearing

Basic requirement: $V_{Ed} < V_{Rd}$

$$V_{Rd} = \frac{n}{\sqrt{\left(\frac{1+\alpha n}{F_{b,ver,Rd}}\right)^2+\left(\frac{\beta n}{F_{b,hor,Rd}}\right)^2}}$$

406 × 178 × 74 UKB, S275

$\alpha = 0$ and $\beta = 0.21$, as above

The vertical bearing resistance for a single bolt:

$$F_{b,ver,Rd} = \frac{k_1\alpha_b f_{u,b1} d t_{wb1}}{\gamma_{M2}}$$

$$k_1 = \min\left(2.8\frac{e_{2,b}}{d_0}-1.7;\ \ 2.5\right) = \min\left(2.8\frac{40}{22}-1.7;\ \ 2.5\right) = \min(3.39;\ \ 2.5) = 2.5$$

$$\alpha_b = \min\left(\frac{e_{1,b}}{3d_0};\ \frac{p_1}{3d_0}-\frac{1}{4};\ \frac{f_{ub}}{f_{u,b1}};1.0\right) = \min\left(\frac{40}{3\times22};\ \frac{70}{3\times22}-\frac{1}{4};\ \frac{800}{410};\ \ 1.0\right)$$

$$= \min(0.61;\ \ 0.81;\ \ 1.95;\ \ 1.0) = 0.61$$

$$\therefore F_{b,ver,Rd} = \frac{2.5\times0.61\times410\times20\times9.5}{1.25}\times10^{-3} = 95\text{ kN}$$

The horizontal bearing resistance for a single bolt:

$$F_{b,hor,Rd} = \frac{k_1\alpha_b f_{u,b1} d t_{w,b1}}{\gamma_{M2}}$$

Title *Example 1 – Fin plates – Beam to beam*	*Sheet 7 of 15*

$k_1 = \min\left(2.8\dfrac{e_{1,b}}{d_0} - 1.7; 1.4\dfrac{p_1}{d_0} - 1.7;\ 2.5\right) = \min\left(2.8\times\dfrac{40}{22} - 1.7;\ 1.4\times\dfrac{70}{22} - 1.7;\ 2.5\right)$

$= \min(3.39;\quad 2.75;\quad 2.5) = 2.5$

$\alpha_b = \min\left(\dfrac{e_{2,b}}{3d_0};\quad \dfrac{f_{ub}}{f_{u,b1}};\quad 1.0\right) = \min\left(\dfrac{40}{3\times 22};\quad \dfrac{800}{410};\quad 1.0\right)$

$= \min(0.61;\quad 1.95;\quad 1.0) = 0.61$

$\therefore\ F_{b,hor,Rd} = \dfrac{2.5\times 0.61\times 410\times 20\times 9.5}{1.25}\times 10^{-3} = 95 \text{ kN}$

$\therefore\ V_{Rd} = \dfrac{4}{\sqrt{\left(\dfrac{1+0\times 4}{95}\right)^2 + \left(\dfrac{0.21\times 4}{95}\right)^2}} = 291 \text{ kN}$

$\therefore\ V_{Ed} = 200 \text{ kN} \qquad < 291 \text{ kN}$ $\qquad\therefore$ **O.K.**

533 × 210 × 92 UKB, S275

$\alpha = 0.02$ and $\beta = 0.11$, as above

The vertical bearing resistance for a single bolt:

$F_{b,ver,Rd} = \dfrac{k_1\alpha_b f_{u,b1} d t_{w,b1}}{\gamma_{M2}}$

$k_1 = \min\left(2.8\dfrac{e_{2,b}}{d_0} - 1.7;\quad 1.4\dfrac{p_2}{d_0} - 1.7;\quad 2.5\right) = \min\left(2.8\times\dfrac{40}{22} - 1.7;\ 1.4\times\dfrac{60}{22} - 1.7;\ 2.5\right)$

$= \min(3.39;\quad 2.12;\quad 2.5) = 2.12$

$\alpha_b = \min\left(\dfrac{e_{1,b}}{3d_0};\quad \dfrac{p_1}{3d_0} - \dfrac{1}{4};\quad \dfrac{f_{ub}}{f_{u,b1}};\quad 1.0\right) = \min\left(\dfrac{40}{3\times 22};\quad \dfrac{70}{3\times 22} - \dfrac{1}{4};\quad \dfrac{800}{410};\quad 1.0\right)$

$= \min(0.61;\quad 0.81;\quad 1.95;\quad 1.0) = 0.61$

$\therefore\ F_{b,ver,Rd} = \dfrac{2.12\times 0.61\times 410\times 20\times 10.1}{1.25}\times 10^{-3} = 86 \text{ kN}$

The horizontal bearing resistance for a single bolt:

$F_{b,hor,Rd} = \dfrac{k_1\alpha_b f_{u,b1} d t_{w,b1}}{\gamma_{M2}}$

$k_1 = \min\left(2.8\dfrac{e_{1,b}}{d_0} - 1.7;\ 1.4\dfrac{p_1}{d_0} - 1.7;\ 2.5\right) = \min\left(2.8\dfrac{40}{22} - 1.7;\ 1.4\times\dfrac{70}{22} - 1.7;\ 2.5\right)$

$= \min(3.39;\quad 2.75;\quad 2.5) = 2.5$

$\alpha_b = \min\left(\dfrac{e_{2,b}}{3d_0};\quad \dfrac{p_2}{3d_0} - 0.25;\quad \dfrac{f_{ub}}{f_{u,b1}};\quad 1.0\right) = \min\left(\dfrac{40}{3\times 22}; \dfrac{60}{3\times 22} - 0.25;\ \dfrac{800}{410};\ 1.0\right)$

$= \min(0.61;\quad 0.66;\quad 1.95;\quad 1.0) = 0.61$

$\therefore\ F_{b,hor,Rd} = \dfrac{2.5\times 0.61\times 410\times 20\times 10.1}{1.25}\times 10^{-3} = 101 \text{ kN}$

$\therefore\ V_{Rd} = \dfrac{10}{\sqrt{\left(\dfrac{1+0.02\times 10}{86}\right)^2 + \left(\dfrac{0.11\times 10}{101}\right)^2}} = 565 \text{ kN}$

$\therefore\ V_{Ed} = 350 \text{ kN} \qquad < 565 \text{ kN}$ $\qquad\therefore$ **O.K.**

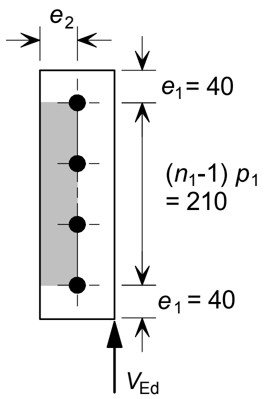
Check 3: Supported beam – Fin plate

Shear

Basic requirement: $V_{Ed} < V_{Rd,min}$

$V_{Rd,min} = \min(V_{Rd,g};\ V_{Rd,n};\ V_{Rd,b})$

406 × 178 × 74 UKB, S275



Gross section:

$$V_{Rd,g} = \frac{h_p t_p}{1.27}\frac{f_{y,p}}{\sqrt{3}\gamma_{M0}}$$

$$V_{Rd,g} = \frac{290\times10}{1.27}\times\frac{275}{\sqrt{3}\times1.0}\times10^{-3} = 363 \text{ kN}$$

Net section

$$V_{Rd,n} = A_{v,net}\frac{f_{u,p}}{\sqrt{3}\gamma_{M2}}$$

Net area: $A_{v,net} = t_p\left(h_p - n_1 d_0\right)$

$A_{v,net} = 10\times\left(290-4\times22\right) = 2020 \text{ mm}^2$

$$\therefore\ V_{Rd,n} = 2020\times\frac{410}{\sqrt{3}\times1.1}\times10^{-3} = 435 \text{ kN}$$

Block tearing

$$V_{Rd,b} = \frac{0.5 f_{u,p} A_{nt}}{\gamma_{M2}} + \frac{f_{y,p} A_{nv}}{\sqrt{3}\gamma_{M0}}$$

Net area subject to tension: $A_{nt} = t_p\left(e_2 - \dfrac{d_0}{2}\right)$

$A_{nt} = 10\times\left(50-0.5\times22\right) = 390 \text{ mm}^2$

Net area subject to shear: $A_{nv} = t_p\left(h_p - e_1 - (n_1 - 0.5)d_0\right)$

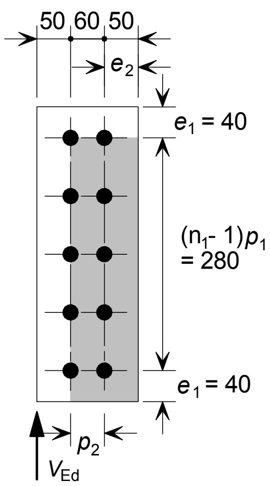
$A_{nv} = 10\times\left(290-40-(4-0.5)\times22\right) = 1730 \text{ mm}^2$

$$\therefore\ V_{Rd,b} = \left(\frac{0.5\times410\times390}{1.1} + \frac{275\times1730}{\sqrt{3}\times1.0}\right)\times10^{-3} = 347 \text{ kN}$$

$\therefore\ V_{Rd,min} = \min(363;\ 435;\ 347) = 347$ kN

$V_{Ed} = 200 \text{ kN} < 347 \text{ kN}$ $\therefore$ **O.K.**

533 × 210 × 92 UKB, S275



Gross section

$$V_{Rd,g} = \frac{h_p t_p}{1.27}\frac{f_{y,p}}{\sqrt{3}\gamma_{M0}}$$

$$V_{Rd,g} = \frac{360\times10}{1.27}\times\frac{275}{\sqrt{3}\times1.0}\times10^{-3} = 450 \text{ kN}$$

Net section

$$V_{Rd,n} = A_{v,net}\frac{f_{u,p}}{\sqrt{3}\gamma_{M1}}$$

Title *Example 1 – Fin plates – Beam to beam*	*Sheet 9 of 15*
Net area: $A_{v,net} = t_p(h_p - n_1 d_0)$	
$A_{v,net} = 10 \times (360 - 5 \times 22) = 2500\ \text{mm}^2$	
$\therefore V_{Rd,n} = 2500 \times \dfrac{410}{\sqrt{3} \times 1.1} \times 10^{-3} = 538\ \text{kN}$	
Block tearing	
$V_{Rd,b} = \dfrac{0.5 f_{u,p} A_{nt}}{\gamma_{M2}} + \dfrac{f_{y,p} A_{nv}}{\sqrt{3}\gamma_{M0}}$	
Net area subject to tension: $A_{nt} = t_p\left(p_2 + e_2 - \dfrac{3d_0}{2}\right)$	
$A_{nt} = 10 \times (60 + 50 - 1.5 \times 22) = 770\ \text{mm}^2$	
Net area subject to shear: $A_{nv} = t_p(h_p - e_1 - (n_1 - 0.5)d_0)$	
$A_{nv} = 10 \times (360 - 40 - (5 - 0.5) \times 22) = 2210\ \text{mm}^2$	
$\therefore V_{Rd,b} =$	
$\left(\dfrac{0.5 \times 410 \times 770}{1.1} + \dfrac{275 \times 2210}{\sqrt{3} \times 1.0}\right) \times 10^{-3} = 494\ \text{kN}$	
$\therefore V_{Rd,min} = \min(450;\ 538;\ 494) = 450\ \text{kN}$	
$\therefore V_{Ed} = 350\ \text{kN} < 450\ \text{kN}$	$\therefore$ **O.K.**
Bending	
Basic requirement: $V_{Ed} \leq V_{Rd}$	
406 × 178 × 74 UKB, S275	
$2.73z = 2.73 \times 50 = 136.5\ \text{mm}$	
$h_p = 290\ \text{mm} > 2.73z$	
$\therefore V_{Rd} = \infty$	
$V_{Ed} < V_{Rd}$	$\therefore$ **O.K.**
533 × 210 × 92 UKB, S275	
$2.73z = 2.73 \times 80 = 218.4\ \text{mm}$	
$h_p = 360\ \text{mm} > 2.73z$	
$\therefore V_{Rd} = \infty$	
$V_{Ed} < V_{Rd}$	$\therefore$ **O.K.**
Lateral torsional buckling of fin plates	
Basic requirement: $V_{Ed} \leq V_{Rd}$	
406 × 178 × 74 UKB, S275	
$z = 50\ \text{mm} < 66.7\ \text{mm}$ Therefore fin plate is short. For short fin plates:	
$V_{Rd} = \dfrac{W_{el,p}}{z}\dfrac{f_{y,p}}{\gamma_{M0}}$	
$W_{el,p} = \dfrac{t_p h_p^2}{6} = \dfrac{10 \times 290^2}{6} = 140167\ \text{mm}^3$	
$V_{Rd} = \dfrac{140167}{50} \times \dfrac{275}{1.0} \times 10^{-3} = 771\ \text{kN}$	
$\therefore V_{Ed} = 200\ \text{kN} < 771\ \text{kN}$	$\therefore$ **O.K.**

533 × 210 × 92 UKB, S275

z = 80 mm > 66.7 mm Therefore fin plate is long. For long fin plates:

$$V_{Rd} = \min\left(\frac{W_{el,p}}{z}\frac{\chi_{LT} f_p}{0.6\gamma_{M1}};\ \frac{W_{el,p}}{z}\frac{f_{y,p}}{\gamma_{M0}}\right)$$

$$W_{el,p} = \frac{t_p h_p^{\ 2}}{6} = \frac{10 \times 360^2}{6} = 216000 \text{ mm}^3$$

$$\bar{\lambda}_{LT} = \frac{2.8}{86.4}\left(\frac{(e_{2,b} + g_h)h_p}{1.5 t_p^{\ 2}}\right)^{1/2} = \frac{2.8}{86.4}\times\left(\frac{(40+10)\times 360}{1.5\times 10^2}\right)^{1/2} = 0.36$$

From Table 5.2, $\chi_{LT} = 0.88$

$$V_{Rd} = \min\left(\frac{216000}{80}\frac{0.88\times 275}{0.6\times 1.0}\times 10^{-3};\ \frac{216000}{80}\frac{275}{1.0}\times 10^{-3}\right)$$

$$= \min(1089;\ 743) = 742 \text{ kN}$$

$\therefore V_{Ed}$ = 350 kN < 743 kN **∴ O.K.**

Check 4: Supported beam – Web in shear

Shear

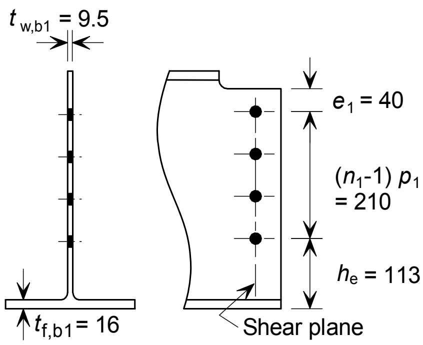
Basic requirement: $V_{Ed} \leq V_{Rd,min}$

$V_{Rd,min} = \min(V_{Rd,g};\ V_{Rd,n};\ V_{Rd,b})$

406 × 178 × 74 UKB, S275

Gross section

$$V_{Rd,g} = A_{v,wb}\frac{f_{y,b1}}{\sqrt{3}\gamma_{M0}}$$



Gross area: $A_v = A_{Tee} - bt_{f,b1} + (t_{w,b1} + 2r_{b1})\frac{t_{f,b1}}{2}$

A_{Tee} = (362.8 – 16) × 9.5 + 179.5 × 16
= 6167 mm^2

A_v = 6167 – 179.5 × 16 + (9.5 + 2 × 10.2) × $\frac{16}{2}$

= 3534 mm^2

$$\therefore V_{Rd,g} = 3534\times\frac{275}{\sqrt{3}\times 1.0}\times 10^{-3} = 561 \text{ kN}$$

Net section

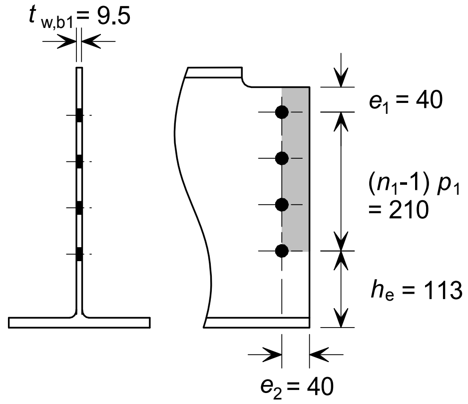
$$V_{Rd,n} = A_{v,net}\frac{f_{u,b1}}{\sqrt{3}\gamma_{M2}}$$

Net area: $A_{v,net} = A_v - n_1 d_0 t_{w,b1}$ = 3534 – 4 × 22 × 9.5 = 2698 mm^2

$$\therefore V_{Rd,n} = 2698\times\frac{410}{\sqrt{3}\times 1.1}\times 10^{-3} = 581 \text{ kN}$$

Title *Example 1 – Fin plates – Beam to beam* *Sheet 11 of 15*

Block tearing



$$V_{\text{Rd,b}} = \frac{0.5 f_{\text{u,b1}} A_{\text{nt}}}{\gamma_{\text{M2}}} + \frac{f_{\text{y,b1}} A_{\text{nv}}}{\sqrt{3}\gamma_{\text{M0}}}$$

Net area subject to tension:

$A_{\text{nt}} = t_{\text{w,b1}}(e_{2,\text{b}} - 0.5d_0)$

$= 9.5 \times (40 - 0.5 \times 22) \quad = 276\ \text{mm}^2$

Net area subject to shear:

$A_{\text{nv}} = t_{\text{w,b1}}(e_{1,\text{b}} + (n_1 - 1)p_1 - (n_1 - 0.5)d_0)$

$= 9.5 \times (40 + (4 - 1) \times 70 - (4 - 0.5) \times 22)$

$= 1644\ \text{mm}^2$

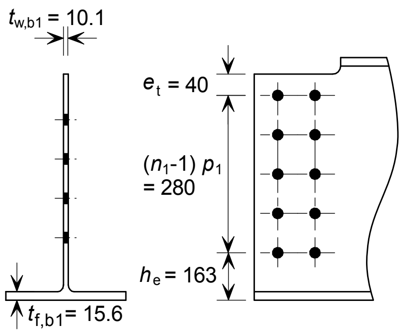
$$V_{\text{Rd,b}} = \left(\frac{0.5 \times 410 \times 276}{1.1} + \frac{275 \times 1644}{\sqrt{3} \times 1.0}\right) \times 10^{-3} \quad = 312\ \text{kN}$$

$V_{\text{Rd,min}} = \min(561;\ 581;\ 312) \quad = 312\ \text{kN}$

$\therefore\ V_{\text{Rd,min}} = 200\ \text{kN} \quad < \quad 312\ \text{kN}$ $\therefore$ **O.K.**

533 × 210 × 92 UKB, S275

Gross section



$$V_{\text{Rd,g}} = A_{\text{v}} \frac{f_{\text{y,b1}}}{\sqrt{3}\gamma_{\text{M0}}}$$

Gross area: $A_{\text{v}} = A_{\text{Tee}} - bt_{\text{f,b1}} + (t_{\text{w,b1}} + 2r_{\text{b1}})\dfrac{t_{\text{f,b1}}}{2}$

$A_{\text{Tee}} = (483.1 - 15.6) \times 10.1 + 209.3 \times 15.6$

$= 7987\ \text{mm}^2$

$A_{\text{v}} = 7987 - 209.3 \times 15.6 + (10.1 + 2 \times 12.7) \times \dfrac{15.6}{2}$

$= 4999\ \text{mm}^2$

$$\therefore\ V_{\text{Rd,g}} = 4999 \times \frac{275}{\sqrt{3} \times 1.0} \times 10^{-3} \quad = 794\ \text{kN}$$

Net section

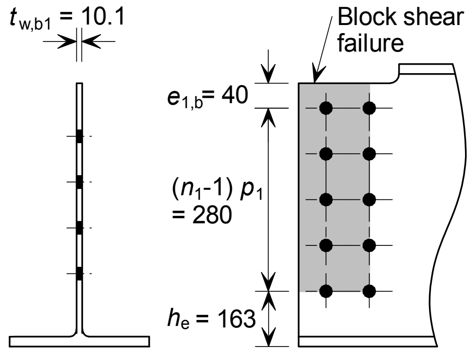
$$V_{\text{Rd,g}} = A_{\text{v,net}} \frac{f_{\text{u,b1}}}{\sqrt{3}\gamma_{\text{M2}}}$$

Net area: $A_{\text{v,net}} = A_{\text{v}} - n_1 d_0 t_{\text{w,b1}} \quad = 4999 - 5 \times 22 \times 10.1 \quad = 3888\ \text{mm}^2$

$$\therefore\ V_{\text{Rd,n}} = 3888 \times \frac{410}{\sqrt{3} \times 1.1} \times 10^{-3} \quad = 837\ \text{kN}$$

Block tearing

$$V_{\mathrm{Rd,b}} = \frac{0.5 f_{\mathrm{u,b1}} A_{\mathrm{nt}}}{\gamma_{\mathrm{M2}}} + \frac{f_{\mathrm{y,b1}} A_{\mathrm{nv}}}{\sqrt{3}\gamma_{\mathrm{M0}}}$$



Net area subject to tension:

$A_{\mathrm{nt}} = t_{\mathrm{w,b1}}(p_2 + e_{2,\mathrm{b}} - 1.5d_0)$

$= 10.1 \times (60 + 40 - 1.5 \times 22)$

$= 677\ \mathrm{mm}^2$

Net area subject to shear:

$A_{\mathrm{nv}} = t_{\mathrm{w,b1}}(e_{1,\mathrm{b}} + (n_1 - 1)p_1 - (n_1 - 0.5)d_0)$

$= 10.1 \times (40 + (5 - 1) \times 70 - (5 - 0.5) \times 22)$

$= 2232\ \mathrm{mm}^2$

$$V_{\mathrm{Rd,b}} = \left(\frac{0.5 \times 410 \times 677}{1.1} + \frac{275 \times 2232}{\sqrt{3} \times 1.0}\right) \times 10^{-3} \quad = 481\ \mathrm{kN}$$

$V_{\mathrm{Rd,min}}$ = min(794; 837; 481) = 481 kN

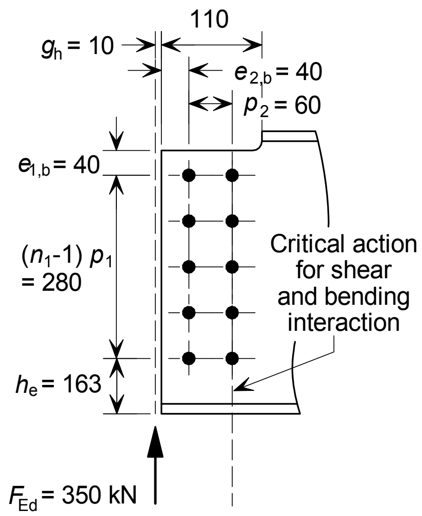
$\therefore\ V_{\mathrm{Rd,min}}$ = 350 kN < 481 kN **$\therefore$ O.K.**

Shear and bending interaction

406 × 178 × 74 UKB, S275

The connection has only one row of bolts (n_2 =1), and orthodox geometry, so the check is not required for this beam

533 × 210 × 92 UKB, S275



This check is applicable if $l_{\mathrm{n}} > (e_{2,\mathrm{b}} + p_2)$

$e_{2,\mathrm{b}} + p_2$ = 40 + 60 = 100 mm

l_{n} = 120 mm > 100 mm

$\therefore$ Interaction check required

Basic requirement:

$V_{\mathrm{Ed}}\,(g_{\mathrm{h}} + e_{2,\mathrm{b}} + p_2) \le M_{\mathrm{c,Rd}}$

$V_{\mathrm{Ed}}\,(g_{\mathrm{h}} + e_{2,\mathrm{b}} + p_2)$ = 350 (10 + 40 + 60)$\times 10^{-3}$

$\therefore\ V_{\mathrm{Ed}}\,(g_{\mathrm{h}} + e_{2,\mathrm{b}} + p_2)$ = 38.5 kNm

$V_{\mathrm{pl,N,Rd}}$ = min($V_{\mathrm{Rd,g}}$; $V_{\mathrm{Rd,n}}$) = min(794; 837)

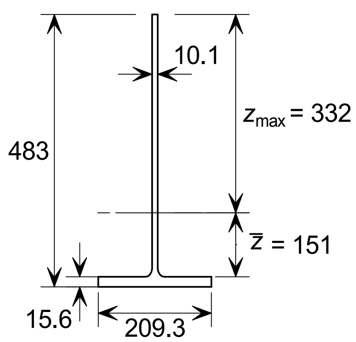
$\therefore\ V_{\mathrm{pl,N,Rd}}$ = 794 kN

$0.5V_{\mathrm{pl,N,Rd}}$ = 0.5 × 794 = 397 kN

V_{Ed} = 350 kN < 397 kN

$\therefore$ Low shear criteria for bending applies

$$M_{\mathrm{c,Rd}} = \frac{f_{\mathrm{y,b1}} W_{\mathrm{el,N}}}{\gamma_{\mathrm{M0}}}$$



For gross tee section:

Taking moments of area about the bottom of the section:

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

$\therefore\ \bar{z}$ = 151 mm

Second moment of area about the neutral axis:

$$I_{yy} = \left(\left(\frac{209.3\times15.6^3}{12}+209.3\times15.6\times143.2^2\right)+\left(\frac{10.1\times467.4^3}{12}+467.4\times10.1\times98.3^2\right)\right)\times10^{-4}$$

$\therefore\ I_{yy}$ = 19858 cm^4

$$W_{el,N} = \frac{I_{yy}}{z_{max}} = \frac{19858}{33.2} = 598\text{ cm}^3$$

Moment resistance of the beam at the notch in the presence of shear

$$M_{c,Rd} = \frac{598\times275}{1.0}\times10^{-3} = 164\text{ kNm}$$

$\therefore\ V_{Ed}\,(g_h + e_{2,b} + p_2) = 350\times(10+40+60)\times10^{-3} = 38.5$ kNm < 164 kNm $\therefore$ **O.K.**

Web resistance with long fin plates

For both connections, $t_p/0.15$ = 67 mm

For both connections, z_p = 50 mm. therefore the fin plates are not long and the resistance of the web needs no further check.

Check 5: Supported beam – Resistance at the notch

Basic requirement: $V_{Ed}\times(g_h + l_n) \le M_{v,N,Rd}$

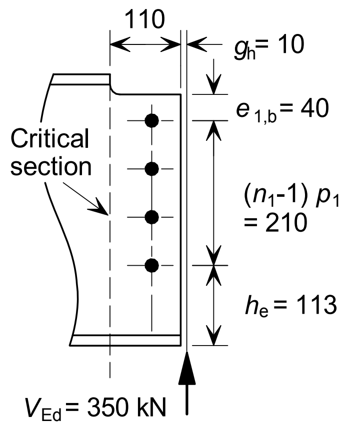
406 × 178 × 74 UKB, S275

$$F_{pl,N,Rd} = \frac{A_{v,N}f_{y,b1}}{\sqrt{3}\gamma_{M0}}$$

$$A_{v,N} = A_{Tee} - bt_{f,b1} + (t_{w,b1} + 2r_{b1})\frac{t_{f,b1}}{2}$$

= 3534 mm^2 from Check 4(i)

$$F_{pl,N,Rd} = \frac{3534\times275}{\sqrt{3}\times1.0}\times10^{-3} = 561\text{ kN}$$



$0.5\times F_{pl,N,Rd}$ = 0.5 × 561 kN = 281 kN

V_{Ed} = 200 kN < 281 kN

$\therefore$ Low shear criteria for bending applies

$$M_{v,N,Rd} = \frac{f_{y,b1}W_{el,N}}{\gamma_{M0}}$$

Taking moments of area about the bottom of the section:

(197.5 × 16 × 8) + (347 × 9.5 × 189.5) = ((179.5 × 16) + (347 × 9.5)) × $\bar{z}$

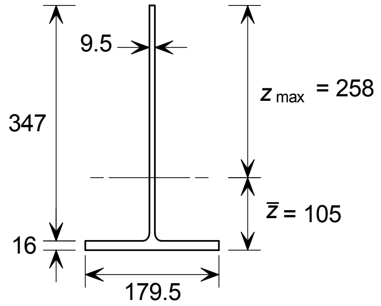
$\therefore\ \bar{z}$ = 105 mm

Second moment of area about the neutral axis:

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

$\therefore\ I_{yy}$ = 8370 cm^4

$$W_{el,N} = \frac{I_{yy}}{z_{max}} = \frac{8370}{25.8} = 324\ \text{cm}^3$$



Moment resistance of the beam at the notch in the presence of shear

$$M_{v,Rd} = \frac{324 \times 275}{1.0} \times 10^{-3} = 89.1\ \text{kNm}$$

$\therefore\ V_{Ed}\times(g_h + l_n)$ = 200 × (10 + 110) × 10^{-3} = 24 kNm < 89 kNm **∴ O.K.**

533 × 210 × 92 UKB, S275

Double bolt lines

x_N = 10 mm and $2d$ = 40 mm

Therefore, the basic requirement is:

$\max(V_{Ed}\times(g_h + l_n);\ V_{Ed}\times(g_h + e_{2,b} + p_2)) \leq M_{v,N,Rd}$

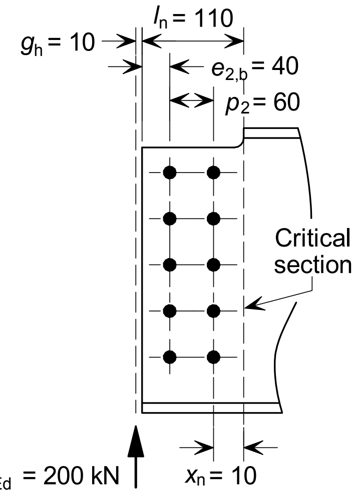
$(g_h + l_n)$ = 120 mm

$(g_h + e_{2,b} + p_2)$ = 10 + 40 + 60 = 110 mm

$M_{v,N,Rd}$ = 164 kNm from Check 4

$V_{Ed}\times(g_h + l_n)$ = 350 × 120 × 10^{-3} = 42 kNm

42 kNm < 164 kNm **∴ O.K.**



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 that:

For one flange notched beam in S275

$$d_{nt} \leq h_{b1}/2$$

and

$$l_n \leq h_{b1} \quad \text{for} \quad h_{b1}/t_{w,b1} \leq 54.3$$

$$l_n \leq \frac{160000\, h_{b1}}{\left(h_{b1}/t_{w,b1}\right)^3} \quad \text{for} \quad h_{b1}/t_{w,b1} > 54.3$$

406 × 178 × 74 UKB, S275

d_{nt} = 50 mm and $\frac{h_{b1}}{2} = \frac{412.8}{2}$ = 206.4 mm

50 < 206.4 **∴ O.K.**

$\frac{h_{b1}}{t_{w,b1}} = \frac{412.8}{9.5}$ = 43.4 < 54.3

l_n = 110 mm < 412.8 mm **∴ O.K.**

533 × 210 × 92 UKB, S275

d_{nt} = 50 mm and $\frac{h_{b1}}{2} = \frac{533.1}{2}$ = 266.6 mm

50 < 266.6 ∴ **O.K.**

$\frac{h_{b1}}{t_{w,b1}} = \frac{533.1}{10.1}$ = 52.8 < 54.3

l_n = 120 mm < 533.1 mm ∴ **O.K.**

Check 8: Supporting beam/column – Welds

For a beam in S275 steel

Basic requirement: $a \geq 0.5t_p$

$0.5t_p$ = 0.5 × 10 = 5 mm

∴ a = 5.7 mm > $0.5t_p$ ∴ **O.K.**

Check 10: Supporting beam – Shear and bearing (two supported beams)

Local shear

Basic requirement: $\frac{V_{Ed,tot}}{2} \leq F_{Rd}$

$$V_{Ed,tot} = \left(\frac{V_{Ed,1}}{h_{p,1}} + \frac{V_{Ed,2}}{h_{p,2}}\right) h_{p,min}$$

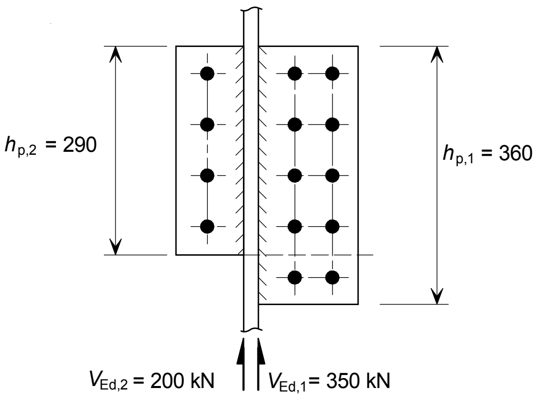
$$F_{Rd} = A_v \frac{f_{y,2}}{\sqrt{3}\gamma_{M0}}$$

A_v = $h_{p,min}t_2$ = 290 × 13.1

= 3799 mm²

$$F_{Rd} = 3799 \times \frac{275}{\sqrt{3} \times 1.0} \times 10^{-3}$$

= 603 kN



$$V_{Ed,1,A} = V_{Ed,1}\frac{h_{p,min}}{h_{p,max}} = 350 \times \frac{290}{360}$$

= 282 kN

$$V_{Ed,tot} = \left(\frac{200}{290} + \frac{350}{360}\right) \times 290 = 482 \text{ kN}$$

$\frac{482}{2}$ = 241 kN < 603 kN ∴ **O.K.**

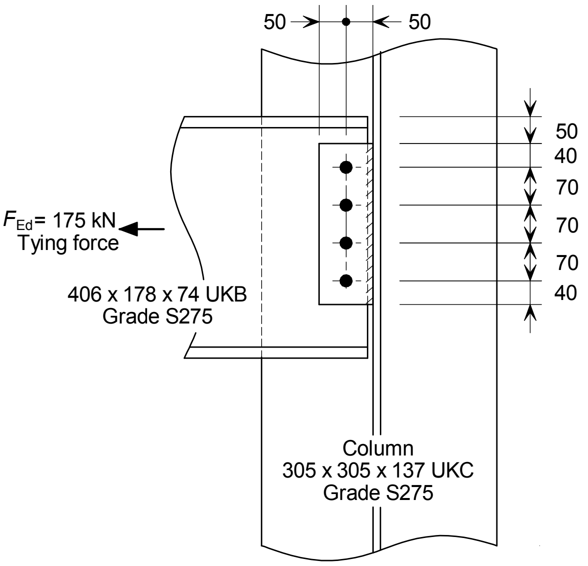
Punching shear

This check is not necessary as the beam is double sided

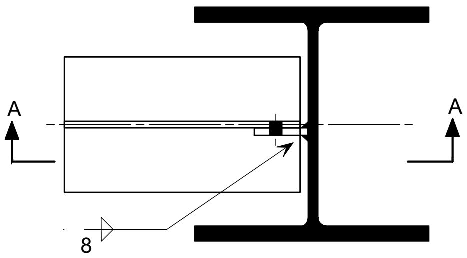
SCI Steel Knowledge CALCULATION SHEET BCSA	Job: *Joints in Steel Construction - Simple Joints*		Sheet 1 of 6
	Title: *Example 2 – Fin plates – Beam to column web – Tying resistance*		
	Client: *Connections Group*		
	Calcs by: *ENM*	Checked by: *DGB*	Date: *Sept 2011*

DESIGN EXAMPLE 2

Check the following beam to column connection for the tying force shown.



Section AA



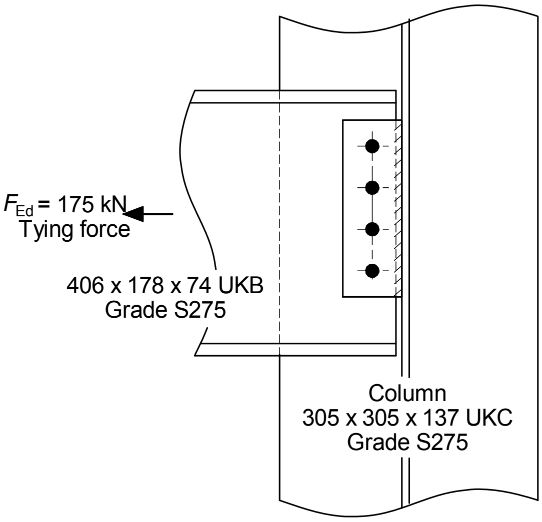
Design information:

Table G.18

Bolts:	M20, 8.8
Welds:	8 mm leg length fillet welds
Material:	All S275
Fin plates:	100 × 10

Title	*Example 2 – Fin Plates – Beam to column web – Tying resistance*	*Sheet 2 of 6*

CONNECTION DESIGN USING RESISTANCE TABLES



Fin plate: 100 × 10, S275

Bolts: M20, 8.8

8 mm fillet welds *(Table G.18)*

From resistance tables G.18:

Connection tying resistance = 428 kN < 175 kN

∴ Beam side of connection is adequate **∴ O.K.**

Note:

(1) The tying resistance of the connection given in the table in the yellow pages is the lesser of the values obtained from Checks 11 and 12.

(2) When beams connect into a column web, the column web must also be checked for column web bending as shown in Check 14.

Title Example 2 – Fin Plates – Beam to column web – Tying resistance *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 b

olt resistance tables in the yellow pages.

Sheet No.	CHECK		406UKB S275 Beam		305UKC S275 Column	
			Resistance	Design force	Resistance	Design force
4	***Check 11*** Tying resistance Plate and bolts	Tension (kN)	661	175	Not applicable	
		Shear (kN)	428	175		
		Bearing (kN)	560	175		
5	***Check 12*** Tying resistance Supported beam web	Tension (kN)	597	175	Not applicable	
		Bearing (kN)	432	175		
	Check 13 Tying resistance Welds		Not applicable		Not applicable (Check 8 governs)	
6	***Check 14*** Tying resistance Supporting column web	(kN)	Not applicable		412	175

Title *Example 2 – Fin Plates – Beam to column web – Tying resistance* *Sheet 4 of 6*

CONNECTION DESIGN FOLLOWING THE DESIGN PROCEDURES

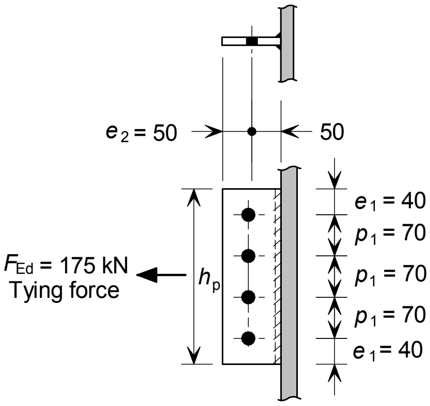
Check 11: Tying resistance – Plate and bolts

Tension resistance of fin plate

Basic requirement: $F_{\mathrm{Ed}} \leq F_{\mathrm{Rd,u}}$

$F_{\mathrm{Rd,u}} = \min(F_{\mathrm{Rd,b}};\ F_{\mathrm{Rd,n}})$

Block tearing:



$$F_{\mathrm{Rd,b}} = \frac{f_{\mathrm{u,p}} A_{\mathrm{nt}}}{\gamma_{\mathrm{Mu}}} + \frac{f_{\mathrm{y,p}} A_{\mathrm{nv}}}{\sqrt{3}\gamma_{\mathrm{M0}}}$$

$$A_{\mathrm{nt}} = t_{\mathrm{p}}\left(\left(n_1 - 1\right)p_1 - \left(n_1 - 1\right)d_0\right)$$

$$= 10 \times \left(\left(4-1\right)\times 70 - \left(4-1\right)\times 22\right)$$

$$= 1440 \text{ mm}^2$$

$$A_{\mathrm{nv}} = 2t_{\mathrm{p}}\left(e_2 - \frac{d_0}{2}\right)$$

$$= 2 \times 10 \times \left(50 - \frac{22}{2}\right) = 780 \text{ mm}^2$$

$$F_{\mathrm{Rd,b}} = \left(\frac{410 \times 1440}{1.1} + \frac{275 \times 780}{\sqrt{3} \times 1.0}\right) \times 10^{-3} = 661 \text{ kN}$$

Net tension:

$$F_{\mathrm{Rd,n}} = 0.9 A_{\mathrm{net,p}} \frac{f_{\mathrm{u,p}}}{\gamma_{\mathrm{Mu}}}$$

$$A_{\mathrm{net,p}} = t_{\mathrm{p}}\left(h_{\mathrm{p}} - d_0 n_1\right) = 10 \times \left(290 - 22 \times 4\right) = 2020 \text{ mm}^2$$

$$F_{\mathrm{Rd,n}} = 0.9 \times 2020 \times \frac{410}{1.1} \times 10^{-3} = 678 \text{ kN}$$

$F_{\mathrm{Rd,u}} = \min(661;\ 678) = 661$ kN

$F_{\mathrm{Ed}} = 175$ kN $<$ 661 kN **∴ O.K.**

Bolt shear

Basic requirement: $F_{\mathrm{Ed}} \leq F_{\mathrm{Rd,u}}$

$F_{\mathrm{Rd,u}} = nF_{\mathrm{v,u}}$

$$F_{\mathrm{v,u}} = \frac{\alpha_{\mathrm{v}} f_{\mathrm{ub}} A}{\gamma_{\mathrm{Mu}}} \qquad \frac{0.6 \times 800 \times 245}{1.1} \times 10^{-3} = 107 \text{ kN}$$

$F_{\mathrm{Rd,u}} = 4 \times 107 = 428$ kN

$F_{\mathrm{Ed}} = 175$ kN $<$ 428 kN **∴ O.K.**

Bolt bearing in the fin plate

Basic requirement: $F_{\mathrm{Ed}} \leq F_{\mathrm{Rd,u}}$

$F_{\mathrm{Rd,u}} = n\, F_{\mathrm{b,hor,Rd,u}}$

$$F_{\mathrm{b,hor,Rd,u}} = \frac{k_1 \alpha_{\mathrm{b}} f_{\mathrm{u,p}} d t_{\mathrm{p}}}{\gamma_{\mathrm{Mu}}}$$

$$k_1 = \min\left(2.8\frac{e_1}{d_0} - 1.7;\quad 1.4\frac{p_1}{d_0} - 1.7;\quad 2.5\right) = \min\left(2.8\frac{40}{22} - 1.7;\quad 1.4\frac{70}{22} - 1.7;\quad 2.5\right)$$

$$= \min\left(3.39;\quad 2.75;\quad 2.5\right) = 2.5$$

$$\alpha_b = \min\left(\frac{e_2}{3d_0};\quad \frac{f_{ub}}{f_{u,p}};\quad 1.0\right) = \min\left(\frac{50}{3\times 22};\quad \frac{800}{410};\quad 1.0\right)$$

$$= \min(0.75;\quad 1.95;\quad 1.0) = 0.75$$

$$\therefore F_{b,hor,Rd,u} = \frac{2.5\times 0.75\times 410\times 20\times 10}{1.1}\times 10^{-3} = 140 \text{ kN}$$

$F_{Rd,u} = 4 \times 140 = 560$ kN

$F_{Ed} = 175$ kN $<$ 560 kN ∴ **O.K.**

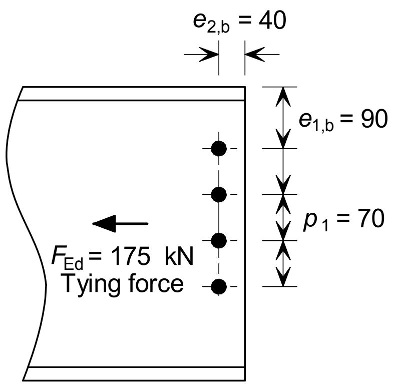
Check 12: Tying resistance – Supported beam web

Tension

Basic requirement: $F_{Ed} \leq F_{Rd,u}$

Block tearing:

$F_{Rd,u} = \min(F_{Rd,b};\quad F_{Rd,n})$



$$F_{Rd,b} = \frac{A_{nt} f_{u,b1}}{\gamma_{Mu}} + \frac{A_{nv} f_{y,b1}}{\sqrt{3}\gamma_{M0}}$$

$$A_{nt} = t_{w,b1}\left((n_1-1)p_1 - (n_1-1)d_0\right)$$

$$= 9.5\times\left((4-1)\times 70 - (4-1)\times 22\right)$$

$$= 1368 \text{ mm}^2$$

$$A_{nt} = 2t_{w,b1}\left(e_{2,b} - \frac{d_0}{2}\right)$$

$$= 2\times 9.5\times\left(40 - \frac{22}{2}\right) = 551 \text{ mm}^2$$

$$F_{Rd,b} = \left(\frac{1368\times 410}{1.1} + \frac{551\times 275}{\sqrt{3}\times 1.0}\right)\times 10^{-3} = 597 \text{ kN}$$

Net tension:

$$F_{Rd,n} = 0.9 A_{net,wb}\frac{f_{u,b1}}{\gamma_{Mu}}$$

$$A_{net,wb} = t_{w,b1}h_{wb} - d_0 n_1 t_{w,b1} = 9.5\times 290 - 22\times 4\times 9.5 = 1919 \text{ mm}^2$$

$$F_{Rd,n} = 0.9\times 1919\times\frac{410}{1.1}\times 10^{-3} = 644 \text{ kN}$$

$F_{Rd,u} = \min(597;\quad 644) = 597$ kN

$F_{Ed} = 175$ kN $<$ 597 kN ∴ **O.K.**

Bearing

Basic requirement: $F_{Ed} \leq F_{Rd,u}$

$F_{Rd,u} = nF_{b,hor,Rd,u}$

$$F_{b,hor,Rd,u} = \frac{k_1\alpha_b f_{u,b1} d t_{w,b1}}{\gamma_{Mu}}$$

$$k_1 = \left(1.4\frac{p_1}{d_0} - 1.7;\quad 2.5\right) = \left(1.4\frac{70}{22} - 1.7;\quad 2.5\right) = (2.75;\quad 2.5) = 2.5$$

$$\alpha_b = \left(\frac{e_{2,b}}{3d_0};\quad \frac{f_{ub}}{f_{u,b1}};\quad 1.0\right) = \left(\frac{40}{3\times 22};\quad \frac{800}{410};\quad 1.0\right) = (0.61;\quad 1.95;\quad 1.0) = 0.61$$

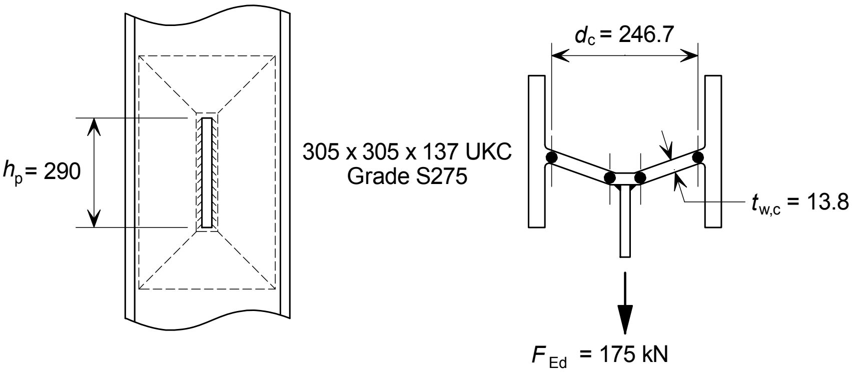
Title	Example 2 – Fin Plates – Beam to column web – Tying resistance	Sheet 6 of 6

$$F_{b,hor,Rd,u} = \frac{2.5 \times 0.61 \times 410 \times 20 \times 9.5}{1.1} \times 10^{-3} = 108 \text{ kN}$$

$F_{Rd,u} = 4 \times 108 = 432$ kN

$F_{Ed} = 175$ kN $<$ 432 kN

Check 14: Tying resistance – Supporting column web



Basic requirement: $F_{Ed} \leq F_{Rd,u}$

$$F_{Rd,u} = \frac{8 M_{pl,Rd,u}}{\gamma_{Mu}\left(1-\beta_1\right)}\left(\eta_1 + 1.5\left(1-\beta_1\right)^{0.5}\right)$$

$$M_{pl,Rd,u} = \frac{1}{4} f_{u,2} t_{w,2}^{\ 2} = 0.25 \times 410 \times 13.8^2 \times 10^{-3} = 19.5 \text{ kNm/mm}$$

$$\eta_1 = \frac{h_p}{d_2} = \frac{290}{246.7} = 1.18$$

$$\beta_1 = \frac{t_p + 2s}{d_2} = \frac{10 + 2 \times 8}{246.7} = 0.105$$

$$F_{Rd,u} = \frac{8 \times 19.5}{1.1 \times \left(1 - 0.105\right)} \times \left(1.18 + 1.5\left(1 - 0.105\right)^{0.5}\right) = 412 \text{ kN}$$

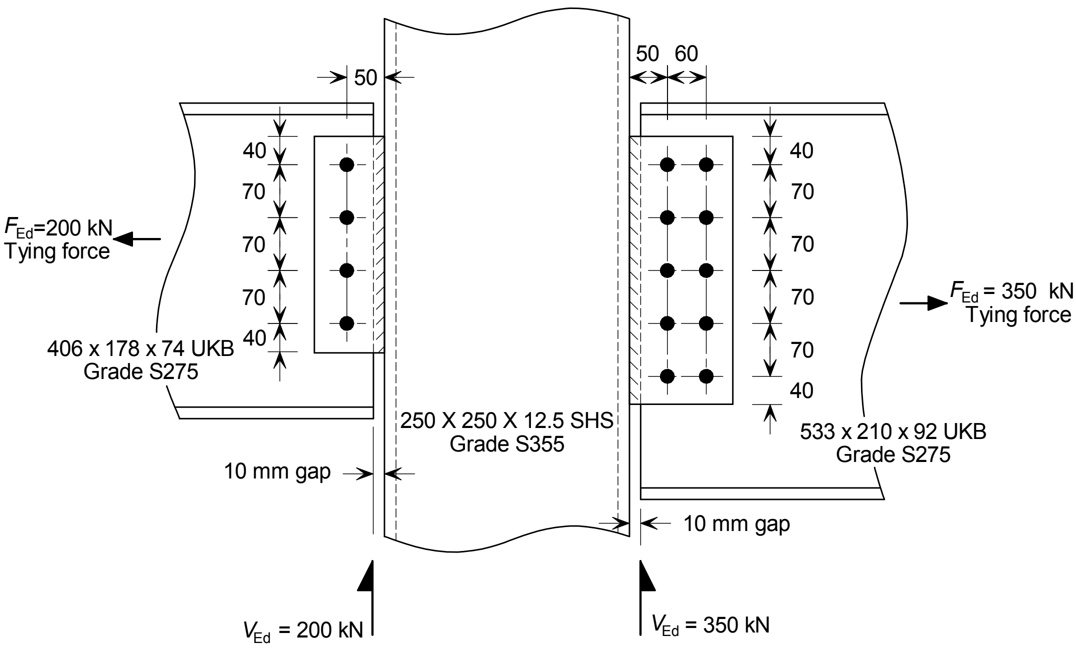
$F_{Ed} = 175$ kN $<$ 412 kN **∴ O.K.**

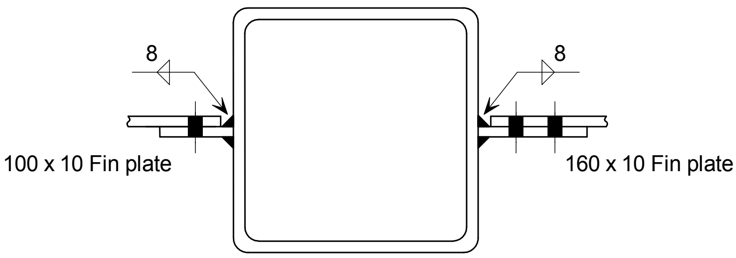
Note: If column web fails to satisfy the above criteria then the column should be strengthened

SCI Steel Knowledge **CALCULATION SHEET** BCSA	Job	*Joints in Steel Construction – Simple Joints*		*Sheet 1 of 9*
	Title	*Example 3 – Fin plates – Beam to hollow section column*		
	Client	*Connections Group*		
	Calcs by *ENM*	Checked by *DGB*	Date *Sept 2011*	

DESIGN EXAMPLE 3

Check the following beam to hollow section column connection for the design forces shown. In this example it is assumed that the tying force is equal to the end reaction.



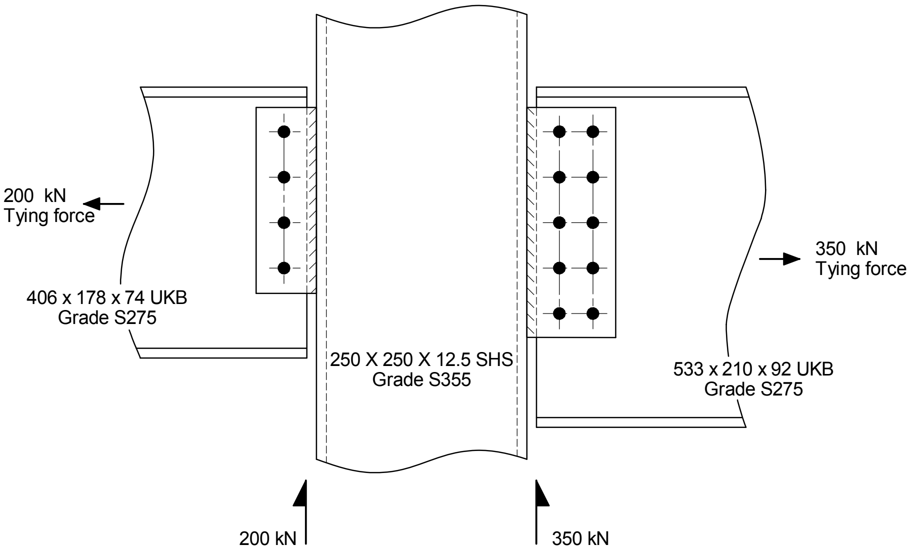


Design information:

Bolts:	M20, 8.8
Welds:	8 mm leg length fillet welds
Column:	S355
Beams:	S275
Fin plates:	S275

Title Example 3 – Fin Plates – Beam to hollow section column	Sheet 2 of 9

CONNECTION DESIGN USING RESISTANCE TABLES



406 × 178 × 74 UKB Grade S275

Fin plate, 100 × 10, S275

Welds 8 mm fillet

Bolts M20 8.8

4 rows of bolts

From Resistance Table G.18

Connection shear resistance (un-notched)
= 286 kN > 200 kN

Maximum notch length
N/A

Minimum support thickness in S355:
2.7 mm < 12.5 mm

Connection tying resistance:
428 kN > 200 kN

Beam side of connection is adequate

533 × 210 × 92 UKB Grade S275

Fin plate, 150 × 10, S275

Welds 8 mm fillet

Bolts M20 8.8

5 rows of bolts

From Resistance Table G.19

Connection shear resistance (un-notched)
= 450 kN > 350 kN

Maximum notch length
N/A

Minimum support thickness in S355:
3.4 mm < 12.5 mm

Connection tying resistance:
839 kN > 350 kN

Beam side of connection is adequate

Tables G.18 and G.19

Title *Example 3 – Fin plates – Beam to hollow section column* *Sheet 3 of 9*

SUMMARY OF FULL DESIGN CHECKS FOR EXAMPLE 3

Notes:

(1) Checks 1 to 9, where applicable, are all as shown in Example 1 and are not repeated in this example, but the calculated resistances are summarised below. Check 4 resistances are higher because the beams in this example are not notched.

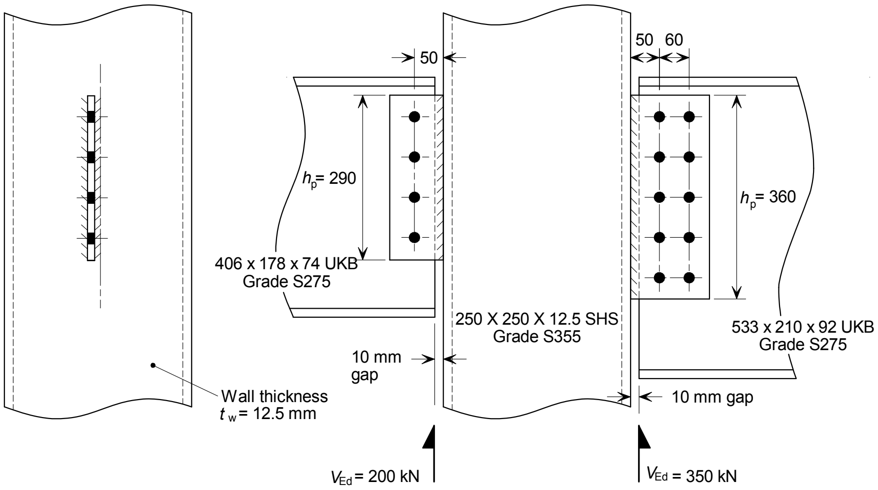
(2) In accordance with BS EN 1991-1-7, tying forces are ignored when checking the resistance to shear forces and shear forces are ignored when calculating the resistance to tying forces.

Sheet No.	CHECK		406 UKB (S275)		533 UKB (S275)		SHS Column (S355) 406 UKB side		SHS Column (S355) 533 UKB side	
			Resistance	**Design force**	**Resistance**	**Design force**	**Resistance**	**Design force**	**Resistance**	**Design force**
See Example 1	***Check 1*** Recommended detailing practice		All recommendations adopted							
	Check 2 Supported beam Bolt group	Shear (kN)	288	200	577	350	Not applicable			
		Fin plate in bearing (kN)	332	200	586	350				
		Beam web in bearing (kN)	291	200	565	350				
	Check 3 Supported beam Fin plate	Shear (kN)	347	200	450	350	Not applicable			
		Bending resistance (kN)	∞	200	∞	350				
		LTB of fin plate	771	200	742	350				
	Check 4 Supported beam Web in shear	Shear (kN)	664	200	909	350	Not applicable			
		Bending resistance (kNm)	NA	NA	NA	NA				
		Long fin plate (kN)	NA	NA	NA	NA				
	Checks 5, 6, 7		Not applicable				Not applicable			
	Check 8 Supporting column Welds	Weld throat, *a* (mm)	Not applicable				5.7	5.0	5.7	5.0
	Check 9		Not applicable				Not applicable			
4	***Check 10*** Supporting column Shear and bearing	Shear (kN)	Not applicable				743	100	922	175
		Punching shear					t_p is adequate			
5	***Check 11*** Structural integrity Plate and bolts	Tension (kN)	661	200	839	350	Not applicable			
		Bolt shear (kN)	428	200	1070	350				
		Bearing (kN)	560	200	1230	350				
7	***Check 12*** Structural integrity Supported beam web	Tension (kN)	597	200	847	350	Not applicable			
		Bearing (kN)	432	200	1150	350				
	Checks 13, 14		Not applicable				Not applicable			
8	***Check 15*** Structural integrity Supporting column wall	Tension (kN)	Not applicable				422	200	472	350

Title Example 3 – Fin plates – Beam to hollow section column | Sheet 4 of 9

CONNECTION DESIGN FOLLOWING THE DESIGN PROCEDURES

Check 10: Supporting column – shear and bearing



Shear and punching shear resistance of column wall

Local shear

Basic requirement: $\frac{V_{Ed}}{2} \leq V_{Rd}$

$$V_{Rd} = \frac{A_v f_{y,2}}{\sqrt{3}\gamma_{M0}}$$

406 × 178 × 74 UKB, S275

$A_v = h_p t_2 = 290 \times 12.5 = 3625 \text{ mm}^2$

$$\therefore V_{Rd} = \frac{3625 \times 355}{\sqrt{3} \times 1.0} \times 10^{-3} = 743 \text{ kN}$$

$\therefore \frac{V_{Ed}}{2} = 100 \text{ kN} < 743 \text{ kN}$ $\therefore$ **OK**

533 × 210 × 92 UKB, S275

$A_v = h_p t_2 = 360 \times 12.5 = 4500 \text{ mm}^2$

$$\therefore V_{Rd} = \frac{4500 \times 355}{\sqrt{3} \times 1.0} \times 10^{-3} = 922 \text{ kN}$$

$\therefore \frac{V_{Ed}}{2} = 175 \text{ kN} < 922 \text{ kN}$ $\therefore$ **OK**

Punching shear

Basic requirement (conservative approach): $t_p \leq t_2 \times \frac{f_{u,2}}{f_{y,p}}$

$$t_2 \times \frac{f_{u,2}}{f_{y,p}} = 12.5 \times \frac{470}{275} = 21.4 \text{ mm}$$

$t_p = 10 \text{ mm} < 21.4 \text{ mm}$ $\therefore$ **OK**

Check 11: Structural integrity – plate and bolts

Tension

Basic requirement: $F_{Ed} \leq F_{Rd,u}$

$F_{Rd,u} = \min\left(F_{Rd,b};\ F_{Rd,n}\right)$

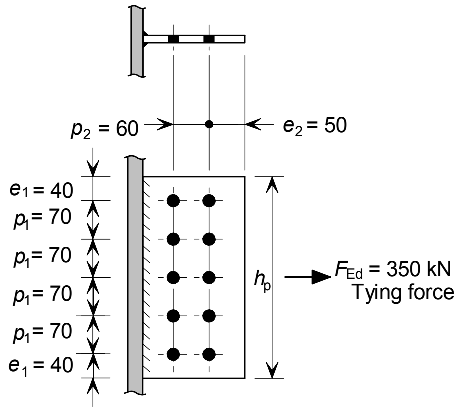
406 × 178 × 74 UKB, S275

See example 2.

533 × 210 × 92 UKB, S275

Hole diameter: d_0 = 22 mm

Fin plate thickness: t_p = 10 mm



Block tearing, $F_{Rd,b} = \dfrac{f_{u,p}A_{nt}}{\gamma_{Mu}} + \dfrac{f_{y,p}A_{nv}}{\sqrt{3}\gamma_{M0}}$

For Case 1:

$A_{nt} = t_p\left((n_1-1)p_1-(n_1-1)d_0\right) = 10\times\left((5-1)\times 70-(5-1)\times 22\right) = 1920\text{ mm}^2$

$A_{nv} = 2t_p\left(p_2+e_2-3\dfrac{d_0}{2}\right) = 2\times 10\times\left(60+50-3\times\dfrac{22}{2}\right) = 1540\text{ mm}^2$

$\therefore F_{Rd,b} = \left(\dfrac{410\times 1920}{1.1}+\dfrac{275\times 1540}{\sqrt{3}\times 1.0}\right)\times 10^{-3} = 960\text{ kN}$

For Case 2:

$A_{nt} = t_p\left((n_1-1)p_1-(n_1-0.5)d_0+e_1\right) = 10\times\left((5-1)\times 70-(5-0.5)\times 22+40\right) = 2210\text{ mm}^2$

$A_{nv} = t_p\left(p_2+e_2-3\dfrac{d_0}{2}\right) = 10\times\left(60+50-3\times\dfrac{22}{2}\right) = 770\text{ mm}^2$

$\therefore F_{Rd,b} = \left(\dfrac{410\times 2210}{1.1}+\dfrac{275\times 770}{\sqrt{3}\times 1.0}\right)\times 10^{-3} = 946\text{ kN}$

Net tension, $F_{Rd,n} = \dfrac{0.9A_{net,p}f_{u,p}}{\gamma_{Mu}}$

$A_{net,p} = t_p\left(h_p-d_0n_1\right) = 10\times\left(360-22\times 5\right) = 2500\text{ mm}^2$

$\therefore F_{Rd,n} = \dfrac{0.9\times 2500\times 410}{1.1}\times 10^{-3} = 839\text{ kN}$

$\therefore F_{Rd,u} = \min\left(960;\ 946;\ 839\right) = 839\text{ kN}$

$\therefore F_{Ed}$ = 350 kN < 839 kN **∴ OK**

Title *Example 3 – Fin plates – Beam to hollow section column* *Sheet 6 of 9*

Bolt shear

Basic requirement: $F_{Ed} \leq F_{Rd,u}$

$F_{Rd,u} = nF_{v,u}$

$$F_{v,u} = \frac{\alpha_v f_{ub} A}{\gamma_{Mu}}$$

406 × 178 × 74 UKB, S275

$\alpha_v = 0.6$

$f_{ub} = 800$ N/mm^2

$A = 245$ mm^2

$$F_{v,u} = \frac{0.6 \times 800 \times 245}{1.1} \times 10^{-3} = 107 \text{ kN}$$

$\therefore F_{Rd,u} = 4 \times 107 = 428$ kN

$\therefore F_{Ed} = 200$ kN $<$ 428 kN — $\therefore$ **OK**

533 × 210 × 92 UKB, S275

$F_{v,u} = 107$ kN (see calculation for 406 × 178 × 74 UKB)

$\therefore F_{Rd,u} = 10 \times 107 = 1070$ kN

$\therefore F_{Ed} = 350$ kN $<$ 1070 kN — $\therefore$ **OK**

Bearing

Basic requirement: $F_{Ed} \leq F_{Rd,u}$

$F_{Rd,u} = \sum F_{b,hor,u,Rd}$

$$F_{b,hor,u,Rd} = \frac{k_1 \alpha_b f_{u,p} d t_p}{\gamma_{Mu}}$$

406 × 178 × 74 UKB, S275

See Example 2.

533 × 210 × 92 UKB, S275

$$k_1 = \min\left(2.8\frac{e_1}{d_0} - 1.7;\quad 1.4\frac{p_1}{d_0} - 1.7;\quad 2.5\right) = \min\left(2.8 \times \frac{40}{22} - 1.7;\quad 1.4 \times \frac{70}{22} - 1.7;\quad 2.5\right)$$

$$= \min\left(3.39;\quad 2.75;\quad 2.5\right) = 2.5$$

$$\alpha_b = \min\left(\frac{e_2}{3d_0};\quad \frac{p_2}{3d_0} - \frac{1}{4};\quad \frac{f_{ub}}{f_{u,p}};\quad 1.0\right) = \min\left(\frac{50}{3 \times 22};\quad \frac{60}{3 \times 22} - \frac{1}{4};\quad \frac{800}{410};\quad 1.0\right)$$

$$= \min\left(0.75;\quad 0.66;\quad 1.95;\quad 1.0\right) = 0.66$$

$$\therefore F_{b,hor,u,Rd} = \frac{2.5 \times 0.66 \times 410 \times 20 \times 10}{1.1} \times 10^{-3} = 123 \text{ kN}$$

$\therefore F_{Rd,u} = 10 \times 123 = 1230$ kN

$\therefore F_{Ed} = 350$ kN $<$ 1230 kN — $\therefore$ **OK**

Check 12: Structural integrity – supported beam web

Tension, shear and bearing resistance of the beam web

Tension resistance of beam web

Basic requirement: $F_{Ed} \leq F_{Rd,u}$

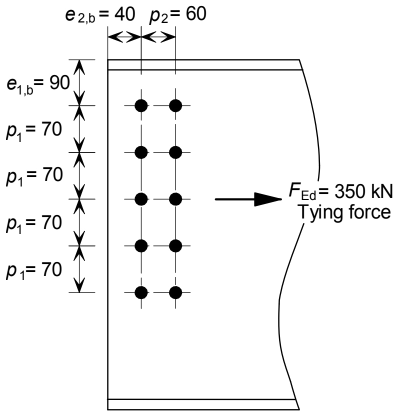
$F_{Rd,u} = \min(F_{Rd,b};\ F_{Rd,n})$

406 × 178 × 74 UKB, S275

See Example 2

533 × 210 × 92 UKB, S275

Block tearing, $F_{Rd,b} = \dfrac{A_{nt} f_{u,b1}}{\gamma_{Mu}} + \dfrac{A_{nv} f_{y,b1}}{\sqrt{3}\gamma_{M0}}$



$A_{nt} = t_{w,b1}\left((n_1-1)p_1-(n_1-1)d_0\right)$

$= 10.1\times\left((5-1)\times 70-(5-1)\times 22\right)$

$= 1939\ mm^2$

$A_{nv} = 2t_{w,b1}\left(p_2+e_{2,b}-3\dfrac{d_0}{2}\right)$

$= 2\times 10.1\times\left(60+40-3\times\dfrac{22}{2}\right)$

$= 1353\ mm^2$

$\therefore F_{Rd,b} = \left(\dfrac{410\times 1939}{1.1}+\dfrac{275\times 1353}{\sqrt{3}\times 1.0}\right)\times 10^{-3} = 938\ kN$

Net tension, $F_{Rd,n} = \dfrac{0.9 A_{net,b1} f_{u,b1}}{\gamma_{Mu}}$

$A_{net,p} = t_{w,b1}\left(h_{w,b1}-d_0 n_1\right) = 10.1\times\left(360-22\times 5\right) = 2525\ mm^2$

$\therefore F_{Rd,n} = \dfrac{0.9\times 2525\times 410}{1.1}\times 10^{-3} = 847\ kN$

$\therefore F_{Rd,u} = \min\left(938;\ 847\right) = 847\ kN$

$F_{Ed} = 350\ kN < 847\ kN$ **∴ OK**

Bolt bearing in beam web

Basic requirement: $F_{Ed} \leq F_{Rd,u}$

$F_{Rd,u} = n\, F_{b,hor,u,Rd}$

$F_{b,hor,u,Rd} = \dfrac{k_1 \alpha_b f_{u,b1} d t_{w,b1}}{\gamma_{Mu}}$

406 × 178 × 74 UKB, S275

See Example 2

Title *Example 3 – Fin plates – Beam to hollow section column* *Sheet 8 of 9*

533 × 210 × 92 UKB, S275

$k_1 \quad = \min\left(1.4\dfrac{p_1}{d_0} - 1.7;\quad 2.5\right) \qquad = \min\left(1.4 \times \dfrac{70}{22} - 1.7;\quad 2.5\right)$

$\quad = \min(2.75;\quad 2.5) \qquad = 2.5$

$\alpha_b \quad = \min\left(\dfrac{e_{2,b}}{3d_0};\quad \dfrac{p_2}{3d_0} - \dfrac{1}{4};\quad \dfrac{f_{ub}}{f_{u,b1}};\quad 1.0\right) \qquad = \min\left(\dfrac{40}{3\times 22};\quad \dfrac{60}{3\times 22} - \dfrac{1}{4};\quad \dfrac{800}{410};\quad 1.0\right)$

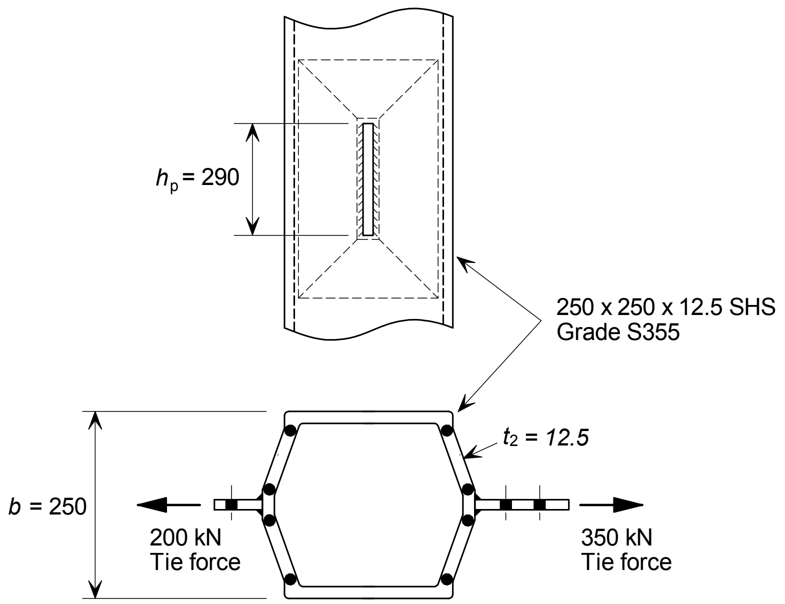
$\quad = \min(0.61;\quad 0.65;\quad 1.95;\quad 1.0) \quad = 0.61$

$\therefore F_{b,hor,u,Rd} \quad = \dfrac{2.5 \times 0.61 \times 410 \times 20 \times 10.1}{1.1} \times 10^{-3} \qquad = 115 \text{ kN}$

$\therefore F_{Rd,u} \quad = 10 \times 115 \quad = 1150 \text{ kN}$

$\therefore F_{Ed} = 350 \text{ kN} \quad < \quad 1150 \text{ kN}$ **∴ OK**

Check 15: Structural integrity – supporting column wall



Basic requirement: $\qquad F_{Ed} \quad \leq \quad F_{Rd,u}$

$F_{Rd,u} \quad = \dfrac{8M_{pl,Rd,u}}{(1-\beta)\gamma_{Mu}}\left(\eta + 1.5(1-\beta)^{0.5}\right)$

$M_{pl,Rd,u} = \dfrac{1}{4} f_{u,2} t_2^{\,2} \quad = \dfrac{1}{4} \times 470 \times 12.5^2 \times 10^{-3} \qquad = 18.4 \text{ kNm/mm}$

$\beta \quad = \dfrac{t_p + 2s}{h_2 - 3t_2} \quad = \dfrac{10 + 2 \times 8}{250 - 3 \times 12.5} \qquad = 0.122$

406 × 178 × 74 UKB, S275

$\eta \quad = \dfrac{h_p}{h_2 - 3t_2} \quad = \dfrac{290}{250 - 3 \times 12.5} \qquad = 1.365$

$\therefore F_{Rd,u} \quad = \dfrac{8 \times 18.4}{(1 - 0.122) \times 1.1}\left(1.365 + 1.5(1 - 0.122)^{0.5}\right) \qquad = 422 \text{ kN}$

$\therefore F_{Ed} = 200 \text{ kN} \quad < \quad 422 \text{ kN}$ **∴ OK**

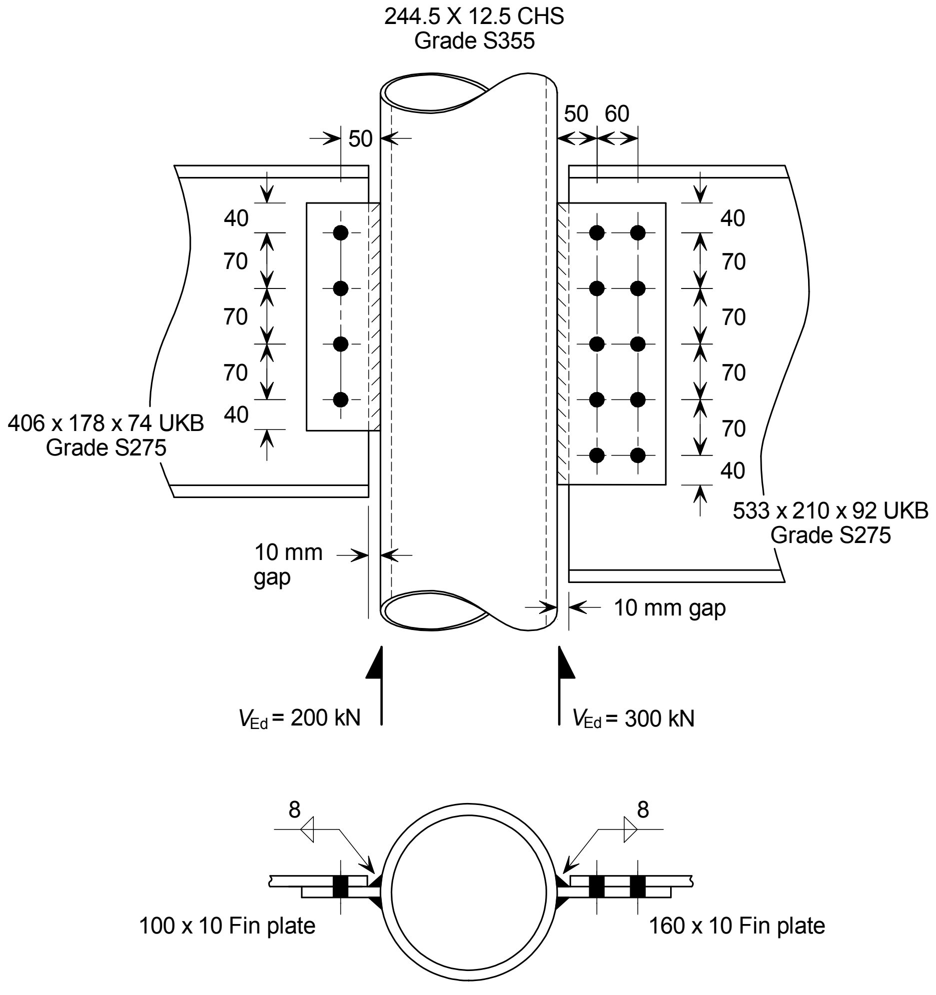
Title Example 3 – Fin plates – Beam to hollow section column	Sheet 9 of 9
533 × 210 × 92 UKB, S275	
$\eta = \dfrac{h_p}{h_2 - 3t_2} = \dfrac{360}{250 - 3\times 12.5} = 1.694$	
$\therefore F_{Ed,u} = \dfrac{8\times 18.4}{(1-0.122)\times 1.1}\left(1.694 + 1.5\times(1-0.122)^{0.5}\right) = 472$ kN	
$\therefore F_{Ed} = 350$ kN $<$ 472 kN	$\therefore$ **OK**

SCI Steel Knowledge **CALCULATION SHEET** BCSA	Job *Joints in Steel Construction – Simple Joints*		Sheet *1* of *5*
	Title *Example 4 – Fin plates – Beam to CHS column*		
	Client *Connections Group*		
	Calcs by *CZT*	Checked by *ENM*	Date *Sept 2011*

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.

Note: The connections should be checked independently for shear forces and tying forces.

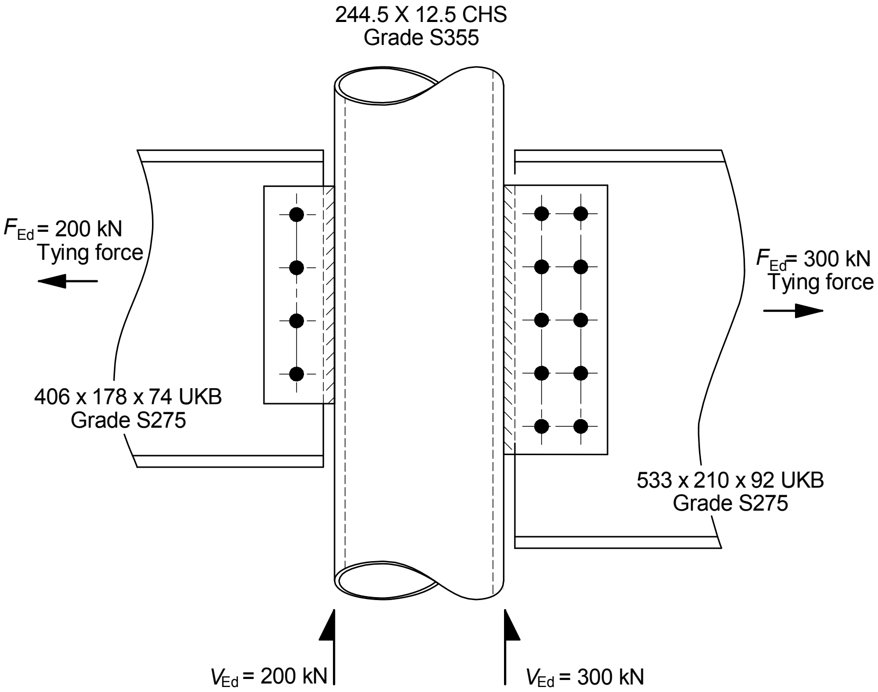


Design Information:

Bolts:	M20 8.8
Welds:	8 mm fillet weld
Column:	S355
Beams:	S275
Fin plates:	S275

Title Example 4 – Fin plates – Beam to CHS column	Sheet 2 of 5

CONNECTION DESIGN USING RESISTANCE TABLES



406 × 178 × 74 UKB, S275	533 × 210 × 92 UKB, S275
Fin plate, S275	**Fin plate, S275**
Bolts M20 8.8	Bolts M20 8.8
4 rows of bolts	5 rows of bolts
From Resistance Table G.18	From Resistance Table G.19
Connection shear resistance = 286 kN > 200 kN	Connection shear resistance = 450 kN > 300 kN
Max notch length *N/A*	Max notch length *N/A*
Minimum support thickness in S355 = 2.7 mm < 12.5 mm	Minimum support thickness in S355 = 3.4 mm < 12.5 mm
Connecting tying resistance = 428 kN > 200 kN	Connecting tying resistance = 839 kN > 300 kN
Beam side of connection is adequate	Beam side of connection is adequate

Tables G.18 and G.19

Title *Example 4 – Fin plates – Beam to CHS column* *Sheet 3 of 5*

SUMMARY OF FULL DESIGN CHECKS FOR EXAMPLE 4

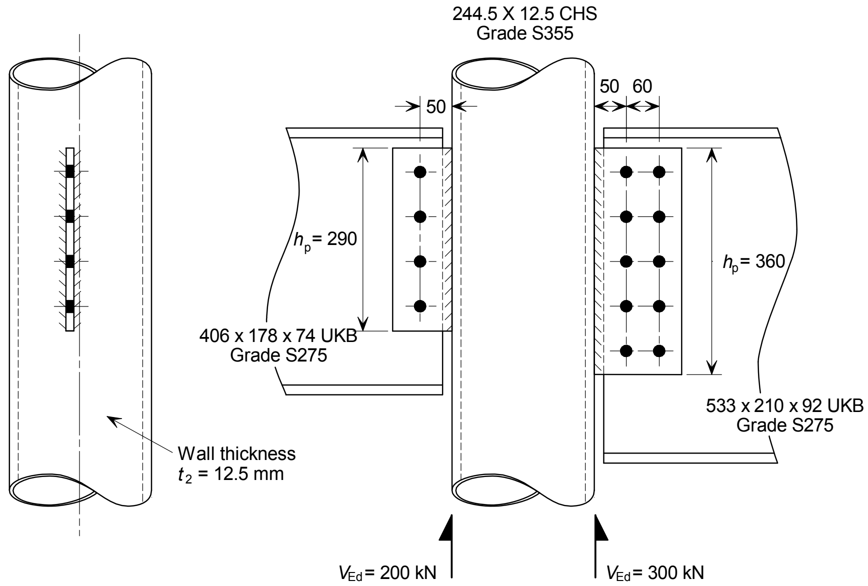
Note:

Checks 1 to 9, where applicable, are all as shown in Example 1 and are not repeated in this example, but the calculated resistances are summarised below. Check 4 resistances are higher because there are no notched beams in Example 4. Similarly, Checks 11 and 12 are as shown in Example 3.

Sheet No.	CHECK		406UKB (S275)		533UKB (S275)		CHS Column S355			
							406UKB Side		533UKB Side	
			Resistance	Design force	Resistance	Design force	Resistance	Design force	Resistance	Design force
See Example 1	***Check 1*** Recommended detailing practice		All recommendations adopted							
	Check 2 Supported beam Bolt group (kN)	Bolt group (kN)	288	200	577	300	Not applicable			
			332	200	586	300				
			291	200	565	300				
	Check 3 Supported beam Fin plate	Shear (kN)	347	200	450	300	Not applicable			
		Bending (kNm)	∞	200	∞	300				
		LTB of fin plate	771	200	742	300				
	Check 4 Supported beam Web in shear	Shear (kN)	664	200	909	300	Not applicable			
		Shear and bending	NA	NA	NA	NA				
		Long fin plate (kN)	NA	NA	NA	NA				
	Checks 5, 6, 7		Not applicable				Not applicable			
	Check 8 Supporting column Welds	*s* (mm)	Not applicable				5.7	5.0	5.7	5.0
	Check 9		Not applicable				Not applicable			
4	***Check 10*** Supporting column Shear and bearing	Shear (kN)	Not applicable				576	100	714	150
		Punching shear					t_p is adequate			
5	***Check 11*** Tying resistance Plate and bolts	Tension (kN)	661	200	839	300	Not applicable			
		Bolt shear (kN)	428	200	1070	300				
		Bearing (kN)	560	200	1230	300				
5	***Check 12*** Tying resistance Supported beam web	Tension (kN)	597	200	847	300	Not applicable			
		Bearing (kN)	432	200	1150	300				
	Checks 13, 14, 15		Not applicable				Not applicable			
5	***Check 16*** Tying resistance Supporting column wall	Tension (kN)	Not applicable				290	200	306	300

CONNECTION DESIGN FOLLOWING THE DESIGN PROCEDURES

Check 10: Supporting beam – Shear and bearing



Local shear

Basic requirement: $\dfrac{V_{Ed}}{2} \le F_{v,Rd}$

$F_{v,Rd} = \dfrac{A_v f_{y,2}}{\sqrt{3}\gamma_{M0}}$

406 × 178 × 74 UKB, S275

$A_v = h_p t_2 = 290 \times 12.5 = 3625 \text{ mm}^2$

$F_{v,Rd} = \dfrac{3625 \times 275}{\sqrt{3} \times 1.0} \times 10^{-3} = 576 \text{ kN}$

$\dfrac{V_{Ed}}{2} = 100 \text{ kN} < 576 \text{ kN}$ $\therefore$ **O.K.**

533 × 210 × 92 UKB, S275

$A_v = h_p t_2 = 360 \times 12.5 = 4500 \text{ mm}^2$

$F_{v,Rd} = \dfrac{4500 \times 275}{\sqrt{3} \times 1.0} \times 10^{-3} = 714 \text{ kN}$

$\dfrac{V_{Ed}}{2} = 150 \text{ kN} < 714 \text{ kN}$ $\therefore$ **O.K.**

Punching shear resistance

Basic requirement: (using the conservative method for both beam sides): $t_p \le t_2 \dfrac{f_{u,2}}{f_{y,p}}$

$t_2 \dfrac{f_{u,2}}{f_{y,p}} = 12.5 \times \dfrac{470}{275} = 21.4 \text{ mm}$

$t_p = 10 \text{ mm} < 21.4 \text{ mm}$ $\therefore$ **O.K.**

Title Example 4 – Fin plates – Beam to CHS column *Sheet 5 of 5*

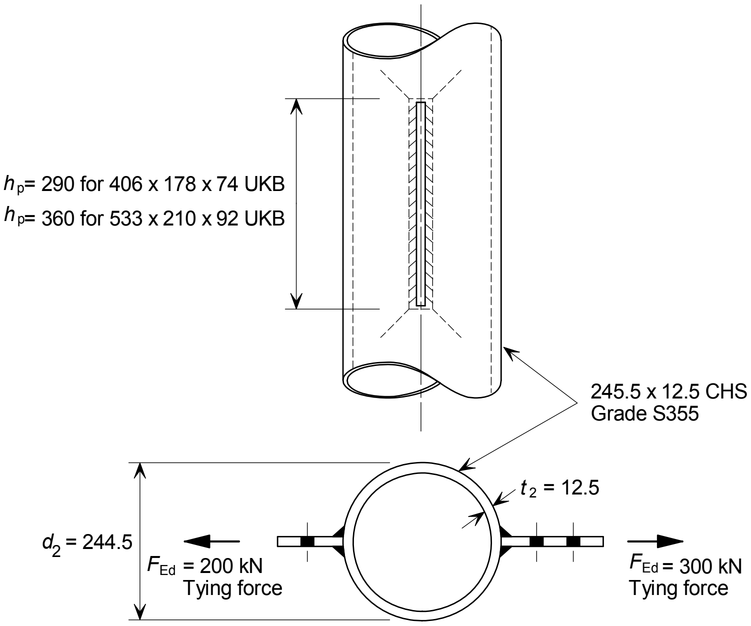
Check 11: Structural Integrity – Plate and bolts

See Example 3

Check 12: Structural Integrity – Supported beam web

See Example 3

Check 16: Structural Integrity –Supporting column Wall



Basic requirement: $F_{Ed} \leq F_{Rd,u}$

$$F_{Rd,u} = \frac{5 f_{u,2} t_2^{\ 2} \left(1 + 0.25\eta\right) \times 0.67}{\gamma_{Mu}}$$

406 × 178 × 74 UB, S275

$$\eta = \frac{h_p}{d_2} = \frac{290}{244.5} = 1.186$$

$$F_{Rd,u} = \frac{5 \times 470 \times 12.5^2 \times \left(1 + 0.25 \times 1.186\right) \times 0.67}{1.1} \times 10^{-3} = 290 \text{ kN}$$

$\therefore F_{Ed}$ = 200 kN $\leq$ 290 kN $\therefore$ **OK**

533 × 210 × 92 UB, S275

$$\eta = \frac{h_p}{d_2} = \frac{360}{244.5} = 1.472$$

$$F_{Rd,u} = \frac{5 \times 470 \times 12.5^2 \times \left(1 + 0.25 \times 1.472\right) \times 0.67}{1.1} \times 10^{-3} = 306 \text{ kN}$$

$\therefore F_{Ed}$ = 300 kN < 306 kN $\therefore$ **OK**

6 COLUMN SPLICES

6.1 INTRODUCTION

Column splices in multi-storey construction are usually provided every two or three storeys and are usually 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 6.1.

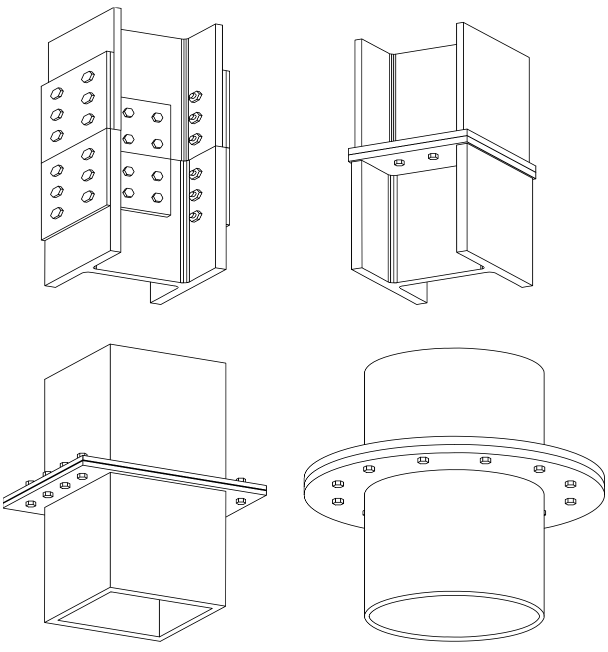


Figure 6.1 Splice connections

Bolted cover plate splices for I sections:

There are two categories for this type of connection:

- **Bearing type** (see Figure 6.3). 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 fewer bolts than the non-bearing splice, and is therefore the one most commonly used in practice.

- **Non-bearing type** (see Figure 6.4). In this case loads are transferred via the bolts and splice plates. Any direct bearing between the members is ignored, the connection sometimes being detailed with a physical gap between the two shafts.

Splices are generally provided just above floor levels (typically 600 mm above the top of the steel) hence the moment due to strut action is considered insignificant. The moments induced in splices placed at other positions should be taken into account.

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. If the column sections are offset (for example to maintain a constant external line) the moment due to the eccentricity should be accounted for in the joint design.

Bolted 'cap and base' or 'end plate' splices for tubular and rolled I sections

This type of splice, consisting of 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 may also be used for open sections.

The most simple form of the connection is as shown in Figure 6.2 and is satisfactory as long as the ends of each shaft are prepared in the same way as for a bearing type splice. The possibility of load reversal should be considered, in addition to stability during erection and tying requirements.

Although commonly used, it is difficult to demonstrate that cap and base splices meet the requirements of BS EN 1993-1-8[1] clause 6.2.7.1(13) and (14). If these types of splices are used, common practice is to ensure that the plates are thick, and that bolts are located close to the flanges to increase the stiffness of the connection. Extended plates, with bolts outside the profile of the section may be used. If cap and base plate splices are located away from a point of restraint, special consideration should be given to ensuring adequate stiffness so that the member design is not invalidated.

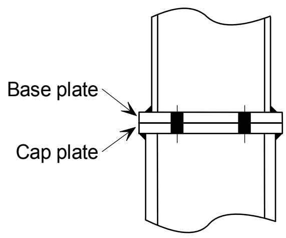


Figure 6.2 'Cap and base' or 'end plate' splice

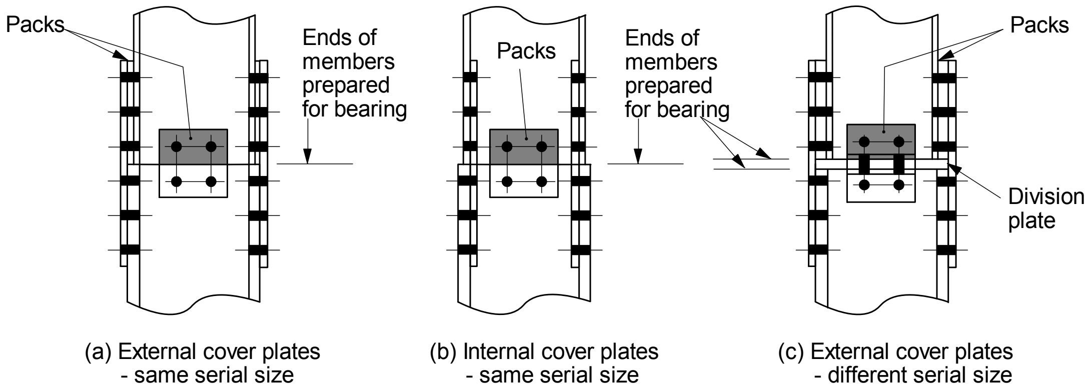


Figure 6.3 Bearing column splices for rolled I sections

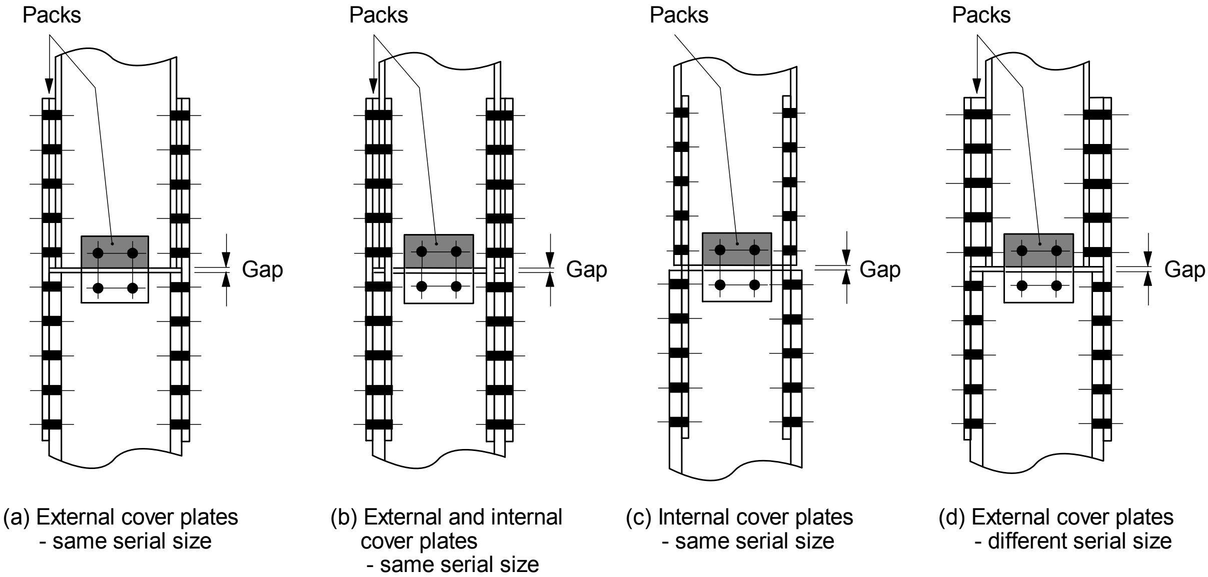


Figure 6.4 Non-bearing column splices for rolled I sections

6.2 PRACTICAL CONSIDERATIONS

Rolled I section column splices

The two types of splices used for universal column sections are shown in Figure 6.3 and Figure 6.4.

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.

Direct bearing does not necessitate the machining or end milling of the columns and full contact over the whole column area is not essential. The allowable tolerances between bearing surfaces are given in BS EN 1090-2[32] and the National Structural Steelwork Specification[10].

Bearing type splices are preferred in most cases for reasons of economy. 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 may be detailed between the columns, and the splice components should be designed accordingly.

In splices joining members of different serial size, multiple packs are necessary to take up the dimensional differences. In order to limit the packing to reasonable proportions, no more than one jump in the column serial size should be made at each splice. Rules on packing follow in the design checks.

These restrictions on packing do not apply when preloaded 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 6.3 and Figure 6.4. By using both these options, it is possible to detail a splice that is no wider than the column section itself.

Fasteners

Generally, the design resistances of fasteners should be based on the rules given in BS EN 1993-1-8. The fastener spacing and edge distances should comply with the requirements of Section 3.5 of that Standard.

For the majority of cases, property class 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.

Non-preloaded 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 reversal. In cases where significant net tension may be present, either preloaded bolts can be used, or alternatively the connection could be detailed with “cap and base” plates as shown in Figure 6.2. As a guide, net tension is considered significant when it exceeds 10% of the yield strength (f_y) of the upper column.

If preloaded bolts are used, it is recommended that the connection be designed as “non slip at serviceability limit state”, i.e. category B in accordance with Table 3.2 of BS EN 1993-1-8.

Holes

Holes in fittings may be punched full size, using semi-automatic equipment, within the limits laid down in BS EN 1090-2[32] and in accordance with the National Structural Steelwork Specification[10]. Holes through thicker fittings, column flanges and column webs may be punched or drilled.

Splice fittings

Splice fittings are usually fabricated using S275 material from standard flats or 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 flat enough for transfer of loads in bearing without the need for machining or flattening.

6.3 RECOMMENDED GEOMETRY

The main aims when detailing column splices are as follows:

- To provide a connection that is capable of carrying the design forces;
- 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[33].

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.

6.4 DESIGN

Full design procedures for column cover plate splices in rolled sections are given for bearing splices in Section 6.5 and for non-bearing splices in Section 6.6.

The design procedures for SHS, RHS and CHS bolted “cap and base” splices, when subject to tensile forces, are described in Section 6.7 and Section 6.8.

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.

Clause 6.2.7.1(14) requires that the splice material (the plates and bolts) must transmit at least 25% of the maximum compressive force in the column.

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. It is possible that the minimum requirements stated in BS EN 1993-1-8, 6.2.7.1(13) will dominate the design of a non-bearing splice, if the applied moments are modest.

Shear force

Any horizontal shear is normally resisted by friction across the contact bearing surfaces and/or by the web cover plates. For steel with no surface treatment and mill scale, the coefficient of friction may be conservatively taken as 0.2. Wind forces on the external elevations of buildings are normally taken directly into the floor slabs. It is rare for column splices in simple construction be expected to transmit significant shears.

Structural integrity

As noted in Section 1.2 the Building Regulations require that multi-storey buildings must be designed so that accidental damage does not lead to disproportionate collapse. Requirements regarding structural integrity are given in BS EN 1991-1-7.

Vertical tying requirements are given in BS EN 1991-1-7, A.6(2), stating that the accidental force to be resisted is the largest design vertical permanent and variable load reaction applied to the column from any one storey. It is recommended that the floors considered are limited to those down to the next lower splice. Use of a partial factor $\gamma_{Mu} = 1.1$ is recommended when calculating the resistance of the splice for vertical tying. Tying resistances for splices are not given in the yellow pages of this publication as the difference between resistance at ULS and in the accidental state is modest. The tying resistance of a splice can be conservatively taken as the twice the tension resistance of one flange connection.

Worked examples

Five worked examples are provided in section 6.9; three examples illustrate the design checks for rolled section cover plate splices and the other two examples illustrate the design checks for hollow section splices in tension.

Column splice tables

A fully detailed set of bearing type splices for common combinations of UKC columns is included in the yellow pages (Tables G.23 and G.24). Rationalised bolt spacing and fittings sizes have been adopted and these may be used for most splices. Tension resistances 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 bolts, see Table G.22 note 7.

Non-bearing splices are infrequently used for universal columns, so tables are not provided for this type of connection. Tables G.27, G.28 and G.29 are provided for bolted splices in CHS, SHS and RHS members in tension.

6.5 DESIGN PROCEDURES FOR COVER PLATE SPLICES FOR I SECTION COLUMNS – BEARING TYPE

Recommended design model

When the splice is 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 equal to half the difference in the section depth of the two sizes) or alternatively they may be connected to the inner flanges.

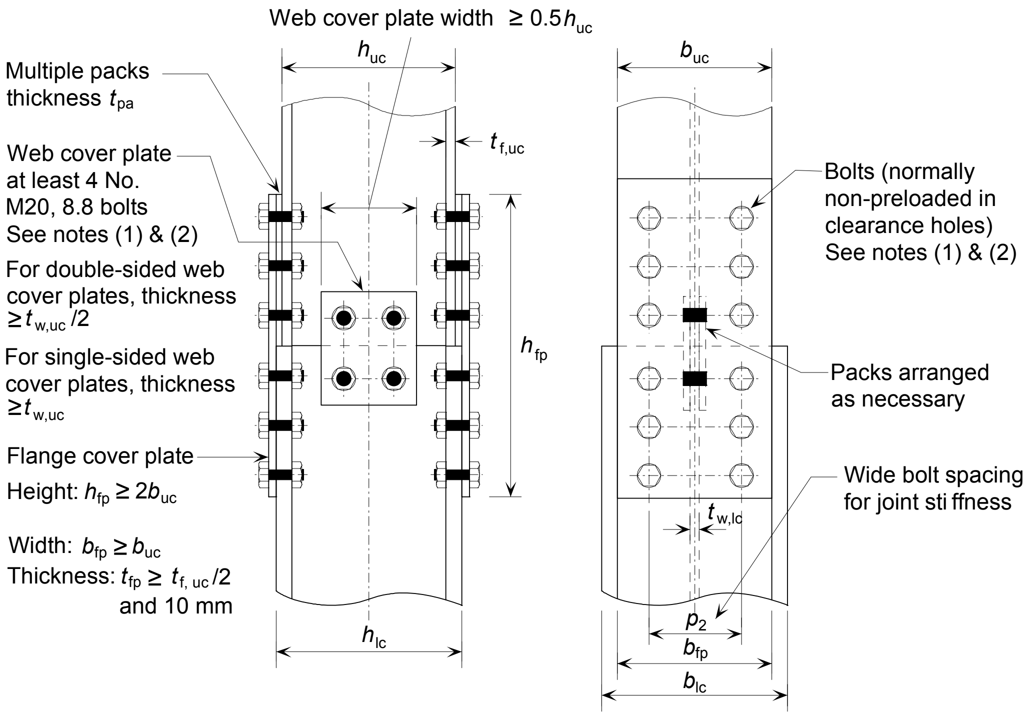
When the splice is being made between columns which are of different serial sizes, the transfer of axial forces from the upper shaft to the lower shaft is made through a horizontal division plate provided between the shafts. The thickness of the division plate is chosen so that the load from the upper section can be transferred to the lower section assuming a spread of load through the division plate at 45°. Flange cover plates then connect to the external faces of the column shafts.

In normal circumstances, any horizontal shear force is resisted by friction across the contact bearing surfaces and/or by the web cover plate, but no specific checks are presented here.

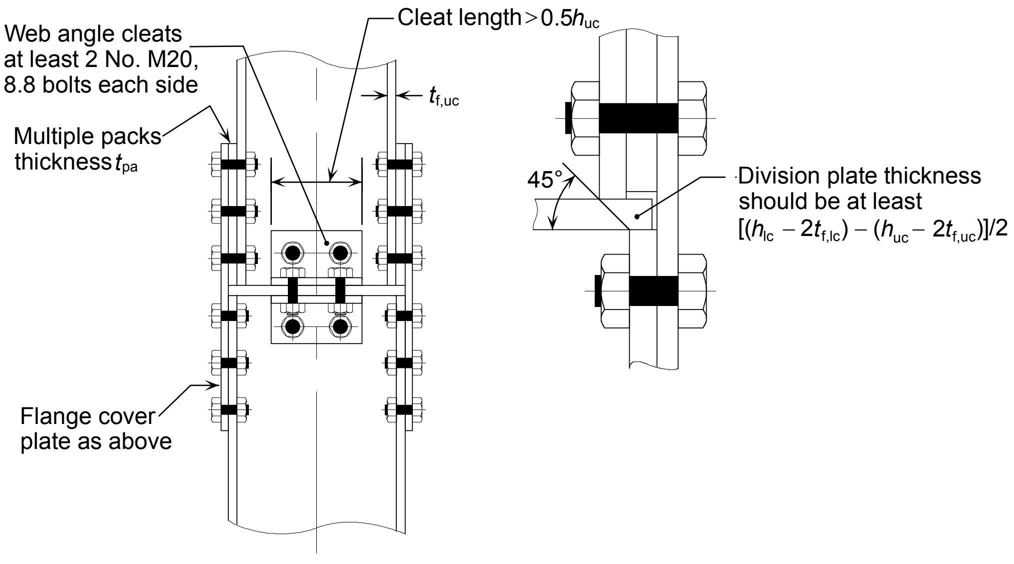
Design procedures are summarised below. Check 1 gives the recommended detailing rules for the splice, Checks 2 to 4 are strength checks required when tension is present. Check 5 covers the minimum resistance requirements specified in the Standard. Check 6 is required for structural integrity.

Check 1	Recommended detailing practice		
Check 2	Flange cover plates	–	Presence of net tension
Check 3	Flange cover plates	–	Plate resistance
Check 4	Flange cover plates	–	Bolt group
Check 5	Minimum resistance	–	Cover plates and bolt group
Check 6	Tying resistance	–	Cover plates and bolt group

CHECK 1	Recommended detailing practice I section columns – Bearing splice External flange cover plates



Butting surfaces of sections in direct bearing



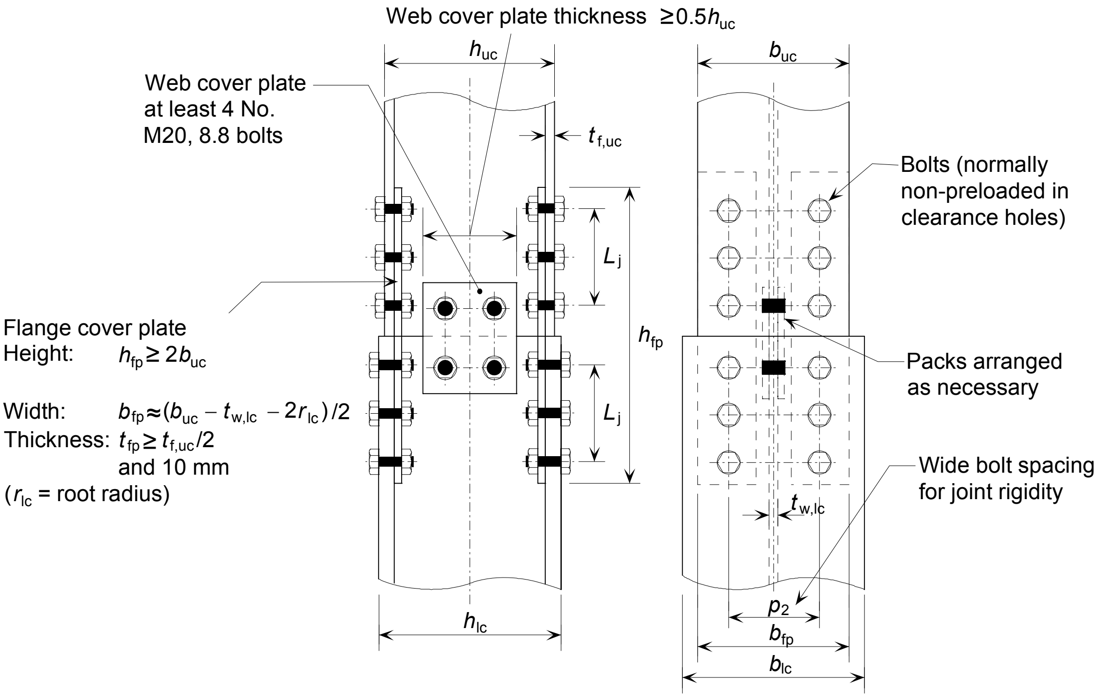
Direct bearing onto a division plate

Notes:

(1) Bolt spacing and edge distances should comply with the recommendations of BS EN 1993-1-8, Table 3.3.

(2) If there is significant net tension (see Check 3) then the notes from Check 1 for non-bearing splices should be followed.

CHECK 1 (continued)	Recommended detailing practice I section columns – Bearing splice Internal flange cover plates



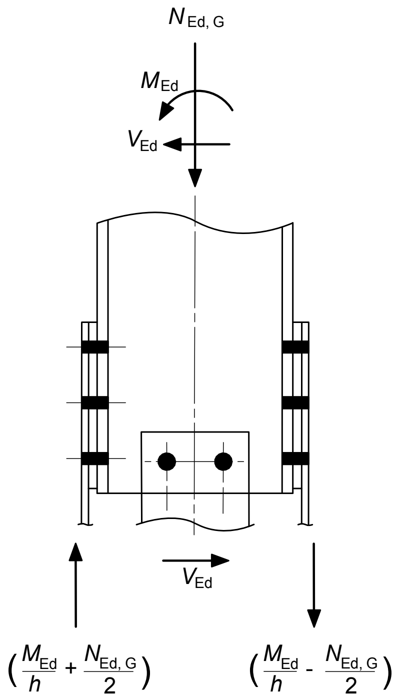
Internal flange cover plates

Notes:

(1) Bolt spacing and edge distances should comply with the recommendations of BS EN 1993-1-8, Table 3.3.

(2) If there is significant net tension (see Check 3) then the notes from Check 1 for non-bearing splices should be followed.

CHECK 2	Flange cover plates – Presence of net tension



Basic requirement:

If $M_{Ed} \leq \dfrac{N_{Ed,G}h}{2}$

net tension does not occur; the splice need only be detailed to transmit axial compression by direct bearing.

If $M_{Ed} > \dfrac{N_{Ed,G}h}{2}$

net tension does occur; Checks 3 and 4 should be used to check the flange cover plates and their fasteners for the tensile force, N_{Ed}, where:

$$N_{Ed} = \frac{M_{Ed}}{h} - \frac{N_{Ed,G}}{2}$$

Preloaded bolts should be used when net tension induces stress in the upper column flange > 10% of the design strength of that column.

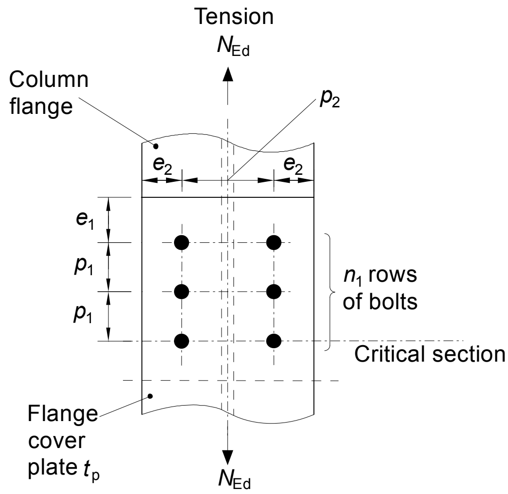
where:

M_{Ed} is the column design moment (due to factored permanent and variable loads) at the floor level immediately below the splice.

$N_{Ed,G}$ is the axial compressive force due to factored permanent load only.

h is, 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 plates – Plate resistance



This check is required when net tension occurs (see Check 2)

Basic requirement:

$N_{Ed} \leq N_{t,Rd}$

$N_{t,Rd} = \min(N_{pl,Rd};\ N_{u,Rd};\ N_{bt,Rd})$

$N_{pl,Rd}$ is the tension resistance of the gross area

$$N_{pl,Rd} = \frac{A_{fp} f_{y,fp}}{\gamma_{M0}}$$

$N_{u,Rd}$ is the tension resistance of the net area

$$N_{u,Rd} = \frac{0.9 A_{fp,net} f_{u,fp}}{\gamma_{M2}}$$

$N_{bt,Rd}$ is the block tearing resistance

For a concentrically loaded bolt group:

$$N_{bt,Rd} = \frac{f_{u,fp} A_{fp,nt}}{\gamma_{M2}} + \frac{f_{y,fp} A_{fp,nv}}{\sqrt{3}\gamma_{M0}}$$

Check for significant net tension:

If $\dfrac{N_{Ed}}{t_{f,uc} b_{f,uc} f_{y,uc}} > 0.1$ then preloaded bolts should be used.

where:

A_{fp} is the gross area of the flange cover plate(s) attached to one flange

$A_{fp,net}$ is the net area of the flange cover plate(s) attached to one flange

$t_{f,uc}$ is the flange thickness of the upper column

$b_{f,uc}$ is the flange width of the upper column

$A_{fp,nv}$ is the net area subjected to shear

$A_{fp,nv} = 2t_p\ (\ e_1+(n_1\text{-}1)p_1 - (n_1\text{-}0.5)d_0\ )$

$A_{fp,nt}$ is the net area subjected to tension

If $p_2 \leq 2e_2$ $\quad A_{fp,nt} = t_p(\ p_2 - d_0\ )$

If $p_2 > 2e_2$ $\quad A_{fp,nt} = t_p(\ 2e_2 - d_0\ )$

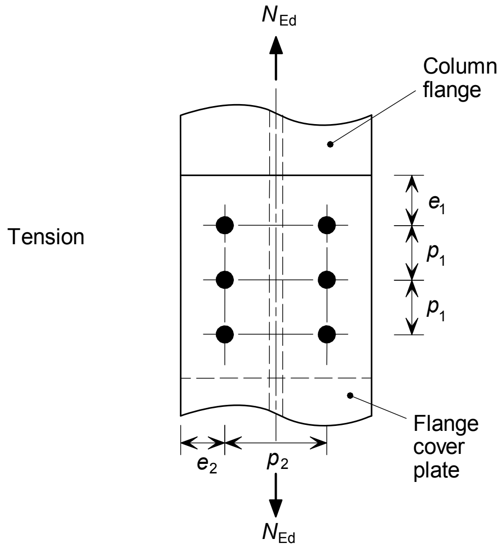
γ_{M2} is the partial factor for the ultimate tension resistance of cross sections (γ_{M2} = 1.1 as given in the National Annex to BS EN 1993-1-1)

Notes:

(1) It is sufficiently accurate to base the calculation for significant net tension on the gross area of the flange.

(2) When the tension is due to structural integrity requirements it is not necessary to use preloaded bolts.

CHECK 4	Flange cover plates – Bolt group



This check is required when net tension occurs (see Check 2)

For non-preloaded bolts:

Basic requirement:

$N_{Ed} \leq F_{Rd,fp}$

$F_{Rd,fp}$ is the design resistance of the bolt group

$F_{Rd,fp} = nF_{b,Rd}$ if $F_{b,Rd} \leq F_{v,Rd}$

$F_{Rd,fp} = nF_{v,Rd}$ if $F_{v,Rd} < F_{b,Rd}$

$F_{v,Rd}$ is the shear resistance of a single bolt

$$= \beta_p \times \frac{\alpha_v f_{ub} A}{\gamma_{M2}}$$

$F_{b,Rd}$ is the bearing resistance of a single bolt

$$= \frac{k_1 \alpha_b f_u d t_{fp}}{\gamma_{M2}}$$

If the length of the joint in each column $L_j = (n_1 - 1)\, p_1$ is greater than 15 d, the design shear resistance of all the fasteners should be reduced by a factor β_{LF} given by:

$$\beta_{LF} = 1 - \frac{L_j - 15d}{200d}$$

Note 1: If the thickness of the column flange is less than the thickness of the flange cover plates, or of a lesser grade, then the bearing resistance of the column flange should also be checked.

where:

n is the number of bolts connecting one flange to the cover plate

α_v = 0.6 for property class 8.8 bolts

A is the tensile stress area of the bolt, A_s

β_p = 1.0 if $t_{pa} < d/3$

$= \frac{9d}{8d + 3t_{pa}}$ if $t_{pa} \geq d/3$

d is the diameter of the bolt

t_{pa} is the total thickness of the packing

α_b $= \min\left(\frac{e_1}{3d_0};\ \frac{p_1}{3d_0} - \frac{1}{4};\ \frac{f_{ub}}{f_{u,fp}};\ 1.0\right)$

k_1 $= \min\left(2.8\frac{e_2}{d_0} - 1.7;\ 1.4\frac{p_2}{d_0} - 1.7;\ 2.5\right)$

d_0 is the diameter of the hole

γ_{M2} is the partial factor for bolt resistance (γ_{M2} = 1.25 as given in the National Annex)

CHECK 4 (continued)	Flange cover plates – Bolt group

For Preloaded bolts:

Basic requirement:

For connections designed to be non-slip under serviceability loads.

$N_{Ed,ser} \leq nF_{s,Rd,ser}$

$F_{s,Rd,ser}$ = design slip resistance

$= \dfrac{k_s n \mu}{\gamma_{M3,ser}} F_{p,C}$ (BS EN 1993-1-8, 3.9.1)

and

$N_{Ed} \leq F_{Rd,fp}$

(shear and bearing resistance – see check for non-preloaded bolts)

where:

n is the number of bolts connecting one flange to the cover plate

k_s = 1.0 for fasteners in standard clearance holes (BS EN 1993-1-8, Table 3.6)

n is the number of the friction surfaces

μ is the slip factor

$F_{p,C}$ is the preloading force

$= 0.7 f_{ub} A_s$

A_s is the tensile stress area of the bolt

$\gamma_{M3,ser}$ is the partial factor for slip resistance of bolts at serviceability limit state ($\gamma_{M3,ser}$ = 1.10 as given in the National Annex)

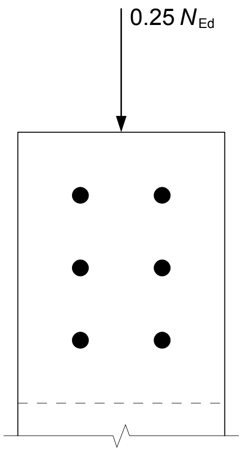
$N_{Ed,ser}$ is calculated at SLS

$F_{Rd,fp}$ is the design resistance of the bolt group (see check for non-preloaded bolts)

From BS EN 1090-2, Table 18, values of the slip factor, μ are:

Surface Treatment	**Slip factor**
Surfaces blasted with shot or grit with loose rust removed, not pitted	0.5
Surfaces blasted with shot or grit:	
a) Spray-metallized with an aluminium or zinc based product	0.4
b) With alkali-zinc silicate paint with a thickness of 50 μm to 80 μm	0.4
Surfaces cleaned by wire brushing or flame cleaning, with loose rust removed	0.3
Surfaces as rolled	0.2

CHECK 5	Minimum resistance Cover plates and bolt group



Basic requirement:

$0.25N_{Ed} \leq N_{Rd}$

Cover plates:

$0.25\ N_{Ed} \leq N_{Rd}$

$$N_{Rd} = \frac{2A_{fp}f_{y,fp}}{\gamma_{M0}}$$

Bolt group:

$0.25\ N_{Ed} \leq 2F_{Rd,fp}$

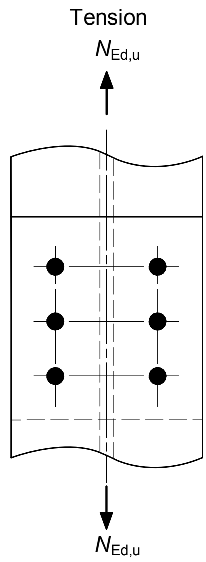
where:

A_{fp} is the area of the flange cover plate

$F_{Rd,fp}$ is the resistance of the bolt group in one flange as calculated in Check 4

Note: Conservatively, it is assumed that the compression is transferred by the flange cover plates and bolts.

CHECK 6	Tying resistance



If it is necessary to comply with structural integrity requirements, then Checks 3 and 4 should be carried out with:

$$N_{Ed} = \frac{N_{Ed,u}}{2}$$

based on the conservative assumption that the tying force is resisted by the two flange cover plates.

$N_{Ed,u}$ is the tensile force from BS EN 1991-1-7, clause A.6.

Although the resistance of the connection can be calculated using $\gamma_{Mu} = 1.1$, it is conservative to adopt the resistances calculated according to Checks 3 and 4.

6.6 DESIGN PROCEDURES FOR COVER PLATE SPLICES FOR I COLUMNS – NON-BEARING TYPE

In a non-bearing splice, all forces and moments are carried by the splice cover plates.

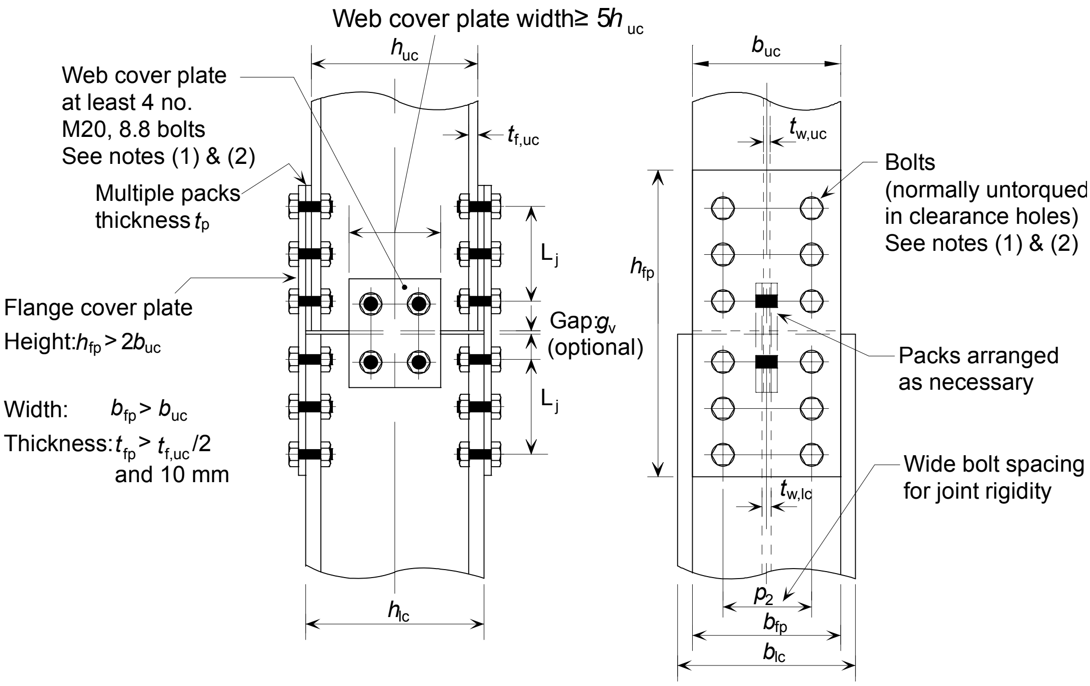
The flange cover plates can be arranged to connect either to the external faces of the column (when pairs of packing plates will be required with a thickness equating to the difference in the depth dimension of the two sizes) or alternatively to the inner flanges by using split cover plates.

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.

Design procedures are summarised in the following Section. Check 1 gives the recommended detailing rules for the splice, Checks 2 to 5 are resistance checks and Check 8 is required for structural integrity design. Checks 6 and 7 cover the minimum resistance requirements specified in BS EN 1993-1-8, 6.2.7.1(13). Note that these minimum requirements may dominate the design.

Check	1	Recommended detailing practice		
Check	2	Flange cover plates	–	Plate resistance
Check	3	Flange cover plates	–	Bolt group
Check	4	Web cover plates	–	Plate resistance
Check	5	Web cover plates	–	Bolt group
Check	6	Minimum resistance – major axis	–	Cover plates and bolt group
Check	7	Minimum resistance – minor axis	–	Cover plates and bolt group
Check	8	Structural integrity	–	Cover plates and bolt group

CHECK 1	Recommended detailing practice I section columns – Non-bearing splice External flange cover plates



Axial compression developed in external flange cover plates

Notes:

(1) Preloaded 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 cover plates in non-bearing splices may be much larger, especially when the minimum requirements of Checks 6 and 7 are considered.

(3) Bolt diameters should be at least 75% of packing thickness t_p.[32]

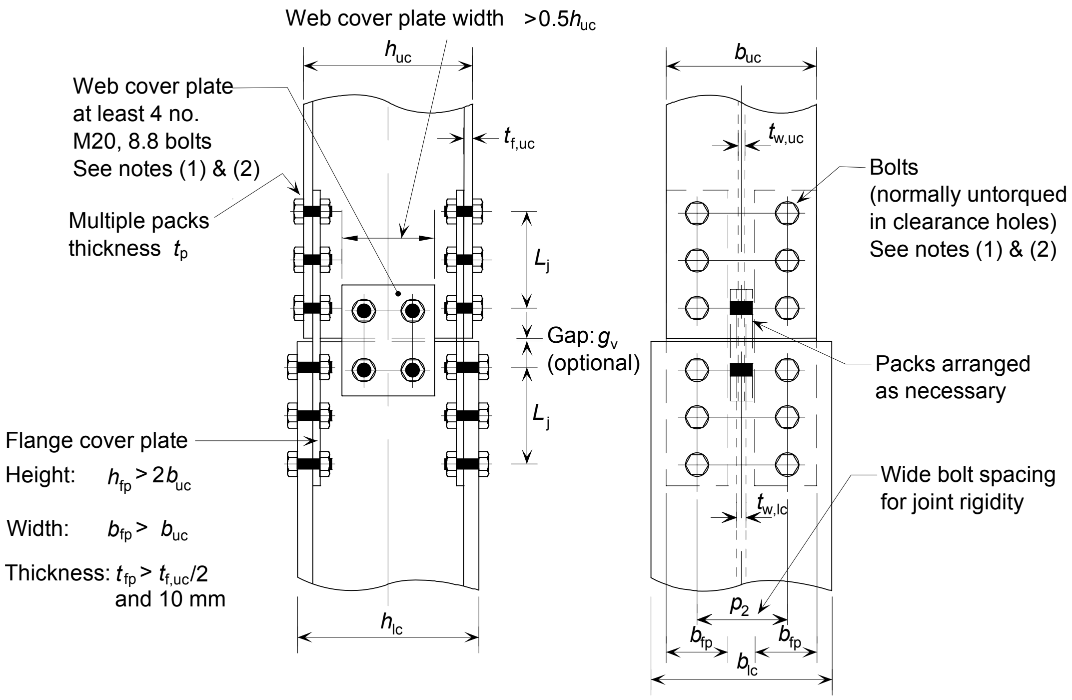
(4) The number of plies in multiple packs should not exceed four.[32]

(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 size should be similar to those shown and the combined thickness of the external and internal cover plates should be at least $t_{f,uc}$ / 2. All cover plates should be at least 10 mm thick.

(7) Bolt spacing and edge distances should comply with the recommendation of BS EN 1993-1-8, Table 3.3.

CHECK 1 (continued)	**Recommended detailing practice I section columns– Non-bearing splice Internal flange cover plates**



Axial compression developed in internal flange cover plates

Notes:

(1) Preloaded bolts are required if slip is unacceptable.

(2) Bolt diameters should be at least 75% of packing thickness t_p.[32]

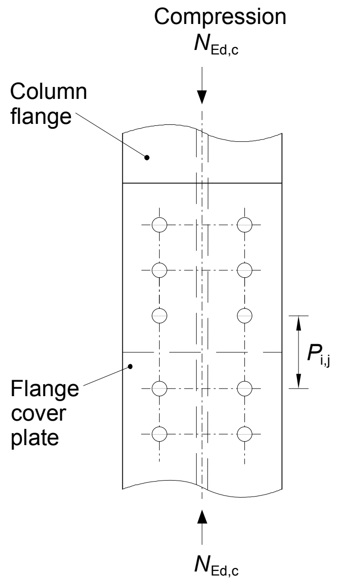
(3) The number of plies in multiple packs should not exceed four.[32]

(4) There should not be more than one jump in serial size of column at the splice.

(5) If external and internal flange covers are to be provided the size should be similar to those shown and the combined thickness of the external and internal cover plates should be at least $t_{f,uc}$ / 2. All cover plates should be at least 10 mm thick.

(6) Bolt spacing and edge distances should comply with the recommendation of BS EN 1993-1-8, Table 3.3.

CHECK 2	Flange cover plates – Plate resistance



Compression

Basic requirement:

$N_{Ed,c} \leq N_{c,Rd}$

$$N_{Ed,c} = \frac{M_{Ed}}{h} + N_{Ed}\left(\frac{A_{f,1}}{A_1}\right)$$

N_{Ed} is the axial compressive force due to factored permanent and variable loads

$$N_{c,Rd} = \chi \frac{A_{fp} f_{y,fp}}{\gamma_{M1}}$$

where:

$A_{f,1}$ is the area of one flange of the smaller column $= h_1\, t_{f,1}$

A_1 is the total area of the smaller column

h is, 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)

χ is the reduction factor for buckling. See BS EN 1993-1-1, clause 6.3. Use of a buckling length of $0.6 \times p_{1,J}$, width $b_{fp}/2$ and buckling curve c are recommended.

χ may be taken as 1.0 if $\frac{p_{1,j}}{t_{fp}} \leq 9\varepsilon$ (no buckling)

$p_{1,j}$ is the pitch between the bolt groups in the upper and lower columns

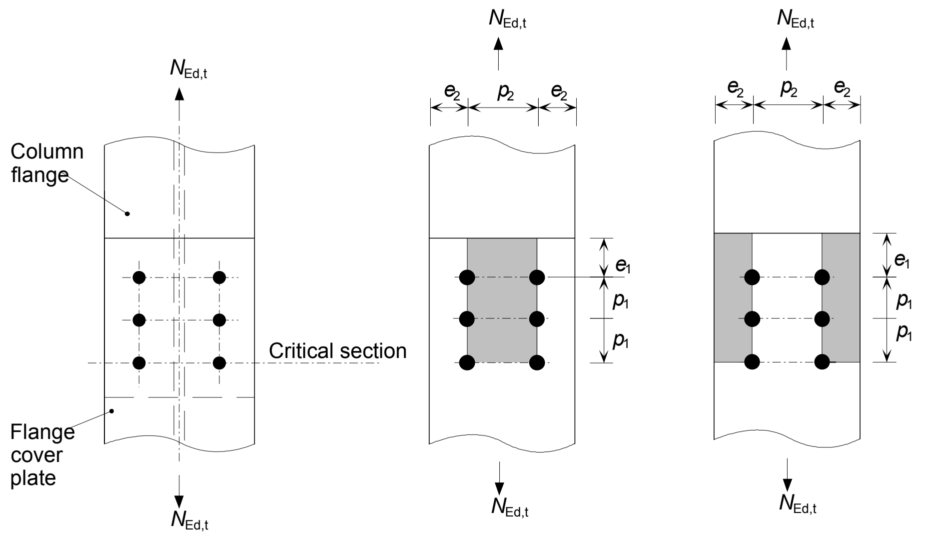
t_{fp} is the thickness of the flange cover plate

$\varepsilon = \sqrt{\frac{235}{f_{y,fp}}}$

A_{fp} is the gross area of flange cover plate(s) attached to one flange

Note that this check assumes $p_{1,j} > p_1$, where p_1 is the vertical pitch between bolt rows.

CHECK 2 (continued)	**Flange cover plates – Plate resistance**



Tension

Basic requirement:

$N_{Ed,t} \leq \quad N_{t,Rd}$

$N_{Ed,t} = \dfrac{M_{Ed}}{h} - N_{Ed,G}\left(\dfrac{A_{f,1}}{A_1}\right)$

$N_{Ed,G}$ is the axial compressive force due to factored permanent loads only

$N_{t,Rd} = \min(N_{pl,Rd};\quad N_{u,Rd};\quad N_{bt,Rd})$

$N_{pl,Rd} = \dfrac{A_{fp} f_{y,fp}}{\gamma_{M0}}$

$N_{u,Rd} = \dfrac{0.9 A_{fp,net} f_{u,fp}}{\gamma_{M2}}$

For a concentrically loaded bolt group:

$N_{bt,Rd} = = \dfrac{f_{u,fp} A_{fp,nt}}{\gamma_{M2}} + \dfrac{f_{y,fp} A_{fp,nv}}{\sqrt{3}\gamma_{M0}}$

where:

$A_{fp,net}$ is the net area of flange cover plate(s) attached to one flange

$A_{fp,nv}$ is the net area subjected to shear

$A_{fp,nv} = t_p\ (\ e_1 + (n_1 - 1)p_1 - (n_1 - 0.5)d_0\ )$

$A_{fp,nt}$ is the net area subjected to tension

If $p_2 < 2e_2$ then:

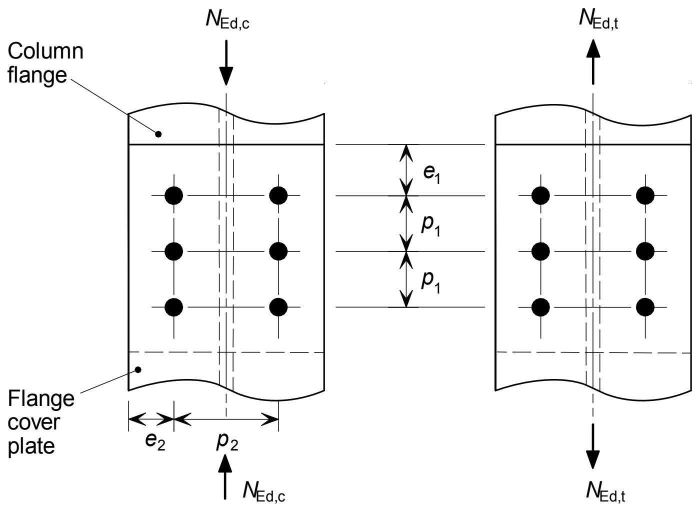
$A_{fp,nt} = t_p(\ p_2 - d_0\ )$

If $p_2 > 2e_2$ then:

$A_{fp,nt} = t_p\ (\ 2e_2 - d_0\ )$

γ_{M2} is the partial factor for tension resistance of net sections (γ_{M2} = 1.1 as given in the National Annex to BS EN 1993-1-1)

CHECK 3	Flange cover plates – Bolt group



For non-preloaded bolts

Basic requirement:

$N_{Ed} \leq F_{Rd,fp}$

$N_{Ed} = \max(N_{Ed,c};\ N_{Ed,t})$

$F_{Rd,fp}$ is the design resistance of flange cover plate bolt group

$F_{Rd,fp} = n_{fp}F_{b,Rd}$ if $F_{b,Rd} \leq F_{v,Rd}$

$F_{Rd,fp} = n_{fp}F_{v,Rd}$ if $F_{v,Rd} \leq F_{b,Rd}$

$F_{v,Rd}$ is the shear resistance of a single bolt

$$= \beta_p \frac{\alpha_v f_{ub} A}{\gamma_{M2}}$$

$F_{b,Rd}$ is the bearing resistance of a single bolt

$$= \frac{k_1 \alpha_b f_u d t_{fp}}{\gamma_{M2}}$$

where:

α_v = 0.6 for 8.8 bolts

A is the tensile stress area of the bolt, A_s

β_p = 1.0 if $t_{pa} < d/3$

$$= \frac{9d}{8d + 3t_{pa}} \quad \text{if } t_{pa} > d/3$$

d is the diameter of the bolt

t_{pa} is the total thickness of the packing

$$\alpha_b = \min\left(\frac{e_1}{3d_0};\ \frac{p_1}{3d_0} - \frac{1}{4};\ \frac{f_{ub}}{f_{u,p}};\ 1.0\right)$$

$$k_1 = \min\left(2.8\frac{e_2}{d_0} - 1.7;\ 1.4\frac{p_2}{d_0} - 1.7;\ 2.5\right)$$

d_0 is the diameter of the hole

t_{fp} is the thickness of the flange cover plate

n_{fp} is the numberof bolts connecting one flange to a flange cover plate

γ_{M2} is the partial factor for bolt resistance (γ_{M2} = 1.25 as given in the National Annex)

Note:

If the thickness of the column flange is less than the thickness of the flange cover plates, or of lower grade, then the bearing resistance of the column flange should also be checked.

CHECK 3 (continued)	Flange cover plates – Bolt group

For Preloaded bolts:

Basic requirement:

For connection designed to be slip resistant under serviceability loads (Category B connection).

$N_{Ed,ser} \leq F_{s,Rd,ser}$

$N_{Ed,ser} = \max\left(N_{Ed,c,ser};\ N_{Ed,t,ser} \right)$

$F_{s,Rd,ser}$ = design slip resistance

$$= \frac{k_s n \mu}{\gamma_{M3}} F_{p,C} \quad \text{(BS EN 1993-1-8, 3.9.1)}$$

and

$N_{Ed} \leq F_{Rd,fp}$

(shear and bearing resistance – see check for non-preloaded bolts)

where:

k_s = 1.0 for fasteners in standard clearance holes (BS EN 1993-1-8, Table 3.6)

n is the number of friction surfaces

μ is the slip factor (see Check 3 for bearing splices)

$F_{p,C}$ is the preloading force

$= 0.7 f_{ub} A_s$

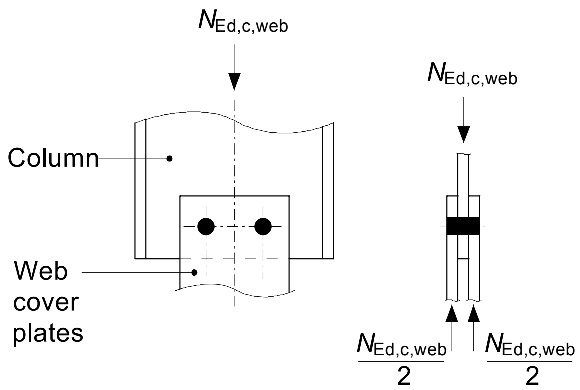
A_s is the tensile stress area of the bolt

$\gamma_{M3,ser}$ is the partial factor for slip resistance of bolts at serviceability limit state ($\gamma_{M3,ser}$ = 1.10 as given in the National Annex)

$N_{Ed,c,ser}$ and $N_{Ed,t,ser}$ are calculated at SLS

$F_{Rd,fp}$ is the design resistance of the bolt group (see check for non-preloaded bolts)

CHECK 4	Web cover plates Plate resistance



Web cover plate resistance

Basic requirement:

$$\frac{N_{Ed,c,web}}{2} \leq N_{c,Rd,wp}$$

$$N_{c,Rd,wp} = \frac{A_{wp} f_{y,wp}}{\gamma_{M0}}$$

Conservatively $N_{Ed,c,web} = \frac{N_{Ed} A_w}{A}$

where

N_{Ed} is the axial compressive force due to factored permanent and imposed loads

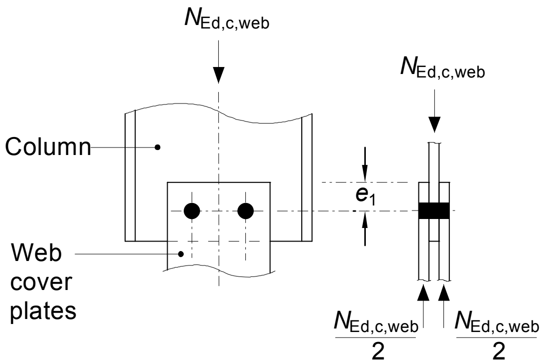
A is the total area of the smaller column

A_w is the area of web of the smaller column
$= (A - 2A_f)$

A_f is the area of one flange of the smaller column

A_{wp} is the gross area of one web cover plate

CHECK 5	Web cover plates – Bolt group



For non-preloaded bolts:

Basic requirement:

$N_{Ed,c,web} \quad \leq \quad F_{Rd,wp}$

$N_{Rd,wp}$ is the design resistance of bolt group in the web

$F_{Rd,wp} = n_{wp}F_{b,Rd}$ if $F_{b,Rd} \leq F_{v,Rd}$

$F_{Rd,wp} = n_{wp}F_{v,Rd}$ if $F_{v,Rd} \leq F_{b,Rd}$

$F_{v,Rd}$ = shear resistance of a single bolt

$= \beta_p \dfrac{2\alpha_v f_{ub} A}{\gamma_{M2}}$ (double shear)

$F_{b,Rd}$ = bearing resistance of a single bolt

$= \dfrac{2k_1 \alpha_b f_u d t_{wp}}{\gamma_{M2}}$

where:

α_v = 0.6 for 8.8 bolts

A is the tensile stress area of the bolt, A_s

β_p = 1.0 if $t_{pa} < d/3$

$\beta_p = \dfrac{9d}{8d + 3t_{pa}}$ if $t_{pa} > d/3$

d is the diameter of the bolt

t_{pa} is the total thickness of the packing

$$\alpha_b = \min\left(\frac{e_1}{3d_0};\ \frac{p_1}{3d_0} - \frac{1}{4};\ \frac{f_{ub}}{f_{u,p}};\ 1.0\right)$$

$$k_1 = \min\left(2.8\frac{e_2}{d_0} - 1.7;\ 1.4\frac{p_2}{d_0} - 1.7;\ 2.5\right)$$

d_0 is the hole diameter

t_{wp} is the thickness of one web cover plate

n_{wp} is the number of bolts connecting one web to the cover plates

γ_{M2} is the partial factor for resistance of bolts (γ_{M2} = 1.25 as given in the National Annex)

Note: If the thickness of the column web is less than the combined thickness of the web cover plates, or of lower grade, then the bearing resistance of the column web should also be checked.

CHECK 5 **(continued)**	**Web cover plates – Bolt group**

For Preloaded bolts:

Basic requirement:

For connection designed to be slip resistant under serviceability loads (Category B connection).

$N_{\mathrm{Ed,c,web,ser}} \leq F_{\mathrm{s,Rd,ser}}$

$F_{\mathrm{s,Rd,ser}}$ = design slip resistance

$$= \frac{k_s n \mu}{\gamma_{\mathrm{M3,ser}}} F_{\mathrm{p,C}} \quad \text{(BS EN 1993-1-8, 3.9.1)}$$

and

$N_{\mathrm{Ed,c,web}} \leq F_{\mathrm{Rd,wp}}$

(shear and bearing resistance – see check for non-preloaded bolts)

where:

k_s = 1.0 for fasteners in standard clearance holes (BS EN 1993-1-8, Table 3.6)

n is the number of friction surfaces

μ is the slip factor (see Check 3 for bearing splices)

$F_{\mathrm{p,C}}$ is the preloading force

$= 0.7 f_{\mathrm{ub}} A_s$

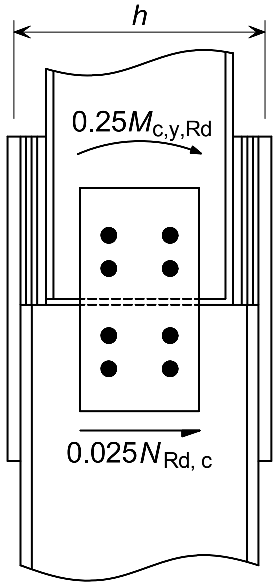
A_s is the tensile stress area of the bolt

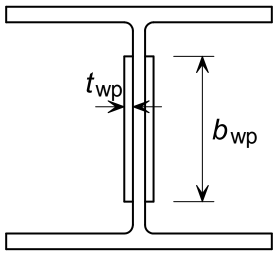
$\gamma_{\mathrm{M3,ser}}$ is the partial factor for slip resistance of bolts at serviceability limit state ($\gamma_{\mathrm{M3,ser}}$ = 1.10 as given in the National Annex)

$N_{\mathrm{Ed,c,web,ser}}$ is calculated at SLS

$F_{\mathrm{Rd,wp}}$ is the design resistance of the bolt group (see check for non-preloaded bolts)

CHECK 6	**Minimum resistance checks – major axis**





According to BS EN 1993-1-8 clause 6.2.7.1(13), the splice must have a resistance greater than 25% of the capacity of the smaller section, in bending, in both axes. The splice must also have a minimum shear resistance in both axes, equal to 2.5% of the compression capacity of the weaker column section. It is recommended that because the checks are designed to ensure an adequately stiff connection, it is not necessary to verify connection components under combinations of moments and forces.

(i) Major axis bending – cover plates

Basic requirement:

0.25 $M_{c,y,Rd} \le h\ N_{C,Rd}$ and

0.25 $M_{c,y,Rd} \le h\ N_{T,Rd}$

$N_{C,Rd} = A_{fp}\ f_{y,fp}/\gamma_{M0}$

$N_{T,Rd}$ is the lesser of:

$N_{pl,Rd} = Af_y/\gamma_{M0}$ and

$N_{u,Rd} = 0.9A_{net}f_u/\gamma_{M2}$

(ii) Major axis bending – bolt group

Basic requirement:

0.25 $M_{c,y,Rd} \le h\ F_{Rd,fp}$

(iii) Major axis shear resistance – cover plates

Basic requirement:

0.025 $N_{Rd,c} \le V_{pl,Rd,wp}$

(iv) Major axis shear resistance – bolt group

Basic requirement:

The bolts in the web should be checked for an applied shear of 0.025 $N_{Rd,c}$. The shear also applies a moment to the bolt group, equal to the applied shear at a lever arm taken from the end of the column to the centroid of the bolt group.

where:

h is the distance between the centres of the cover plates

$F_{Rd,fp}$ is the resistance of the bolt group from Check 3

$$V_{pl,Rd,wp} = \frac{2 \times 0.9 t_{wp} b_{wp} f_{y,wp}}{\sqrt{3}\gamma_{M0}}$$

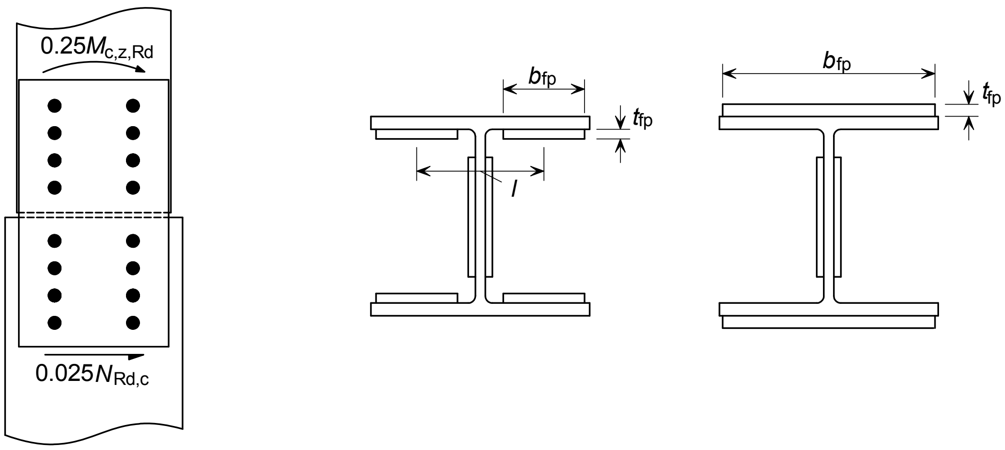
$N_{Rd,c}$ is the design resistance in compression of the weaker section

$M_{c,y,Rd}$ is the design resistance for bending of the weaker section in the major axis

γ_{M0} is the partial factor for resistance of cross sections (γ_{M0} = 1.0 as given in the National Annex to BS EN 1993-1-1)

γ_{M2} is the partial factor for resistance of net sections (γ_{M2} = 1.1 as given in the National Annex to BS EN 1993-1-1)

CHECK 7	Minimum resistance checks – minor axis



(i) Minor axis bending – cover plates

Basic requirement:

$0.25\ M_{c,z,Rd} \leq M_{c,Rd,fp}$

(ii) Minor axis bending – bolt group

Basic requirement:

The resistance of the bolt group in each flange must be verified under the applied moment (equal to 0.125 $M_{c,z,Rd}$ in each flange).

Both shear and bearing resistances must be checked. Bearing resistances may conveniently be checked in two perpendicular directions, as permitted by BS EN 1993-1-8, Table 3.4, note 3.

The shear resistance of the bolts must be reduced if packs are used, as required in Check 3.

(iii) Minor axis shear resistance – cover plates

Basic requirement:

$0.025\ N_{Rd,c} \leq V_{pl,Rd,fp}$

(iv) Minor axis shear resistance – bolt group

Basic requirement:

The resistance of the bolt group should be calculated for the moment due to the applied shear multiplied by the lever arm from the end of the column to the centroid of the bolt group.

where:

$M_{c,Rd,fp}$ is the bending resistance of the cover plates

For 2 external cover plates:

$$M_{c,Rd,fp} = \frac{2t_{fp}b_{fp}^{2}f_{y,fp}}{6\gamma_{M0}}$$

For internal cover plates:

$$M_{c,Rd,fp} = 8\left[\frac{t_{fp}b_{fp}^{3}}{12} + t_{fp}b_{fp}\left(\frac{l}{2}\right)^{2}\right]\frac{f_{y,fp}}{(l+b_{fp})\gamma_{M0}}$$

l is the distance between the centres of the plates

b_{fp} is the width of the individual plate

$V_{pl,Rd,fp}$ is the shear resistance of the flange cover plates

For external cover plates:

$$V_{pl,Rd,fp} = \frac{2\times 0.9t_{fp}b_{fp}f_{y,fp}}{\sqrt{3}\gamma_{M0}}$$

For internal cover plates:

$$V_{pl,Rd,fp} = \frac{4\times 0.9t_{fp}b_{fp}f_{y,fp}}{\sqrt{3}\gamma_{M0}}$$

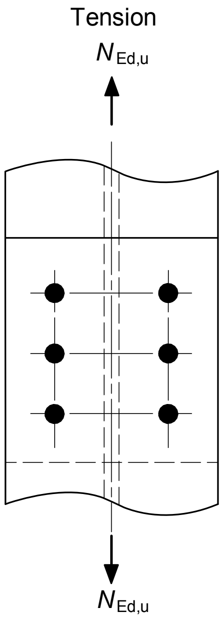
$F_{Rd,fp}$ is the resistance of the bolt group from Check 3

$N_{Rd,c}$ is the design resistance in compression of the weaker section

$M_{c,z,Rd}$ is the design resistance for bending of the weaker section in the minor axis

γ_{M0} is the partial factor for resistance of cross sections (γ_{M0} = 1.0 as given in the National Annex to BS EN 1993-1-1)

CHECK 8	Tying resistance



If it is necessary to comply with structural integrity requirements, then the bearing type splice checks 2 and 3 should be carried out with:

$$N_{Ed} = \frac{N_{Ed,u}}{2}$$

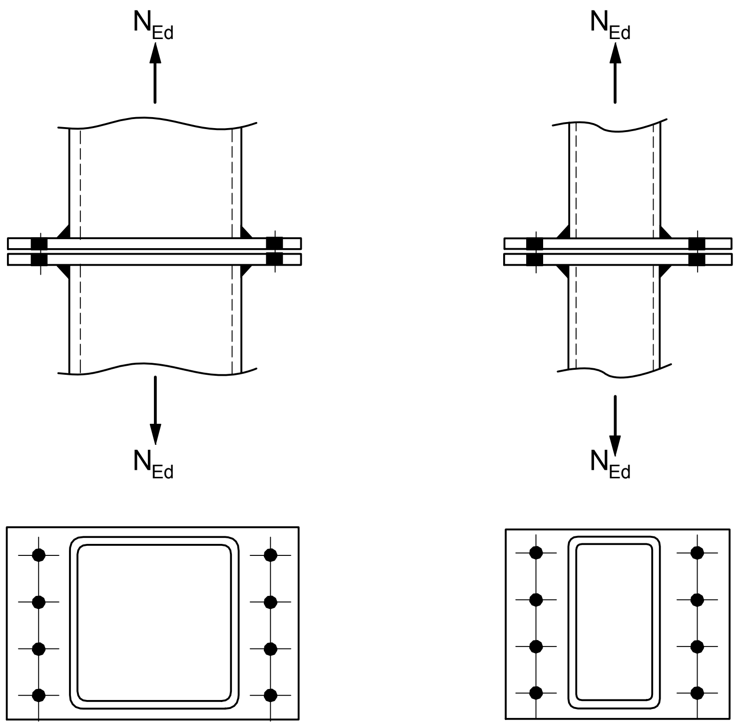
based on the conservative assumption that the tie force is resisted by the flange cover plates.

$N_{Ed,u}$ is the tensile force from BS EN 1991-1-7, A.6

6.7 DESIGN PROCEDURES FOR HOLLOW SECTION ‘CAP AND BASE’ SPLICES IN TENSION

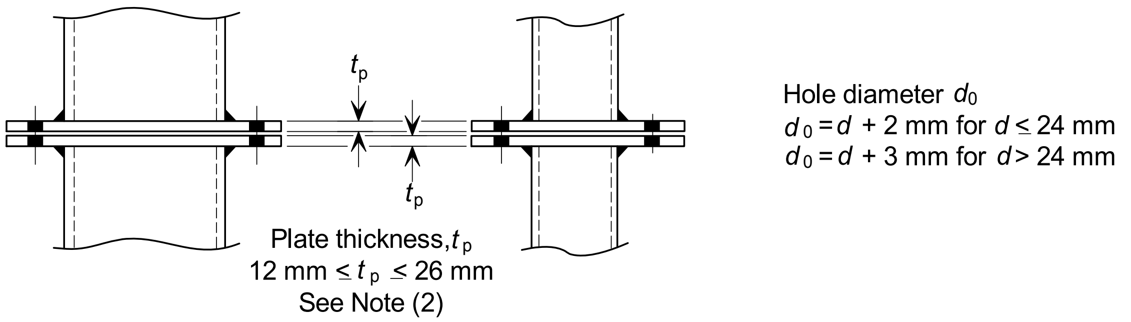
The design procedures for tension splices in square and rectangular hollow sections considers tension bolts being placed along two parallel faces as shown below. There are no structural integrity checks given since the design force is in tension.

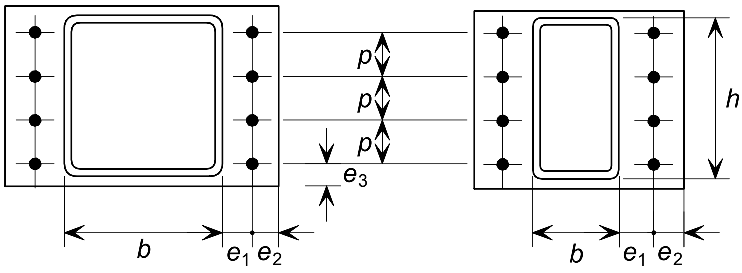
The semi-empirical rules are based on the recommendations given in the CIDECT Design Guide[34] and take account of prying forces.



Check	1	Recommended detailing practice		
Check	2	Connection	-	*(not applicable)*
Check	3	Connection	-	Plate and bolt resistance
Check	4	Connection	-	Bolt group
Check	5	Connection	-	Welds

CHECK 1	Recommended detailing practice





Edge distance $e_2 \geq 1.2d_0$

Edge distance $e_3 \geq 1.2d_0$

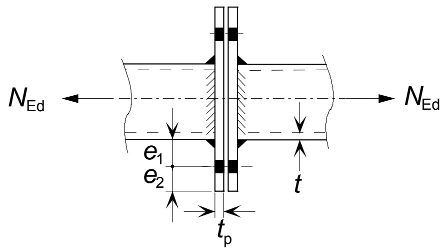
Total number of bolts $n \leq \dfrac{2h}{p} + 2$ but ≥ 4

Notes:

(1) Dimension e_1 should be minimised

(2) Plate thickness t_p should be limited to between 12 mm and 26 mm because this is the range for which the design method has been validated experimentally

CHECK 3	Plate and bolt resistance



Basic requirement:

$N_{Ed} \leq F_{Rd}$

F_{Rd} = design tension resistance of the T-stub

$$= \frac{t_p^{\,2}\left(1+\delta\alpha\right)n}{K}$$

$$\delta = 1-\frac{d_0}{p}$$

$$K = \frac{4\left(e_1-\left(d/2\right)+t\right)}{f_{y,p}\times p}$$

$$\alpha = \left(\frac{KF_{t,Fd}}{t_p^{\,2}}-1\right)\left(\frac{e_{eff}+\left(d/2\right)}{\delta\left(e_{eff}+e_1+t\right)}\right)$$

but $\alpha \geq 0$

where:

n is the total number of bolts

d_0 is the bolt hole diameter

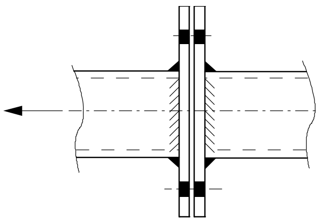
$F_{t,Rd}$ is the tension resistance of bolts (see box below)

e_{eff} = min (e_2; $1.25e_1$)

Tension resistance – 8.8 bolts

Bolt size	$F_{t,Rd}$ **(kN)**
M20	141
M24	203
M30	323

CHECK 4	Bolt group



Basic requirement:

$N_{Ed} \leq nF_{t,Rd}$

$F_{t,Rd} = \dfrac{k_2 f_{ub} A_s}{\gamma_{M2}}$

where:

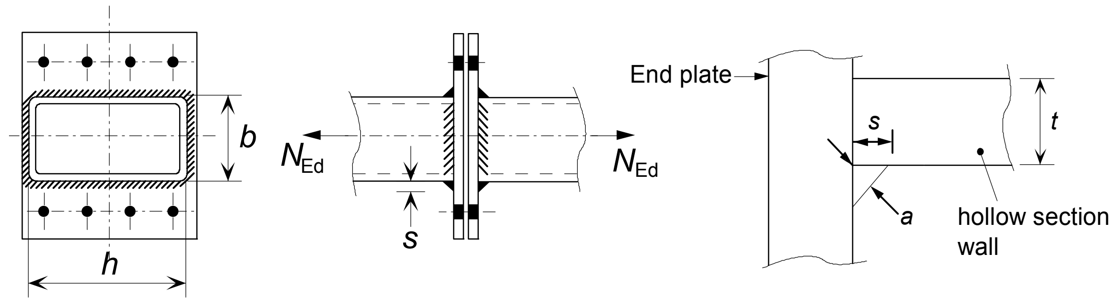
k_2 = 0.63 for countersunk bolts
= 0.9 for ordinary bolts

A_s is the tensile stress area of bolt

n is the total number of bolts

γ_{M2} is the partial factor for resistance of bolts (γ_{M2} = 1.25 as given in the National Annex)

CHECK 5	Welds



Weld resistance

Basic requirement:

$t \leq a$ (continuous full strength weld)

or

$$N_{Ed} \leq 2h\, a\, 1.225\, \frac{f_u}{\sqrt{3}\beta_w \gamma_{M2}}$$

where:

h	is the RHS depth
β_w	= 0.85for S275
	= 0.9 for S355
γ_{M2}	is the partial factor for the resistance of welds (γ_{M2} = 1.25 as given in the National Annex)
a	is the weld throat thickness

For a fillet weld:

a	= 0.7s
s	is the weld leg length

Notes:

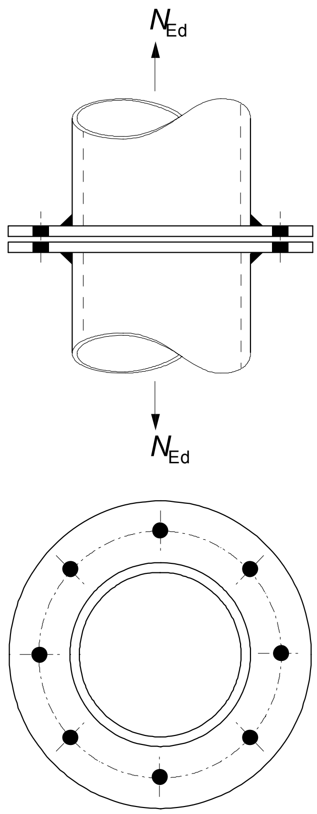
(1) The weld should be capable of developing the full strength of the hollow section. A fillet weld should normally be used, but if the required leg length exceeds 12 mm then a partial penetration butt weld with additional fillets may be a more economical solution.

(2) Depending on splice plate stiffness, the welded perimeter of the hollow section will not be uniformly loaded. In the absence of more precise design guidance, the effective weld length should be taken as the lengths adjacent to the bolts in tension. The same size weld should be used around the entire perimeter.

6.8 DESIGN PROCEDURES FOR CHS END PLATE SPLICE IN TENSION

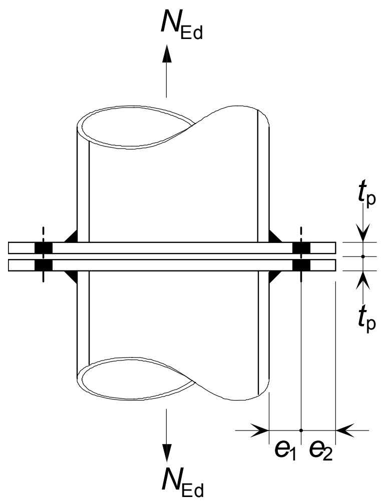
The design procedures for tension splices in circular hollow sections consider tension bolts being evenly placed around the section. There are no structural integrity checks given, since the principal force is tension.

The semi-empirical rules are based on the recommendations given in the CIDECT Design Guide[34].



Check 1	Recommended detailing practice		
Check 2	Connection	-	Plate resistance
Check 3	Connection	-	Plate and bolt resistance
Check 4	Connection	-	Bolt group
Check 5	Connection	-	Welds

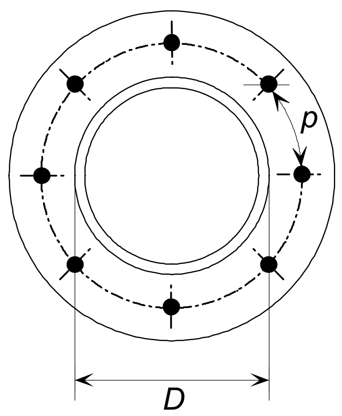
CHECK 1	Recommended detailing practice



Hole diameter d_0:

d_0 = d + 2 mm for $d \leq$ 24 mm

d_0 = d + 3 mm for $d >$ 27 mm



Bolt spacing p:

$p \geq 2.5d$

$p \leq 10d$

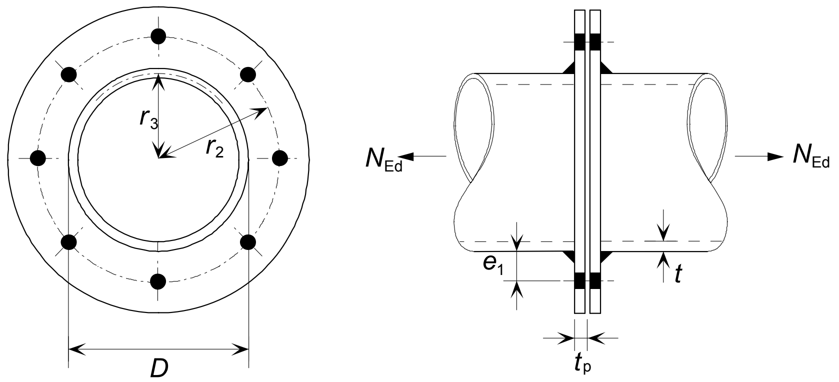
Notes:

(1) Bolts to be equally spaced around the member

(2) At least 4 bolts to be used

(3) Dimension e_1 to be minimised

CHECK 2	Plate resistance



Basic requirement:

$$F_{\mathrm{Ed}} \quad \leq \quad \frac{t_{\mathrm{p}}^{\;2} f_{\mathrm{y,p}}\, \pi\, f_3}{2\gamma_{\mathrm{M0}}}$$

where:

$$f_3 = \frac{1}{2k_1}\left(k_3 + \left(k_3^{\;2} - 4k_1\right)^{0.5}\right)$$

$$k_1 = \ln\left(\frac{r_2}{r_3}\right)$$

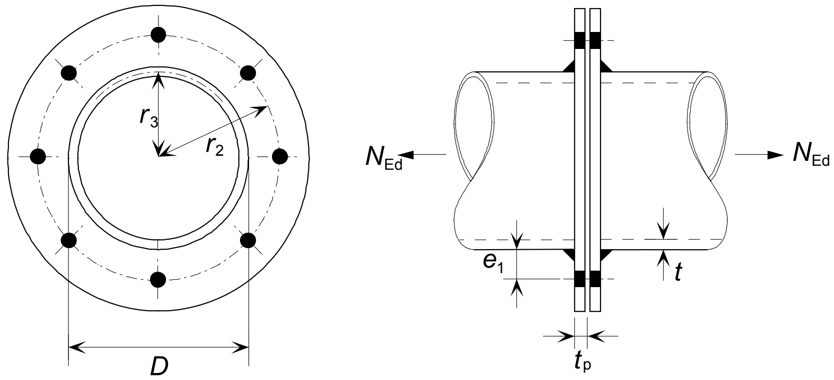
(ln = natural logarithm)

$$r_2 = \frac{D}{2} + e_1$$

$$r_3 = \frac{D - t}{2}$$

$$k_3 = k_1 + 2$$

CHECK 3	Plate and bolt resistance



Basic requirement:

$$F_{Ed} \leq \frac{nF_{t,Rd}}{\left(1 - \frac{1}{f_3} + \frac{1}{f_3 \ln(r_1 / r_2)}\right)\gamma_{M0}}$$

(ln = natural logarithm)

$F_{t,Rd}$ = tension resistance of a bolt

$$= \frac{k_2 f_{ub} A_s}{\gamma_{M2}}$$

Tension resistance – 8.8 bolts

Bolt size	**$F_{t,Rd}$ (kN)**
M20	141
M24	203
M30	323

where:

n is the total number of bolts

$$f_3 = \frac{1}{2k_1}\left(k_3 + \left(k_3^2 - 4k_1\right)^{0.5}\right)$$

$$k_1 = \ln\left(\frac{r_2}{r_3}\right)$$

(ln = natural logarithm)

$$r_1 = \frac{D}{2} + e_1 + e_{eff}$$

e_{eff} = min (e_2 ; $1.25e_1$)

$$r_2 = \frac{D}{2} + e_1$$

$$r_3 = \frac{D - t}{2}$$

$k_3 = k_1 + 2$

CHECK 4	Bolt group

Basic requirement:

$N_{Ed} \leq nF_{t,Rd}$

$F_{t,Rd} = \frac{k_2 f_{ub} A_s}{\gamma_{M2}}$ (see Check 3)

where:

k_2 = 0.63 for countersunk bolts

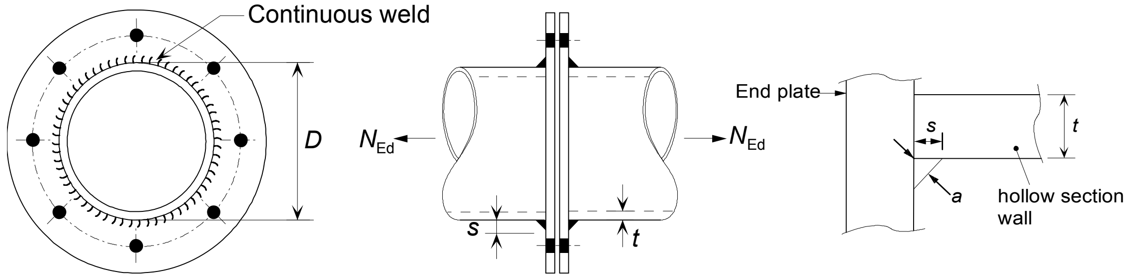
= 0.9 for ordinary bolts

A_s is the tensile stress area of the bolts

n is the total number of bolts

γ_{M2} is the partial factor for resistance of bolts (γ_{M2} = 1.25 as given in the National Annex)

CHECK 5	Welds



Weld resistance

Basic requirement:

$t \leq a$ (continuous full strength weld)

or

$$N_{Ed} \leq \pi \times D \times a \times \frac{f_u}{\sqrt{3}\beta_w \gamma_{M2}}$$

where:

D is the diameter of the CHS

β_w = 0.85 for S275

= 0.9 for S355

γ_{M2} is the partial factor for the resistance of welds (γ_{M2} = 1.25 as given in the National Annex)

a is the weld throat thickness

For a fillet weld:

a = 0.7s

s is the weld leg length

Note:

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 12 mm then a partial penetration butt weld with additional fillets may be a more economical solution.

6.9 WORKED EXAMPLES

The five worked examples for column 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 producing net tension (additional checks which have to be made are shown).

Example 3

A non-bearing splice for universal columns, with all forces transferred by the cover plates, and including a structural integrity check.

Example 4

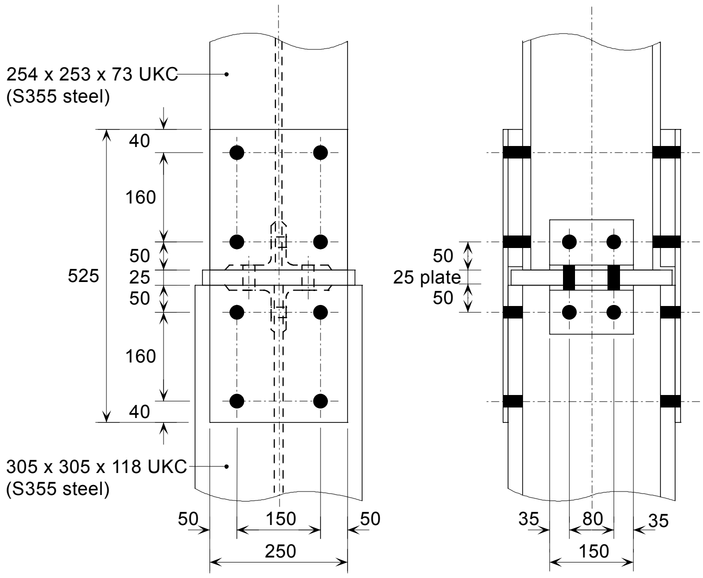
A splice in a rectangular hollow section subject to axial tension.

Example 5

A splice in a circular hollow section subject to axial tension.

SCI Steel Knowledge CALCULATION SHEET BCSA	Job	*Joints in Steel Construction - Simple Joints*	Sheet 1 of 4
	Title	*Example 1 – Column splice – Bearing splice (No net tension)*	
	Client	*Connections Group*	
	Calcs by *ENM*	Checked by *DGB*	Date *Sept 2011*

DESIGN EXAMPLE 1 – BEARING TYPE



Splice details

Flange cover plates:	2/250 × 12 × 525
Flange packs:	2/250 × 30 × 240
Cleats:	4/90 × 90 × 8 Angles × 150 Long
Web Packs:	2/85 × 2 × 150
Division plate:	265 × 25 × 310
Bolts:	M20 8.8
Fittings material:	S275 steel

Check the column splice for the following design forces.

$N_{Ed,G}$	825 kN	(permanent actions)
N_{Ed}	1767 kN	(total actions)
$M_{y,Ed}$	15 kNm	(about major axis of column)
V_{Ed}	8 kN	

Title Example 1 – Column Splices – UKC bearing splice (No net tension)	Sheet 2 of 4

SUMMARY OF FULL DESIGN CHECKS FOR EXAMPLE 1

No checks needed on shear, which can be resisted by friction across the bearing surfaces.

Sheet No.	CHECK	Resistance	Design force	Comments
3	***Check 1*** Recommended detailing practice	*N/A*	*N/A*	All recommendations adopted, within reasonable practical limits
3	***Check 2*** Flange cover plates presence of net tension	*N/A*	*N/A*	No net tension developed
3	***Check 5*** Minimum resistance	544	442	25% of N_{Ed}

Title Example 1 – Column splice – Bearing splice (No net tension) *Sheet 3 of 4*

Check 1: Recommended Detailing Practice

External flange cover plates

Height, h_{fp} $\geq 2b_{uc} + t_{dp}$ and ∴ Acceptable

Width, b_{fp} $\geq b_{uc}$ = 254.6 mm Say 250 mm ∴ Acceptable

Maximum vertical bolt spacing, $p_1 = 14t$, i.e. minimum thickness is $p_1/14$

Thickness, t_{fp} $\geq \frac{t_{f,uc}}{2}$, 10 mm, $\frac{p_1}{14}$

$= \frac{14.2}{2}$, 10 mm, $\frac{160}{14}$

= 7.1 mm, 10 mm, 11.4 mm Say 12 mm ∴ Acceptable

Packs, t_{pa} $= \frac{h_{lc} - h_{uc}}{2} = \frac{314.5 - 254.1}{2}$ = 30.2 mm Say 30 mm

Division plate

Thickness,

$$t_{dp} \geq \frac{\left[\left(h_{lc} - 2t_{f,lc}\right) - \left(h_{uc} - 2t_{f,uc}\right)\right]}{2}$$

$$= \frac{\left[\left(314.5 - 2\times 18.7\right) - \left(254.1 - 2\times 14.2\right)\right]}{2} = 25.7 \text{ mm} \quad \text{Say 25 mm}$$

Web cleats

Use 90×90×8 angles to accommodate M20 bolts.

Length $\geq 0.5h_{uc}$ = 0.5×254.1 = 127 mm Say 150 mm ∴ Acceptable

Packs, t_p $= \frac{t_{w,lc} - t_{w,uc}}{2} = \frac{12.0 - 8.6}{2}$ = 1.7 mm Say 2 mm ∴ Acceptable

Check 2: Flange Cover Plates

Presence of net tension

Basic requirement for no net tension: $M_{Ed} \leq \frac{N_{Ed,G} \times h}{2}$

$$\frac{N_{Ed,G} \times h}{2} = \frac{825 \times 254.1}{2} \times 10^{-3} = 105 \text{ kNm}$$

M_{Ed} = 15 kNm $\leq$ 105 kNm

Tension does not occur. ∴ **O.K.**

Check 5: Minimum resistance

Cover plates

Basic requirement: $0.25\,N_{Ed} \leq N_{Rd}$

$0.25\,N_{Ed} = 0.25 \times 1767 \times 10^{-3} = 442$ kN

$$N_{Rd} = \frac{2A_{fp} f_{y,fp}}{\gamma_{M0}} = \frac{2 \times 250 \times 12 \times 275}{1.0} \times 10^{-3} = 1650 \text{ kN}$$

1650 kN > 442 kN ∴ **O.K.**

Bolt group

Basic requirement: $0.25\,N_{Ed} \leq 2F_{Rd,fp}$

Title Example 1 – Column splice – Bearing splice (No net tension) | Sheet 4 of 4

$F_{Rd,fp} = nF_{b,Rd}$ if $F_{b,Rd} \leq F_{v,Rd}$

$F_{Rd,fp} = nF_{v,Rd}$ if $F_{v,Rd} < F_{b,Rd}$

Shear resistance of one bolt:

$$F_{v,Rd} = \beta_p \frac{\alpha_v f_{ub} A}{\gamma_{M2}}$$

$$\beta_p = \frac{9d}{8d + 3t_{pa}} = \frac{9 \times 20}{8 \times 20 + 3 \times 30} = 0.72$$

$$\therefore F_{v,Rd} = 0.72 \times \frac{0.6 \times 800 \times 245}{1.25} \times 10^{-3} = 68 \text{ kN}$$

Bearing resistance of one bolt:

$$F_{b,Rd} = \frac{k_1 \alpha_b f_u d t_{fp}}{\gamma_{M2}}$$

$$\alpha_d = \min\left(\frac{e_1}{3d_0};\quad \frac{p_1}{3d_0} - \frac{1}{4};\quad \frac{f_{ub}}{f_{u,p}};\quad 1.0\right) = \min\left(\frac{40}{3 \times 22};\quad \frac{160}{3 \times 22} - \frac{1}{4};\quad \frac{800}{410};\quad 1.0\right)$$

$$= \min\left(0.61;\quad 2.17;\quad 1.95;\quad 1.0\right) = 0.61$$

$$k_1 = \min\left(\frac{2.8 e_2}{d_0} - 1.7; \frac{1.4 p_1}{d_0} - 1.7; 2.5\right) = \min\left(\frac{2.8 \times 50}{22} - 1.7;\ \frac{1.4 \times 160}{22} - 1.7;\ 2.5\right)$$

$$= \min\left(4.66;\quad 8.48;\quad 2.5\right) = 2.5$$

$$\therefore F_{b,Rd} = \frac{2.5 \times 0.61 \times 410 \times 20 \times 12}{1.25 \times 10^3} = 120 \text{ kN}$$

As the column flange is thicker, with no reduced edge or end distances, and of higher grade, it is not necessary to check bearing in the flange.

$\therefore F_{Rd,fp} = nF_{v,Rd} = 4 \times 68 = 272$ kN

$2F_{Rd,fp} = 2 \times 272 = 544$ kN

442 kN < 544 kN $\qquad$ $\therefore$ **O.K.**

Steel Knowledge **CALCULATION SHEET** BCSA	Job *Joints in Steel Construction - Simple Joints*		Sheet *1* of *4*
	Title *Example 2 – Column splice – Bearing splice (Net tension developed)*		
	Client *Connections Group*		
	Calcs by *ENM*	Checked by *DGB*	Date *Sept 2011*

DESIGN EXAMPLE 2 – BEARING TYPE

Check the splice for the following design forces. The splice details are the same as Example 1.

$N_{Ed,G}$ 760 kN (permanent actions)

N_{Ed} 1630 kN (total actions)

$M_{y,Ed}$ 110 kNm (About major axis of column)

V_{Ed} 60 kN

(Note : In this example net tension is developed in the flange cover plates)

SUMMARY OF FULL DESIGN CHECKS FOR EXAMPLE 2

Sheet No.	CHECK	Resistance	Design force	Comments
2	***Check 1*** Recommended detailing practice	Not Applicable	Not Applicable	All recommendations adopted, within reasonable practical limits
2	***Check 2*** Flange cover plate Presence of net tension	96.6	110	Net tension developed, ordinary bolts adequate
2	***Check 3*** Plate resistance	825	53	
3	***Check 4*** Bolt group	271	53	No need to check column flange in this example
4	***Check 5*** Minimum resistance	542	408	
4	***Check 6*** Tying resistance	Not applicable		If necessary, carry out Checks 2 and 3 with the tying force from BS EN 1991-1-7, A.6

Title Example 2 – Column splices –Bearing splice (Net tension developed) Sheet 2 of 4

Check 1: Recommended Detailing Practice

As for Example 1

Check 2: Flange cover plates

Presence of net tension

Basic requirement for no net tension: $M_{Ed} \leq \dfrac{N_{Ed,G} \times h}{2}$

$$\frac{N_{Ed,G} \times h}{2} = \frac{760 \times 254.1}{2} \times 10^{-3} = 96.6 \text{ kNm}$$

$M_{Ed} = 110$ kNm $\nleq$ 96.6 kNm

Net tension does occur and the flange cover plates and their fastenings must be checked for a tensile force N_{Ed}.

$$N_{Ed} = \frac{M_{Ed}}{h} - \frac{N_{Ed,G}}{2} = \left(\frac{110 \times 10^6}{254.1} - \frac{760 \times 10^3}{2}\right) \times 10^{-3} = 52.9 \text{ kN}$$

Check for the suitability of ordinary bolts.

It is sufficiently accurate to base the folowing calculation on the gross area of the flange

$$\frac{N_{Ed}}{t_{f,uc} b_{f,uc} f_{y,uc}} = \frac{52.9 \times 10^3}{14.2 \times 254.6 \times 355} = 0.04 < 0.1$$

$\therefore$ **O.K.**

There is no significant net tension in the column flange and the use of ordinary bolts in clearance holes is satisfactory.

Check 3: Plate resistance

Basic requirement: $N_{Ed} \leq N_{t,Rd}$

$N_{t,Rd} = \min(N_{pl,Rd}\ ;\ N_{u,Rd};\ N_{bt,Rd})$

Tension resistance of the gross area

$$N_{pl,Rd} = \frac{A_{fp} f_{y,fp}}{\gamma_{M0}}$$

Gross area:

$A_{fp} = 250 \times 12 = 3000$ mm^2

$$\therefore N_{pl,Rd} = \frac{3000 \times 275}{1.0} \times 10^{-3} = 825 \text{ kN}$$

Tension resistance of the net area

$$N_{u,Rd} = \frac{0.9 A_{fp,net} f_{u,fp}}{\gamma_{M2}}$$

Net area:

$A_{fp,net} = 12 \times (250 - 2 \times 22) = 2472$ mm^2

$$\therefore N_{u,Rd} = \frac{0.9 \times 2472 \times 410}{1.1} \times 10^{-3} = 829 \text{ kN}$$

Block tearing resistance

For a concentrically loaded bolt group: $N_{bt,Rd} = \dfrac{f_{u,fp} A_{fp,nt}}{\gamma_{M2}} + \dfrac{f_{y,fp} A_{fp,nv}}{\sqrt{3}\gamma_{M0}}$

Title *Example 2 – Column splice –Bearing splice (Net tension developed)* *Sheet 3 of 4*

p_2 = 150 mm

$2e_2$ = 2×50 = 100 mm

p_2 > $2e_2$, therefore:

$A_{\mathrm{fp,nt}} = t_{\mathrm{p}}(2e_2 - d_0) = 12 \times (2 \times 50 - 22) = 936\ \mathrm{mm}^2$

$A_{\mathrm{fp,nv}} = 2t_{\mathrm{p}}(e_1 + (n_1 - 1)p_1 - (n_1 - 0.5)d_0) = 2 \times 12 \times (40 + (2-1) \times 160 - (2-0.5) \times 22)$

$\therefore A_{\mathrm{fp,nv}} = 4008\ \mathrm{mm}^2$

$$N_{\mathrm{bt,Rd}} = \left(\frac{410 \times 936}{1.1} + \frac{275 \times 4008}{\sqrt{3} \times 1.0}\right) \times 10^{-3} = 985\ \mathrm{kN}$$

$\therefore N_{\mathrm{t,Rd}} = \min(825;\ 829;\ 985) = 825\ \mathrm{kN}$

$F_{\mathrm{Ed}} = 52.9\ \mathrm{kN} < 825\ \mathrm{kN}$ **$\therefore$ O.K.**

Check 4: Bolt group

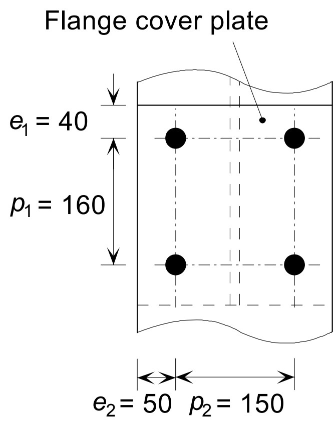
Shear and bearing resistance of the bolt group

Basic requirement: $N_{\mathrm{Ed}} \leq F_{\mathrm{Rd,fp}}$

The design resistance of the bolt group, $F_{\mathrm{Rd,fp}}$:

$F_{\mathrm{Rd,fp}} = \Sigma F_{\mathrm{b,Rd}}$ if $F_{\mathrm{b,Rd}} \leq F_{\mathrm{v,Rd}}$

$F_{\mathrm{Rd,fp}} = nF_{\mathrm{v,Rd}}$ if $F_{\mathrm{v,Rd}} < F_{\mathrm{b,Rd}}$



The shear resistance of a single bolt, $F_{\mathrm{v,Rd}} = \dfrac{\alpha_{\mathrm{v}} f_{\mathrm{ub}} A}{\gamma_{\mathrm{M2}}}$

Joint length, L_{j} = 160 mm

$15d = 15 \times 20 = 300$ mm

$L_{\mathrm{j}} < 15d$

Therefore there is no long joint effect.

Total thickness of flange pack, t_{pa} = 30 mm

$\dfrac{d}{3} = 6.7$ mm

$t_{\mathrm{p}} \not< \dfrac{d}{3}$

Therefore $F_{\mathrm{v,Rd}}$ must be multiplied by a reduction factor β_{p}.

$$\beta_{\mathrm{p}} = \frac{9d}{8d + 3t_{\mathrm{p}}} = \frac{9 \times 20}{8 \times 20 + 3 \times 30} = 0.72$$

$$F_{\mathrm{v,Rd}} = 0.72 \times \frac{0.6 \times 800 \times 245}{1.25} \times 10^{-3} = 67.7\ \mathrm{kN}$$

Bearing resistance, $F_{\mathrm{b,Rd}} = \dfrac{k_1 \alpha_{\mathrm{b}} f_{\mathrm{u}} d t_{\mathrm{fp}}}{\gamma_{\mathrm{M2}}}$

$$k_1 = \min\left(2.8\frac{e_2}{d_0} - 1.7;\quad 1.4\frac{p_2}{d_0} - 1.7;\quad 2.5\right)$$

$$k_1 = \min\left(2.8 \times \frac{50}{22} - 1.7;\ 1.4 \times \frac{150}{22} - 1.7;\ 2.5\right) = \min(4.66;\ \ 7.85;\ \ 2.5) = 2.5$$

Title Example 2 – Column splices –Bearing splice (Net tension developed) Sheet 4 of 4

$\alpha_b = \min\left(\dfrac{e_1}{3d_0};\ \dfrac{p_1}{3d_0}-\dfrac{1}{4};\ \dfrac{f_{ub}}{f_{u,fp}};\ 1.0\right)$

$\alpha_b = \min\left(\dfrac{40}{3\times 22};\ \dfrac{150}{3\times 22}-\dfrac{1}{4};\ \dfrac{800}{410};\ 1.0\right) = \min(0.61;\ 2.02;\ 1.95;\ 1.0) = 0.61$

$F_{b,Rd,end} = \dfrac{2.5\times 0.61\times 410\times 20\times 12}{1.25}\times 10^{-3} = 120$ kN

67.7 kN < 120 kN

$\therefore F_{Rd,fp} = n\times F_{v,Rd} = 4\times 67.7 = 271$ kN

$\therefore N_{Ed} = 52.9$ kN < 271 kN ∴ **O.K.**

Check 5: Minimum resistance

Cover plates

Basic requirement: $0.25\,N_{Ed} \leq N_{c,Rd}$

$0.25\,N_{Ed} = 0.25\times 1630 = 408$ kN

From Example 1 the resistance in compression is 1650 kN.

408 kN < 1650 kN ∴ **O.K.**

Bolt group

Basic requirement: $0.25\,N_{Ed} \leq 2F_{Rd,fp}$

From Check 4 $F_{Rd,fp}$ is 271 kN.

$2\times 271 = 542$ kN

408 kN < 542 kN ∴ **O.K.**

Check 6: Tying resistance

If it is necessary to comply with structural integrity requirements, then Checks 3, 4 and 5 should be carried out with:

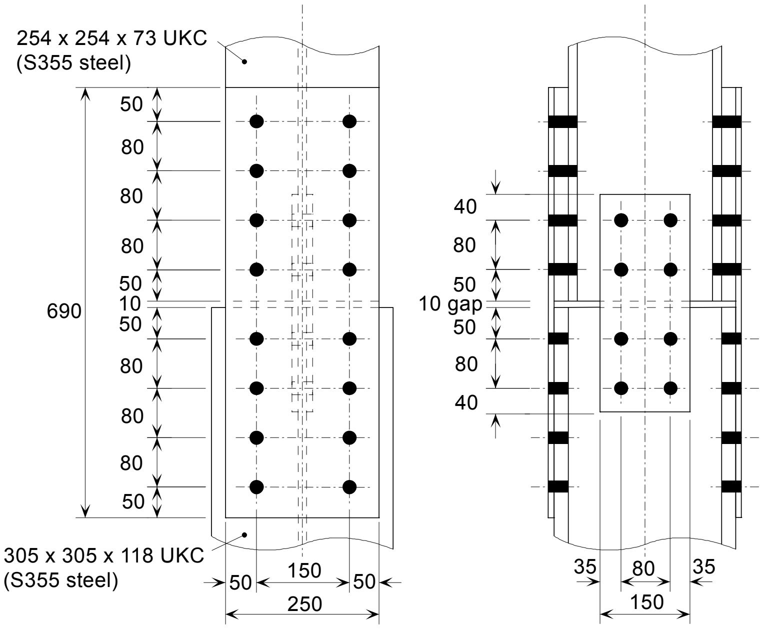
$N_{Ed} = \dfrac{N_{Ed,u}}{2}$

Based on the conservative assumption that the tying force is resisted by the flange cover plates.

$N_{Ed,u}$ is the tying force obtained from EN 1991-1-7, clause A.6.

SCI Steel Knowledge CALCULATION SHEET BCSA	Job: *Joints in Steel Construction – Simple Joints*		Sheet *1* of *9*
	Title: *Example 3 - Column splice – Non-bearing splice*		
	Client: *Connections Group*		
	Calcs by: *CZT*	Checked by: *ENM*	Date: *Sept 2011*

DESIGN EXAMPLE 3 – NON BEARING COLUMN SPLICE



Splice details

Flange cover plates:	2/250 × 12 × 690
Flange packs:	2/250 × 30 × 340
Web cover plates:	2/150 × 8 × 350
Web Packs:	2/150 × 2 × 170
Bolts:	M24 8.8
Fittings material:	S275 steel

Check the column splice shown for the following design forces:

$N_{Ed,G}$	825 kN	(permanent actions)
N_{Ed}	942 kN	(total actions)
$M_{y,Ed}$	15 kNm	(about major axis of the column)
V_{Ed}	8 kN	
$N_{Ed,u}$	400 kN	(from floor below splice)

Title Example 3 – Column splice – Non-bearing splice	Sheet 2 of 9

SUMMARY OF FULL DESIGN CHECKS FOR EXAMPLE 3

Sheet No.	CHECK		Resistance	Design force	Comments
3	***Check 1*** Recommended detailing practice		Not Applicable	Not Applicable	All recommendations adopted, within reasonable practical limits
3	***Check 2*** Flange cover plate Plate resistance		No net tension 817	745	
4	***Check 3*** Flange cover plate Bolt Group		840	745	
5	***Check 4*** Web cover plates Plate resistance		330	199	
5	***Check 5*** Web cover plates Bolt group		532	398	
6	***Check 6*** Minimum resistance – major axis	I ii iii iv	269 260 274 343 136	88 88 88 83 24.8	
7	***Check 7*** Minimum resistance – minor axis	i ii iii iv	69 105 779 105	27 17.7 83 13.9	
8	***Check 8*** Tying resistance Cover plates Bolt group		797 840	200 200	

Title Example 3 – Column splices – Non-bearing splice *Sheet 3 of 9*

Check 1: Recommended Detailing Practice

Bolt diameter $\geq$ 75% of thickness of packing either side

24 mm > 0.75×30 = 22.5 mm

If this is found to be impractical then the packs may be welded to the column flanges and this check completed for any remaining loose (unwelded) packs. All other requirements are satisfied as in worked example 1.

Check 2: Flange cover plates

Plate resistance

Compression

Basic requirement: $N_{\mathrm{Ed,c}} \leq N_{\mathrm{c,Rd}}$

Conservatively:

$$N_{\mathrm{Ed,c}} = \frac{M_{\mathrm{Ed}}}{h} + N_{\mathrm{Ed}}\left(\frac{A_{\mathrm{f,1}}}{A_1}\right)$$

A_1 $= 93.1 \times 10^2$ mm^2

$A_{\mathrm{f,1}}$ $= h_1 t_{\mathrm{f,1}} = 254.6 \times 14.2$ $= 3615$ mm^2

$$\therefore N_{\mathrm{Ed,c}} = \left(\frac{15 \times 10^6}{254.1} + (825 + 942) \times 10^3 \times \left(\frac{3615}{93.1 \times 10^2}\right)\right) \times 10^{-3} = 745 \text{ kN}$$

$$N_{\mathrm{c,Rd}} = \chi \frac{A_{\mathrm{fp}} f_{\mathrm{y,fp}}}{\gamma_{\mathrm{M0}}}$$

$$\frac{p_{1,\mathrm{j}}}{t_{\mathrm{fp}}} = \frac{110}{12} = 9.17$$

Note 2 in Table 3.3 of BS EN 1993-1-8

$$\varepsilon = \sqrt{\frac{235}{f_{\mathrm{y,fp}}}} = \sqrt{\frac{235}{275}} = 0.92$$

9ε $= 9 \times 0.92$ $= 8.28$

$\frac{p_{1,\mathrm{j}}}{t_{\mathrm{fp}}} \not< 9\varepsilon$, therefore plate buckling must be checked

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \quad \text{but} \quad \chi \leq 1.0$$

$$\bar{\lambda} = \frac{L_{\mathrm{cr}}}{i_z}\frac{1}{\lambda_1}$$

$$i_z = \sqrt{\frac{I_z}{A}} = \sqrt{\frac{\frac{250 \times 12^3}{12}}{250 \times 12}} = 3.46 \text{ mm}$$

The buckling length L_{cr} is taken as $0.6 \times p_{1,\mathrm{j}}$

$$\bar{\lambda} = \frac{0.6 \times 110}{3.46}\frac{1}{93.9 \times 0.92} = 0.22$$

$$\phi = 0.5\left[1 + \alpha\left(\bar{\lambda} - 0.2\right) + \bar{\lambda}^2\right]$$

For a solid section in S275 steel use buckling curve *c*, therefore, $\alpha = 0.49$

$$\phi = 0.5\left[1 + 0.49(0.22 - 0.2) + 0.22^2\right] = 0.53$$

$\therefore \chi = \dfrac{1}{0.53 + \sqrt{0.53^2 - 0.22^2}} = 0.99$

$\therefore N_{c,Rd} = 0.99 \times \dfrac{250 \times 12 \times 275}{1.0} \times 10^{-3} = 817$ kN

$\therefore N_{Ed,c} = 745$ kN < 817 kN **$\therefore$ O.K.**

Tension

Basic requirement: $N_{Ed,t} \leq N_{t,Rd}$

$N_{Ed,t} = \dfrac{M_{Ed}}{h} - F_{Ed,G}\left(\dfrac{A_{f,1}}{A_1}\right) = \dfrac{15 \times 10^6}{254.1} - 825 \times 10^3 \times \left(\dfrac{3615}{93.1 \times 10^2}\right) \times 10^{-3} = -261$ kN

This indicates a net compression of 261 kN. Therefore tension is not present.

Assuming that slip is acceptable, and since there is no net tension, the use of ordinary bolts is satisfactory.

Check3: Flange bolt group

Bolt group

Basic requirement: $N_{Ed} \leq F_{Rd,fp}$

The design resistance of the bolt group, $F_{Rd,fp}$:

$F_{Rd,fp} = nF_{b,Rd}$ if $F_{b,Rd} \leq F_{v,Rd}$

$F_{Rd,fp} = nF_{v,Rd}$ if $F_{v,Rd} < F_{b,Rd}$

The shear resistance of a single bolt:

$F_{v,Rd} = \dfrac{\alpha_v f_{ub} A}{\gamma_{M2}}$

Joint length, $L_j = 3 \times 80 = 240$ mm

$15d = 15 \times 24 = 360$ mm

$L_j < 15d$

Therefore there is no long joint effect.

Total thickness of flange pack, $t_{pa} = 30$ mm

$\dfrac{d}{3} = 8$ mm

$t_{pa} \not< \dfrac{d}{3}$

Therefore $F_{v,Rd}$ must be multiplied by a reduction factor β_p.

$\beta_p = \dfrac{9d}{8d + 3t_{pa}} = \dfrac{9 \times 24}{8 \times 24 + 3 \times 30} = 0.77$

$\therefore F_{v,Rd} = 0.77 \times \dfrac{0.6 \times 800 \times 353}{1.25} \times 10^{-3} = 105$ kN

Bearing resistance, $F_{b,Rd} = \dfrac{k_1 \alpha_b f_u d t_{fp}}{\gamma_{M2}}$

$k_1 = \min\left(2.8\dfrac{e_2}{d_0} - 1.7;\quad 1.4\dfrac{p_2}{d_0} - 1.7;\quad 2.5\right)$

$k_1 = \min\left(2.8 \times \dfrac{50}{26} - 1.7;\quad 1.4 \times \dfrac{150}{26} - 1.7;\quad 2.5\right) = \min(3.7;\quad 6.4;\quad 2.5) = 2.5$

Title Example 3 – Column splices – Non-bearing splice | Sheet 5 of 9

$\alpha_b = \min\left(\frac{e_1}{3d_0};\ \frac{p_1}{3d_0} - \frac{1}{4};\ \frac{f_{ub}}{f_{u,fp}};\ 1.0\right)$

$\alpha_b = \min\left(\frac{50}{3\times 26};\ \frac{80}{3\times 26} - \frac{1}{4};\ \frac{800}{410};\ 1.0\right) = \min(0.64;\ 0.78;\ 1.95;\ 1.0) = 0.64$

$F_{b,Rd} = \frac{2.5\times 0.64\times 410\times 24\times 12}{1.25}\times 10^{-3} = 151$ kN

Thus $F_{v,Rd} < F_{b,Rd}$

$\therefore F_{Rd,fp} = n\times F_{v,Rd} = 8\times 105 = 840$ kN

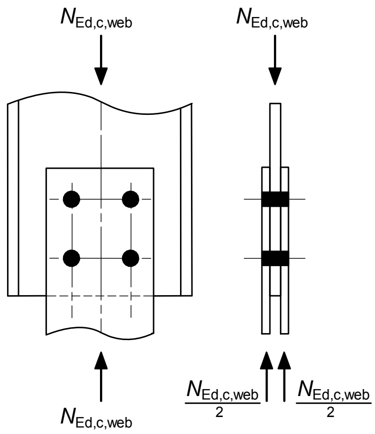
$\therefore N_{Ed} = 745$ kN < 840 kN

∴ **O.K.**

Check 4: Web cover plate

Plate resistance

Basic requirement: $\frac{N_{Ed,c,web}}{2} \leq N_{c,Rd,wp}$



$N_{Ed,c,web} = \frac{N_{Ed}A_w}{A_1}$

$N_{c,Rd,wp} = \frac{A_{wp}f_y}{\gamma_{M0}}$

$A_w = A - 2A_f$

$A_w = 93.1\times 10^2 - 2\times 254.1\times 14.2 = 2094$ mm^2

$N_{Ed,c,web} = \frac{(825+942)\times 2094}{93.1\times 10^2} = 398$ kN

$N_{c,Rd,wp} = \frac{150\times 8\times 275}{1.0}\times 10^{-3} = 330$ kN

$\frac{N_{Ed,c,web}}{2} = \frac{398}{2} = 199$ kN

199 kN < 330 kN

∴ **O.K.**

Check 5: Web cover plate bolt group

Shear and bearing resistance of the bolt group

Basic requirement: $N_{Ed,c,web} \leq F_{Rd,wp}$

The design resistance of the bolt group, $F_{Rd,fp}$

$F_{Rd,wp} = nF_{b,Rd}$ if $F_{b,Rd} \leq F_{v,Rd}$

$F_{Rd,wp} = nF_{v,Rd}$ if $F_{v,Rd} < F_{b,Rd}$

The resistance of a single bolt in double shear, $F_{v,Rd} = \beta_p \frac{2\alpha_v f_{ub} A}{\gamma_{M2}}$

Joint length, L_j = 80 mm

$15d = 15\times 24 = 360$ mm

$L_j < 15d$

Therefore there is no long joint effect.

Total thickness of flange pack, t_{pa} = 2 mm

$$\frac{d}{3} = 8 \text{ mm}$$

$$t_{pa} < \frac{d}{3}$$

Therefore $F_{v,Rd}$ does not need to be multiplied by a reduction factor β_p.

$$\therefore F_{v,Rd} = \frac{2 \times 0.6 \times 800 \times 353}{1.25} \times 10^{-3} = 272 \text{ kN}$$

Bearing resistance, $F_{b,Rd} = \dfrac{2k_1 \alpha_b f_u d t_{fp}}{\gamma_{M2}}$

$$k_1 = \min\left(2.8\frac{e_2}{d_0} - 1.7;\quad 1.4\frac{p_2}{d_0} - 1.7;\quad 2.5\right)$$

$$k_1 = \min\left(2.8 \times \frac{35}{26} - 1.7;\quad 1.4 \times \frac{80}{26} - 1.7;\quad 2.5\right) = \min(2.07;\quad 2.61;\quad 2.5) = 2.07$$

$$\alpha_b = \min\left(\frac{e_1}{3d_0};\quad \frac{p_1}{3d_0} - \frac{1}{4};\quad \frac{f_{ub}}{f_{u,fp}};\quad 1.0\right)$$

$$\alpha_b = \min\left(\frac{40}{3 \times 26};\quad \frac{80}{3 \times 26} - \frac{1}{4};\quad \frac{800}{410};\quad 1.0\right) = \min(0.51;\quad 0.78;\quad 1.95;\quad 1.0) = 0.51$$

$$F_{b,Rd} = \frac{2 \times 2.07 \times 0.51 \times 410 \times 24 \times 8}{1.25} \times 10^{-3} = 133 \text{ kN}$$

Thus $F_{b,Rd} < F_{v,Rd}$

$\therefore F_{Rd,fp} = n \times F_{b,Rd} = 4 \times 133 = 532$ kN

$N_{Ed,c,web}$ = 398 kN

398 kN < 532 kN **∴ O.K.**

Check 6: Minimum resistance requirements – major axis

(i) Bending – cover plates

Basic requirement: $0.25\, M_{c,y,Rd} \leq h\, N_{C,Rd}$

$M_{c,y,Rd}$ = 352 kNm (upper section) P363[23]

h = 314.5 + 12 = 326.5 mm

$$N_{Rd,c} = N_{pl,Rd} = \frac{250 \times 12 \times 275}{1.0} \times 10^{-3} = 825 \text{ kN}$$

$h \times N_{C,Rd}$ = 326.5 × 825 × 10^{-3}= 269 kNm

0.25 $M_{c,y,Rd}$ = 0.25 × 352 = 88 kNm

88 kNm < 269 kNm **∴ O.K.**

A_{net} = 250 × 12 – (2 × 26 × 12) = 2376 mm^2

$N_{u,Rd}$ = 0.9 × 2376 × 410 / 1.1 × 10^{-3} = 797 kN

$h \times N_{u,Rd}$ = 326.5 × 797 × 10^{-3} = 260 kNm

88 kNm < 260 kNm **∴ O.K.**

(ii) Bending – bolt group

Basic requirement: $0.25\, M_{c,y,Rd} \leq h\, F_{Rd,fp}$

From Check 3, $F_{Rd,fp}$ = 840 kN

$h\, F_{Rd,fp}$ = 326.5 × 840 × 10^{-3}= 274 kNm

88 kNm < 274 kNm **∴ O.K.**

Title *Example 3 – Column splices – Non-bearing splice* | *Sheet 7 of 9*

(iii) Shear resistance – cover plates

Basic requirement: $0.025\ N_{\text{Rd,c}} \quad \leq \quad V_{\text{pl,Rd,wp}}$

$N_{\text{Rd,c}}$ = 3310 kN — P363[23]

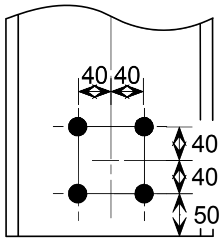
$0.025\ N_{\text{Rd,c}}$ = 0.025 × 3310 = 83 kN

$$V_{\text{pl,Rd,wp}} = \frac{2 \times 0.9 \times 8 \times 150 \times 275}{\sqrt{3} \times 1.0} \times 10^{-3} = 343 \text{ kN}$$

83 kN < 343 kN — ∴ **O.K.**

(iv)Shear resistance – bolt group

The bolt group must be checked for an applied shear of $0.025\ N_{\text{Rd,c}}$



$0.025\ N_{\text{Rd,c}}$ = 0.025 × 3310= 83 kN

Inertia of bolt group = $4 \times 40^2 + 4 \times 40^2$

= 12800 mm^4

Applied moment = $83 \times (50 + 0.5 \times 80) \times 10^{-3}$

= 7.5 kNm

Horizontal force per bolt due to moment = $\dfrac{7.5 \times 40 \times 10^3}{12800}$ = 23 kN

Vertical force per bolt due to moment = $\dfrac{7.5 \times 40 \times 10^3}{12800}$ = 23 kN

Horizontal force per bolt due to shear = $\dfrac{83}{4}$ = 21 kN

Resultant = $\sqrt{(23 + 21)^2 + 23^2}$ = 49.6 kN or 24.8 kN per cover plate

From Check 5, $F_{\text{v,Rd}}$ = 272 kN in double shear.

In single shear, $F_{\text{v,Rd}}$ = 136 kN > 24.8 kN, — ∴ **O.K.**

By inspection, the bearing resistance is not critical.

Title *Example 3 – Column splice – Non-bearing splice* *Sheet 8 of 9*

Check 7 Minimum resistance checks – minor axis

(i) Bending – cover plates

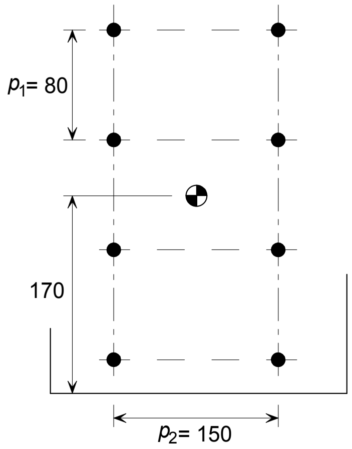
Basic requirement: $0.25\ M_{c,z,Rd} \leq M_{c,Rd,fp}$

$M_{c,z,Rd}$ = 109 kNm (upper section) P363[23]

$$M_{c,Rd,fp} = \frac{2 \times 12 \times 250^2 \times 275}{6 \times 1.0} \times 10^{-6} = 69 \text{ kNm}$$

$0.25\ M_{c,z,Rd} = 0.25 \times 109 = 27$ kNm

27 kNm < 69 kNm ∴ **O.K.**



(ii) Bending – bolt group

The resistance of the bolt group in each flange must be verified under the applied moment ($0.025\ M_{c,z,Rd}$) in each flange.

Inertia of bolt group

$= 8 \times 75^2 + 4 \times 40^2 + 4 \times 120^2$

$= 109000$ mm^4

Vertical force on extreme bolt

$$= \frac{0.125 \times 109 \times 75 \times 10^3}{109000} = 9.4 \text{ kN}$$

Horizontal force on extreme bolt

$$= \frac{0.125 \times 109 \times 120 \times 10^3}{109000} = 15 \text{ kN}$$

Resultant $= \sqrt{15^2 + 9.4^2} = 17.7$ kN

From Check 3, $F_{v,Rd}$ = 105 kN

17.7 kN < 105 kN ∴ **O.K.**

By inspection, the bearing resistance is not critical

(iii) Shear resistance – cover plates

Basic requirement: $0.025\ N_{Rd,c} \leq V_{pl,Rd,fp}$

$0.025\ N_{Rd,c} = 0.025 \times 3310 = 83$ kN

$$V_{pl,Rd,fp} = \frac{2 \times 0.9 \times 250 \times 12 \times 275}{\sqrt{3} \times 1.1} \times 10^{-3} = 779 \text{ kN}$$

83 kN < 779 kN ∴ **O.K.**

(iv) Shear resistance – bolt group

The bolt group must be checked for an applied shear of $0.025\ N_{Rd,c}$

$0.025\ N_{Rd,c} = 0.025 \times 3310 = 83$ kN

Applied moment $= 83 \times 170 \times 10^{-3} = 14$ kNm or 7 kNm per flange

From Check 7(ii):

Vertical force on extreme bolt due to moment $= \dfrac{7 \times 75}{109000} \times 10^3 = 4.8$ kN

Title *Example 3 – Column splices – Non-bearing splice*	*Sheet 9 of 9*

Horizontal force on extreme bolt due to moment $= \frac{7 \times 120}{109000} \times 10^3 = 7.7$ kN

Shear per bolt $= \frac{83}{2 \times 8} = 5.2$ kN

Resultant $= \sqrt{(5.3 + 7.7)^2 + 4.8^2} = 13.9$ kN

From Check 3, $F_{\mathrm{v,Rd}} = 105$ kN

13.9 kN < 105 kN ∴ **O.K.**

Check 8: Tying resistance

The maximum factored vertical load applied to the column from any floor down to the next splice

$N_{\mathrm{Ed,u}} = 400$ kN

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:

$$N_{\mathrm{Ed}} = \frac{400}{2} = 200 \text{ kN}$$

Plate resistance

Basic requirement: $N_{\mathrm{Ed}} \leq N_{\mathrm{t,Rd}}$

$N_{\mathrm{t,Rd}} = \min(N_{\mathrm{pl,Rd}};\ N_{\mathrm{u,Rd}};\ N_{\mathrm{bt,Rd}})$

Tension resistance of the gross area

$$N_{\mathrm{pl,Rd}} = \frac{A_{\mathrm{fp}} f_{\mathrm{y,fp}}}{\gamma_{\mathrm{M0}}}$$

Gross area:

$A_{\mathrm{fp}} = 250 \times 12 = 3000\ \mathrm{mm}^2$

$$\therefore N_{\mathrm{pl,Rd}} = \frac{3000 \times 275}{1.0} \times 10^{-3} = 825 \text{ kN}$$

Tension resistance of the net area at ultimate

$$N_{\mathrm{u,Rd}} = \frac{0.9 A_{\mathrm{fp,net}} f_{\mathrm{u,fp}}}{\gamma_{\mathrm{M2}}}$$

Net area:

$A_{\mathrm{fp,net}} = 12 \times (250 - 2 \times 26) = 2376\ \mathrm{mm}^2$

$$\therefore N_{\mathrm{u,Rd}} = \frac{0.9 \times 2376 \times 410}{1.1} \times 10^{-3} = 797 \text{ kN}$$

Block tearing resistance

For a concentrically loaded bolt group: $N_{\mathrm{bt,Rd}} = \frac{f_{\mathrm{u,fp}} A_{\mathrm{fp,nt}}}{\gamma_{\mathrm{M2}}} + \frac{f_{\mathrm{y,fp}} A_{\mathrm{fp,nv}}}{\sqrt{3}\gamma_{\mathrm{M0}}}$

$p_2 = 150$ mm

$2e_2 = 2 \times 50 = 100$ mm

$p_2 > 2e_2$, therefore:

$\therefore A_{\mathrm{fp,nt}} = t_{\mathrm{p}}(2e_2 - d_0) = 12 \times (2 \times 50 - 26) = 888\ \mathrm{mm}^2$

$A_{\mathrm{fp,nv}} = 2t_{\mathrm{p}}(e_1 + (n_1 - 1)p_1 - (n_1 - 0.5)d_0) = 2 \times 12 \times (50 + (4 - 1) \times 80 - (4 - 0.5) \times 26)$

$\therefore A_{\mathrm{fp,nv}} = 4776\ \mathrm{mm}^2$

Title Example 3 – Column splice – Non-bearing splice *Sheet 10 of 9*

$N_{bt,Rd} = \left(\frac{410 \times 888}{1.1} + \frac{275 \times 4776}{\sqrt{3} \times 1.0} \right) \times 10^{-3} = 1089$ kN

$\therefore N_{t,Rd} = \min(825;\ 797;\ 1089) = 797$ kN

$F_{Ed} = 200$ kN < 797 kN $\therefore$ **O.K.**

Bolt group

Design resistance $F_{b,Rd} = 840$ kN (as before)

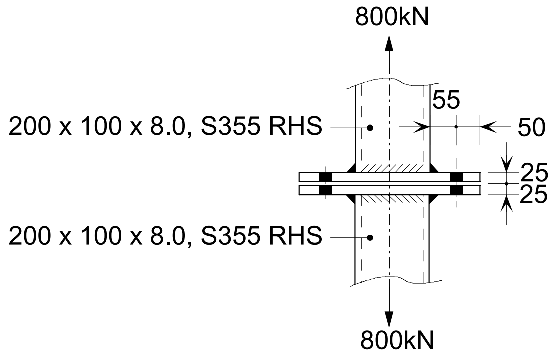
$F_{Ed,t} = 200$ kN < 840 kN $\therefore$ **O.K.**

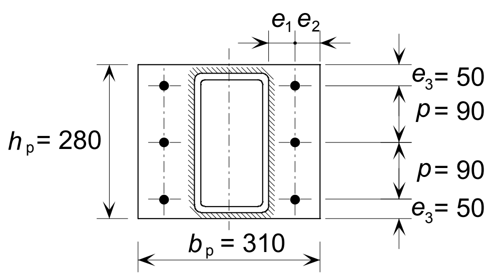
Therefore, the splice detail is adequate.

SCI Steel Knowledge CALCULATION SHEET BCSA	Job *Joints in Steel Construction – Simple Joints*		*Sheet 1 of 4*
	Title *Example 4 – Splices – RHS tension splice*		
	Client *Connections Group*		
	Calcs by *ENM*	Checked by *DGB*	Date *Sept 2011*

DESIGN EXAMPLE 4 – RHS TENSION SPLICE

Check the following connection for the design forces shown:





Design information:

Total number of bolts:	n = 6
End plates:	280 × 310 × 25
Bolts:	M24 8.8
Material:	S275 steel
Weld:	12 mm leg length fillet weld

Title Example 4 – Splices – RHS tension splice | Sheet 2 of 4

SUMMARY OF FULL DESIGN CHECKS FOR EXAMPLE 4

Sheet No.	CHECK	Resistance (kN)	Design force (kN)	Comments
3	***Check 1*** Recommended detailing practice	n/a	n/a	All recommendations adopted
3	***Check 2***	–	–	Not applicable
3	***Check 3*** Plate and bolt resistance	867	800	
4	***Check 4*** Bolt group	1218	800	
4	***Check 5*** Welds			Full strength weld

CONNECTION DESIGN USING RESISTANCE TABLES

200×100×8.0 RHS S355 Standard connection — Table G.29

Total number of bolts n = 6

End plate thickness t_p = 25 mm

Axial resistance N_{Rd} = 809 kN

N_{Ed} = 800 kN

800 kN < 809 kN — ∴ **O.K.**

Note: the resistance from Table G.29 is conservatively based on the thickest section in the range.

Title *Example 4 – Splices – RHS tension splice* *Sheet 3 of 4*

Connection design following the design procedures

Check 1: Recommended detailing practice

End plates: (≥ 12 mm < 26 mm) t_p = 25 mm ∴ **O.K.**

Bolts: ($n \leq \frac{2h}{p} + 2$ = 6.4) n = 6 ≥ 4 ∴ **O.K.**

Spacing: ($p \geq 2.5d$ = 48 mm)

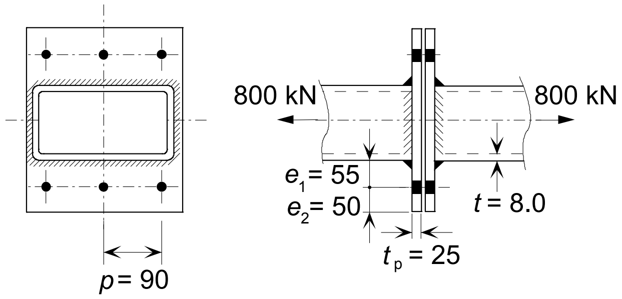
Edge distance ($e_2 \geq 1.2d_0$ = 31 mm) e_2 = 50 mm

Edge distance ($e_3 \geq 1.2d_0$ = 31 mm) e_3 = 50 mm ∴ **O.K.**

Check 2: Complete end plate yielding – Not applicable

(because end plate is within recommended detailing rules)

Check 3: Plate and bolt resistance



Basic requirement:

$F_{Ed} \leq F_{Rd}$

$$F_{Rd} = \frac{t_p^{\,2}(1+\delta\alpha)n}{K}$$

e_{eff} = min (e_2; $1.25e_1$) = min (50; 1.25 × 55) = 50 mm

$$\delta = 1 - \frac{d_0}{p} = 1 - \frac{26}{90} = 0.711$$

$$K = \frac{4\left(e_1 - (d/2) + t\right)10^3}{f_{y,plate} \times p} = \frac{4 \times \left(55 - (24/2) + 8.0\right) \times 10^3}{265 \times 90} = 8.55$$

$$\alpha = \left(\frac{KF_{t,Rd}}{t_p^{\,2}} - 1\right)\left(\frac{e_{eff} + (d/2)}{\delta\left(e_{eff} + e_1 + t\right)}\right) = \left(\frac{8.55 \times 203}{25^2} - 1\right)\left(\frac{50 + (24/2)}{0.711 \times (50 + 55 + 8.0)}\right) = 1.374$$

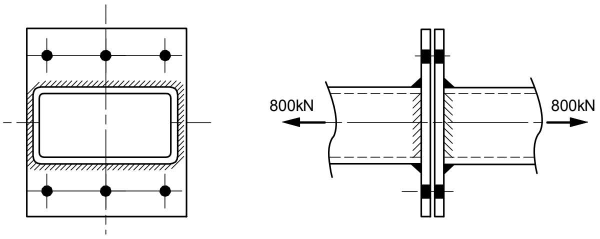
$$\therefore F_{Rd} = \frac{25^2 \times (1 + 0.711 \times 1.374) \times 6}{8.55} = 867 \text{ kN}$$

∴ F_{Ed} = 800 kN < 867 kN ∴ **O.K.**

Note that the resistances in Table G.29 conservatively assume t = 16 mm

Title *Example 4 – Splices – RHS tension splice* *Sheet 4 of 4*

Check 4: Bolt group



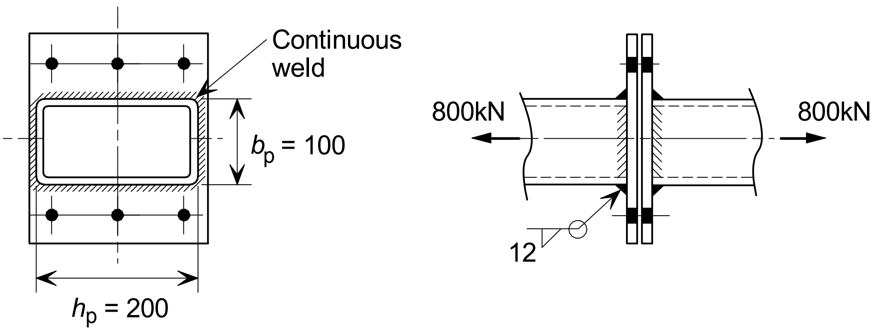
Basic requirement:

$F_{\mathrm{Ed}} \leq nF_{\mathrm{t,Rd}}$

$$nF_{\mathrm{t,Rd}} = n \times \frac{k_2 f_{\mathrm{ub}} A_{\mathrm{s}}}{\gamma_{\mathrm{M2}}} = 6 \times \frac{0.9 \times 800 \times 353}{1.25} \times 10^{-3} = 1218 \text{ kN}$$

$\therefore F_{\mathrm{Ed}} = 800$ kN < 1218 kN — $\therefore$ **O.K.**

Check 5: Welds



Basic requirement:

(1) $t \leq a$ (continuous full strength weld)

or

(2) $N_{\mathrm{Ed}} \leq 2h\,a \times 1.225 \times \dfrac{f_{\mathrm{u}}}{\sqrt{3}\beta_{\mathrm{w}}\gamma_{\mathrm{M2}}}$

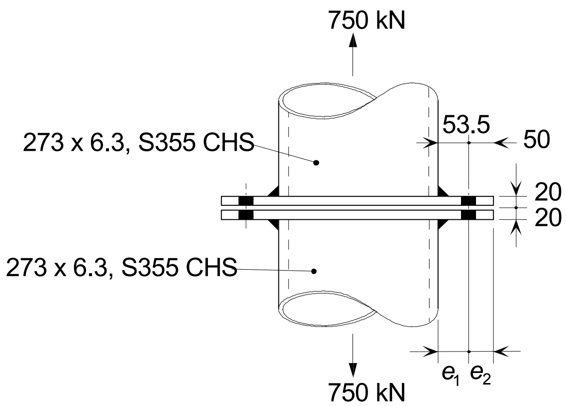
The weld throat is: $a = 0.7s = 0.7 \times 12 = 8.4$ mm

$t = 8.0$ mm ≤ 8.4 mm — $\therefore$ **O.K.**

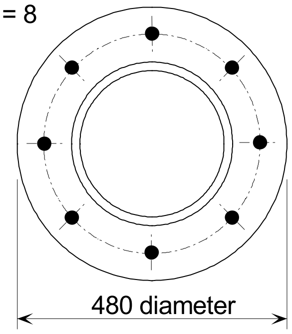
Steel Knowledge CALCULATION SHEET BCSA	Job *Joints in Steel Construction – Simple Joints*		Sheet *1* of *5*
	Title *Example 5 – Splices – CHS tension splice*		
	Client *Connections Group*		
	Calcs by *ENM*	Checked by *DGB*	Date *Sept 2011*

DESIGN EXAMPLE 5 – CHS TENSION SPLICE

Check the following connection for the design forces shown:



Total number of bolts n= 8



Design information:

Total number of bolts	n	= 8
End plates	t_p	= 20 mm
Bolts	M24, 8.8	
Material	S275 steel	
Weld	10 mm leg length fillet weld	

Title Example 5 – Splices – CHS tension splice *Sheet 2 of 5*

SUMMARY OF FULL DESIGN CHECKS FOR EXAMPLE 5

Sheet No	CHECK	Resistance (kN)	Design force (kN)	Comments
3	***Check 1*** Recommended detailing practice	n/a	n/a	All recommendations adopted
3	***Check 2*** Plate resistance	1030	750	
4	***Check 3*** Plate and bolt resistance	1061	750	
4	***Check 4*** Bolt group	1624	750	
5	***Check 5*** Weld			Full strength welds

CONNECTION DESIGN USING RESISTANCE TABLES

Table G.27

273 × 6.3 CHS Grade S355 Standard connection

Total number of bolts, n = 8

End plate thickness t_p = 20 mm

Axial resistance N_{Rd} = 943 kN

Applied force N_{Ed} = 750 kN

750 kN < 943 kN

∴ **O.K.**

Note: the resistance from Table G.27 is conservatively based on the thickest section in the range.

Title *Example 5 – Splices – CHS tension splice* *Sheet 3 of 5*

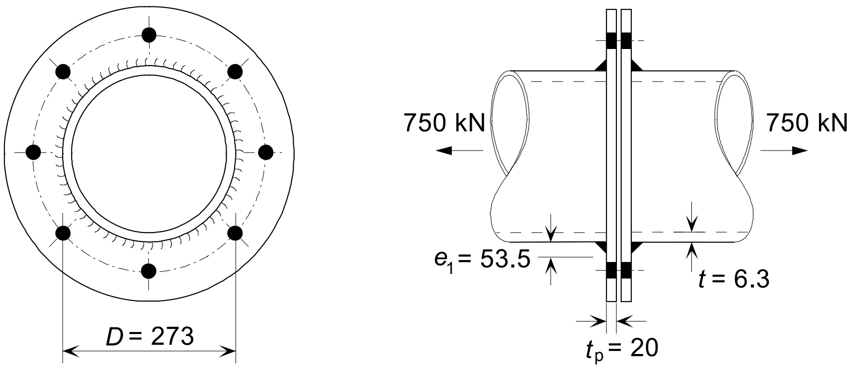
Check 1: Recommended detailing practice

End plates: ($t_p \geq$ 12 mm) t_p = 20 mm
($t_p <$ 26 mm) ∴ **O.K.**

Spacing: ($p \geq 2.5d = 60$) p = 380×π / 8 = 149 mm ∴ **O.K.**
($p < 10d = 240$)

Edge distance: ($e_2 \geq 1.2d_0$ = 31 mm) e_2 = 50 mm ∴ **O.K.**

Check 2: Plate resistance



Basic requirement:

$$F_{Ed} \leq \frac{t_p{}^2 f_{y,plate}\, \pi\, f_3}{2\gamma_{M0}}$$

$$f_3 = \frac{1}{2k_1}\left(k_3 + \left(k_3{}^2 - 4k_1\right)^{0.5}\right)$$

$$k_1 = \ln\left(\frac{r_2}{r_3}\right) \quad \text{(ln = natural logarithm)}$$

$$r_2 = \frac{D}{2} + e_1 = \frac{273}{2} + 53.5 = 190 \text{ mm}$$

$$r_3 = \frac{D - t_w}{2} = \frac{273 - 6.3}{2} = 133.4 \text{ mm}$$

$$k_1 = \ln\left(\frac{190}{133.4}\right) = 0.354$$

$$k_3 = k_1 + 2 = 0.354 + 2 = 2.354$$

$$\therefore f_3 = \frac{1}{2 \times 0.354}\left(2.354 + \left(2.354^2 - 4 \times 0.354\right)^{0.5}\right) = 6.19$$

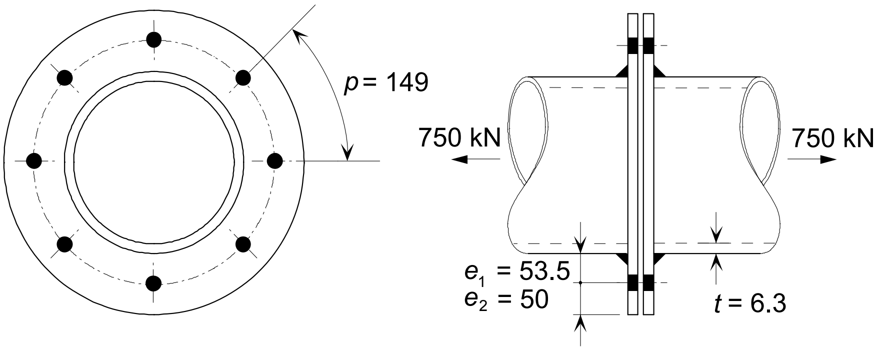
$$\therefore \text{Plate resistance} = \frac{20^2 \times 265 \times \pi \times 6.19}{2 \times 1.0} \times 10^{-3} = 1030 \text{ kN}$$

∴ F_{Ed} = 750 kN ≤ 1030 kN ∴ **O.K.**

Note that the resistance in Table G.27 conservatively assume t = 16 mm

Title Example 5 – Splices – CHS tension splice	Sheet 4 of 5

Check 3: Plate and bolt resistance



Basic requirement:

$$F_{Ed} \le \frac{nF_{t,Rd}}{\left(1-\frac{1}{f_3}+\frac{1}{f_3 \ln(r_1/r_2)}\right)\gamma_{M0}}$$

$F_{t,Rd}$ = 203 kN — Check 4

f_3 = 6.19 — Check 2

$r_1 = \frac{D}{2} + e_1 + e_{eff}$

e_{eff} = min (e_2 ; 1.25e_1) = min (50 ; 1.25 × 53.5) = 50 mm

$r_1 = \frac{273}{2} + 53.5 + 50$ = 240 mm

r_2 = 190 mm — Check 2

$$\therefore \text{Bolt group resistance} = \frac{8 \times 203}{1-\frac{1}{6.19}+\frac{1}{6.19 \times \ln\left(\frac{240}{190}\right)}} = 1061 \text{ kN}$$

∴ F_{Ed} = 750 kN < 1061 kN — **∴ O.K.**

Check 4: Bolt group

Basic requirement:

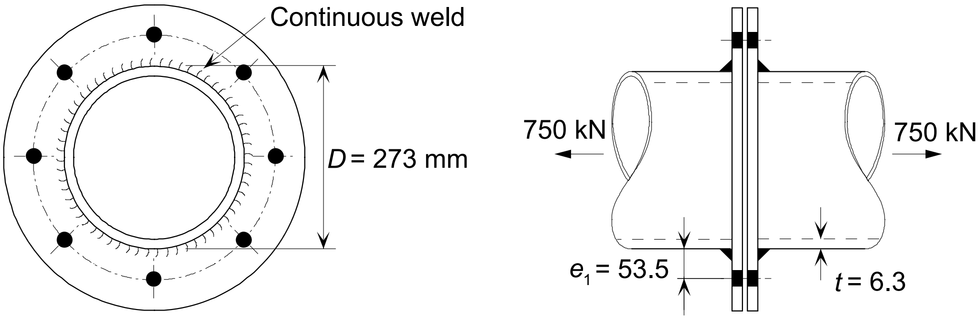
$F_{Ed} \le nF_{t,Rd}$

$$nF_{t,Rd} = n\frac{k_2 f_{ub} A_s}{\gamma_{M2}} = 8 \times \frac{0.9 \times 800 \times 353}{1.25} \times 10^{-3} = 1624 \text{ kN}$$

∴ F_{Ed} = 750 kN < 1624 kN — **∴ O.K.**

Title	*Example 5 – Splices – CHS tension splice*	*Sheet 5 of 5*

Check 5: Welds



Basic requirement:

(1) $t \leq a$ (continuous full strength weld)

or

(2) $N_{Ed} \leq \pi D\, a \dfrac{f_u}{\sqrt{3}\beta_w \gamma_{M2}}$

The weld throat is: $a = 0.7s = 0.7 \times 10 \text{ mm} = 7 \text{ mm}$

$t = 6.3 \text{ mm} \leq 7 \text{ mm}$ ∴ **O.K.**

7 COLUMN BASES

7.1 INTRODUCTION

Typical column bases, as shown in Figure 7.1, 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. High strength grout is poured into the space below the plate (see Figure 7.2).

Such column bases are usually assumed to be subject to axial compression and shear only. However, uplift may be a design case 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 resistance of the foundation.

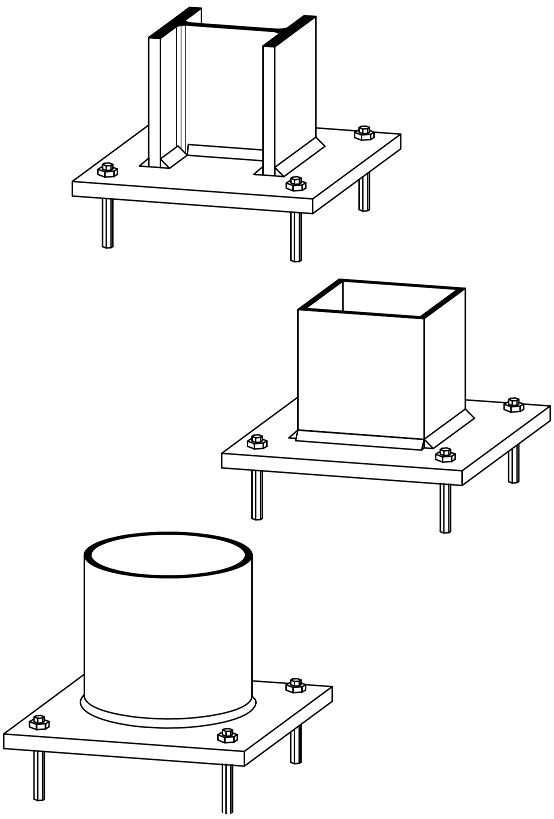


Figure 7.1 Typical column bases

Holding down systems are designed to stabilise the column during construction, and resist any uplift in braced bays. In some cases it is assumed that horizontal shear is also carried by the holding down bolts.

A simple rectangular or square baseplate is almost universally used for columns in simple construction.

Column bases are generally designed to transfer the force from the column to the baseplate in direct bearing.

The way in which horizontal shear forces are transferred to the foundation is not well researched. Some designers check the resistance of the holding down bolts, and ensure that the holding down bolts are adequately grouted. This practice has been successfully followed for portal frame bases, which carry a significant shear.

Braced bays may have relatively high shear forces. Designers may opt to provide a shear stub welded to the underside of the baseplate, though the recess may complicate the casting of the foundation, and special attention must be paid to the grouting operation. Design methods are available that cover this type of detail[35][36].

Shear between the column end and the base plate can be transmitted by friction or by nominal welds between the column and the base plate. Welds may be provided to the web only, or around parts of the profile – it will generally be found that the weld resistance is more than adequate for small shear forces.

7.2 PRACTICAL CONSIDERATIONS

Column

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. Direct bearing does not necessitate the machining or milling of the column ends. Tolerances for bearing surfaces can be found in BS 1090-2[32] or the National Structural Steelwork Specification[10].

Base plates

Base plates will usually be flame cut or sawn from S275 or S355 plate. 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 and to ensure the column is stable in any temporary condition.

Fillet welds are generally provided, 6 mm or 8 mm leg length, usually along the outside of the flanges and for a short distance either side of the web. Full profile welds will usually only be used if additional resistance is needed during erection or as an anti-corrosion measure.

Holding down bolts

Holding down bolts are normally manufactured in accordance with BS 7419[41] which covers:

a) bolts with square head and neck, and
b) 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 100 mm plus the bolt diameter.

Holding down bolts are usually property class 8.8 (property class 4.6 is not commonly used). It is bad practice to specify different bolt strengths on one site, unless the bolts are physically different in length or diameter, as confusion can easily lead to the incorrect bolt being cast into the foundation. M20 bolts are often used, although M24 bolts are recommended for bases up to 50 mm thick, increasing to M36 for plates over 50 mm thick.

Although bolts are normally used in the non-coated condition, they can be supplied:

a) electroplated to BS EN ISO 4042[37]
b) galvanized to BS EN ISO 10684 [38]
c) sherardized to BS 7371-8[39]

Hole sizes and washers

Clearance holes in the base plate should be 6 mm larger than the bolt diameter, to allow for adjustment; for bases thicker than 60 mm, this figure may need to be increased.

Washers to BS EN ISO 7091[40],[41] are needed under the nuts, or alternatively washers can be made from plate.

Location tubes

Normally the bolts are set into location tubes (usually wire mesh) or cones (usually polystyrene or cardboard). The diameter of the tube or the tops of the cones should be at least 100 mm or 3 times the diameter of the bolt, whichever is greater. The threaded portion of the bolts should be protected during the concreting operation. As the concrete cures, the bolts should be moved so that they can be freely adjusted as the steel is erected.

Bolts are occasionally cast solidly into the concrete. However, this is not recommended as a general practice as the bolts must be located with precision and there is no opportunity for subsequent adjustment.

Base packs

Columns are normally erected on central steel levelling packs that are left permanently in position. Steel wedges placed around the edges of the base plate are also necessary to ensure stability during erection.

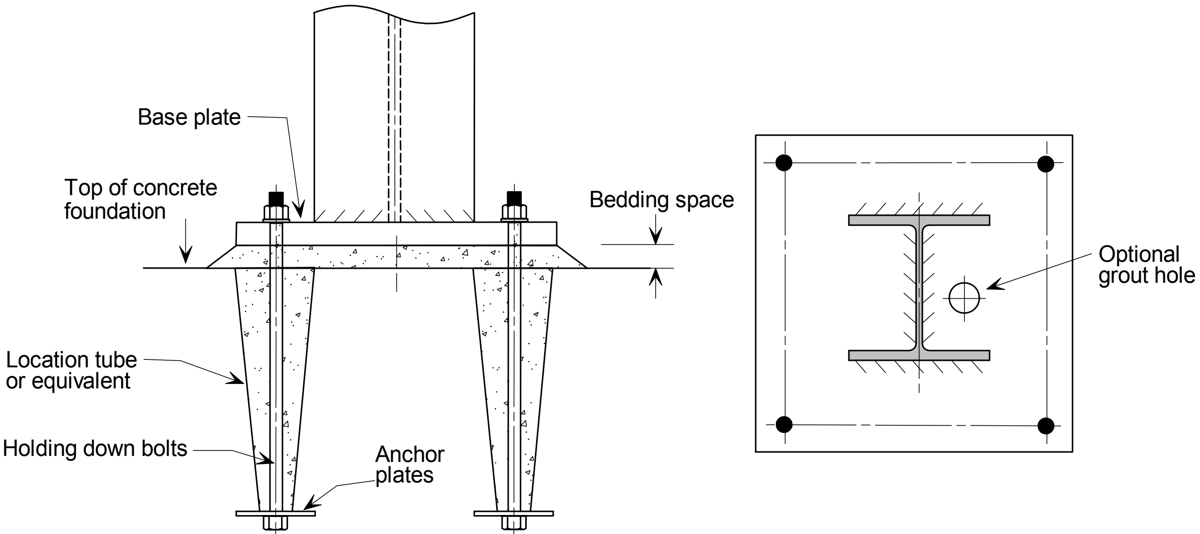


Figure 7.2 Column base holding down bolts

Concrete strength

Typical concrete strengths are shown in Table 7.1, taken from BS EN 1992-1-1.

The design compressive strength $f_{cd} = \dfrac{\alpha_{cc} f_{ck}}{\gamma_c}$, in which α_{cc} = 0.85 and γ_c = 1.5. Thus $f_{cd} = 0.56 f_{ck}$. It may be assumed that, for initial design, the design bearing strength f_{jd} is equal to the design compressive strength, although the design bearing strength also depends on the physical geometry of the foundation.

Table 7.1 Concrete strengths

Concrete class	Cylinder strength (N/mm^2)	Cube strength (N/mm^2)	Design compressive strength (N/mm^2)
	f_{ck}	$f_{ck,cube}$	f_{cd}
C25/30	25	30	14.2
C30/37	30	37	16.8
C35/45	30	45	17.0
C40/50	40	50	22.7

For design purposes, the lowest strength of either the grout or the concrete in the foundation should be used. In most situations, the design will be based on the strength of the concrete in the foundation, and non-shrink cementitious grout will be specified to be at least as strong. Other bedding materials may be specified.

Clearance under the baseplate

A space between the top of the foundation and the baseplate of between 25 mm and 50 mm is the normal allowance when using grout. 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.

In slab bases of size 700 mm × 700 mm or larger, 50 mm diameter grout holes should be provided to allow trapped air to escape and also for inspection. A hole should be provided for each 0.5 m^2 of base area. If it is expected to place grout through these holes then the diameter should be increased to 100 mm.

7.3 RECOMMENDED GEOMETRY

No specific detailing requirements for column bases can be provided, although in practice there are three main considerations:

a) The plate dimensions must be sufficient to spread the design loads to the foundation and to accommodate the holding down bolts;

b) 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.

c) The setting out dimensions for the holding down bolts should be on a regular, simple geometry

In view of this, normal practice is for the base to be at least 100 mm 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.

Baseplate dimensions are generally rounded up to the nearest 50 mm.

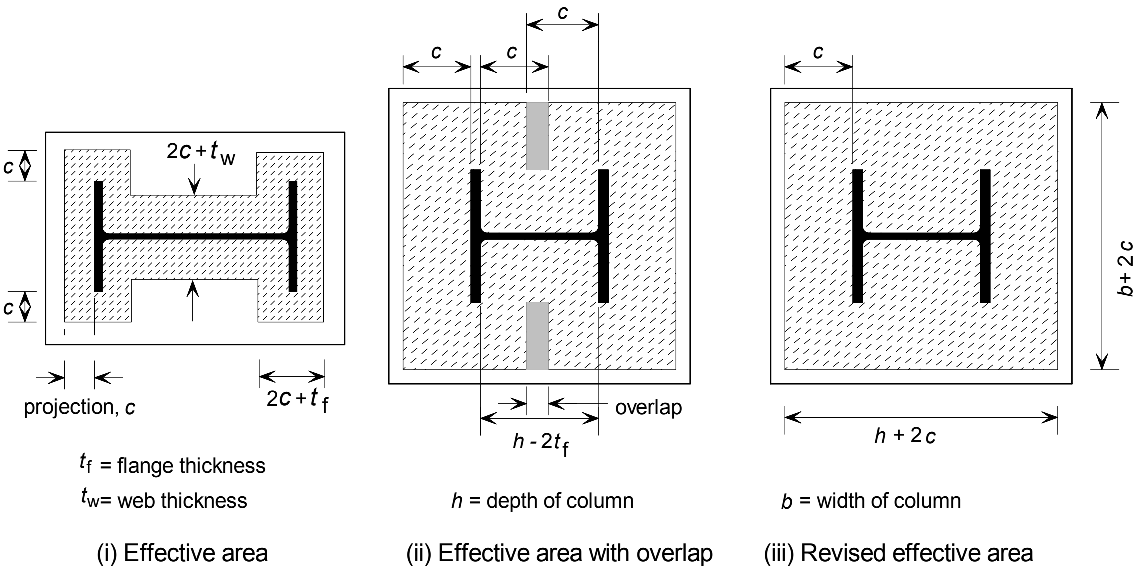


Figure 7.3 Calculated effective area for a rolled section

7.4 DESIGN

The design procedure for column bases is taken from BS EN 1993-1-8 and follows an effective area approach. The procedure covers the design of bases under axial compression only.

The process is to:

1. Find the required area, A_{req}.
2. Determine the effective area, A_{eff}, in terms of the projection width from the steel profile, *c*.
3. By equating A_{req} and A_{eff}, calculate *c.*
4. Calculate the required plate thickness, assuming that the projection width *c* is a uniformly loaded cantilever.

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. Guidance can be found in *Joints in Steel Construction: Moment-resisting Joints*[19].

Effective area 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 a constant width *c* either side of each flange and web, as illustrated in Figure 7.3 with respect to a rolled I section.

The projection width *c*, shown in Figure 7.3(i) is the minimum that is needed to ensure that the base pressure does not exceed the design bearing strength.

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 Figure 7.3(ii), i.e.

$c > (h–2t_f)/2$

The overlapping area cannot be double counted, so the effective area must be recalculated on the basis shown in Figure 7.3(iii).

For hollow section columns, the design procedure is similar and is illustrated for RHS and CHS columns in Figure 7.4. If the internal projection overlaps in the centre of the section, a readjusted effective area must be recalculated in a similar manner to that for the open section.

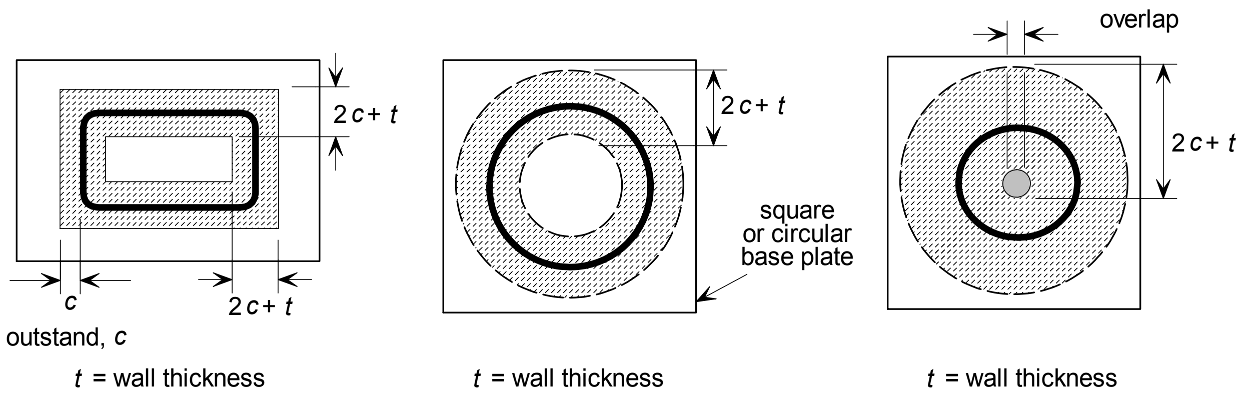


Figure 7.4 Calculated effective area for RHS and CHS sections

Although the shaded area represents the size of the base plate theoretically required, the overall size of the plate can be made larger, to utilise rounded dimensions and to accommodate the holding down bolts.

Worked examples

Four design examples are provided to illustrate the design checks for the effective area method.

Base plate resistance tables

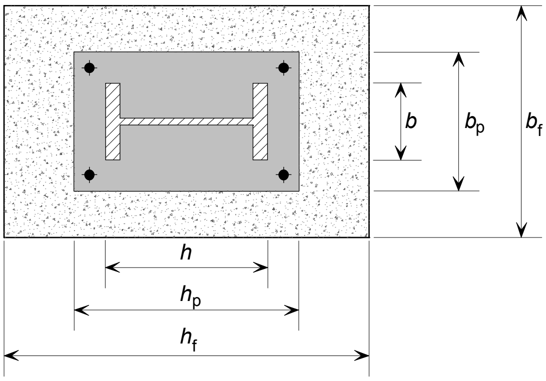
Four sets of resistance tables for S275 base plates are provided in the yellow pages (Tables G.32 to G.35). These cover UKC columns, CHS columns, SHS and RHS columns

7.5 DESIGN PROCEDURES

The design models shown in Figure 7.3 are appropriate for I sections, and Figure 7.4 are appropriate for RHS and CHS sections, Detail design checks are as follows:

Check 1	–	Required area
Check 2	–	Effective area
Check 3	–	Plate thickness
Check 4	–	Welds

CHECK 1	**Required area**



Basic requirement:

$A_p \geq A_{req}$

A_p is the area of base plate

$= h_p b_p$ for rectangular plates

$= \dfrac{\pi d_p^{\,2}}{4}$ for circular plates

A_{req} is the required area of base plate

$= \dfrac{N_{Ed}}{f_{jd}}$

where:

$f_{jd} = \beta_j \, \alpha \, f_{cd}$

β_j may be taken as $\dfrac{2}{3}$ (see note 1)

α is a coefficient which accounts for diffusion of the concentrated force within the foundation. α depends on the foundation depth and the distance between the plate and the edge(s) of the foundation.

Where the foundation's dimensions are unknown, but will be orthodox (i.e. not narrow or shallow) it is reasonable to assume $\alpha = 1.5$, and hence

$f_{jd} = f_{cd} = 0.85 \dfrac{f_{ck}}{\gamma_c}$.(see note 2)

$f_{cd} = \alpha_{cc} \dfrac{f_{ck}}{\gamma_c}$

α_{cc} = 0.85(National Annex to BS EN 1992-1-1)

γ_c is the material factor for concrete (γ_c = 1.5 as given in the UK NA)

Concrete class	**C20/25**	**C25/30**	**C30/37**	**C35/45**
Cylinder strength, f_{ck} (N/mm^2)	20	25	30	35
Cube strength, $f_{ck,cube}$, (N/mm^2)	25	30	37	45

Notes:

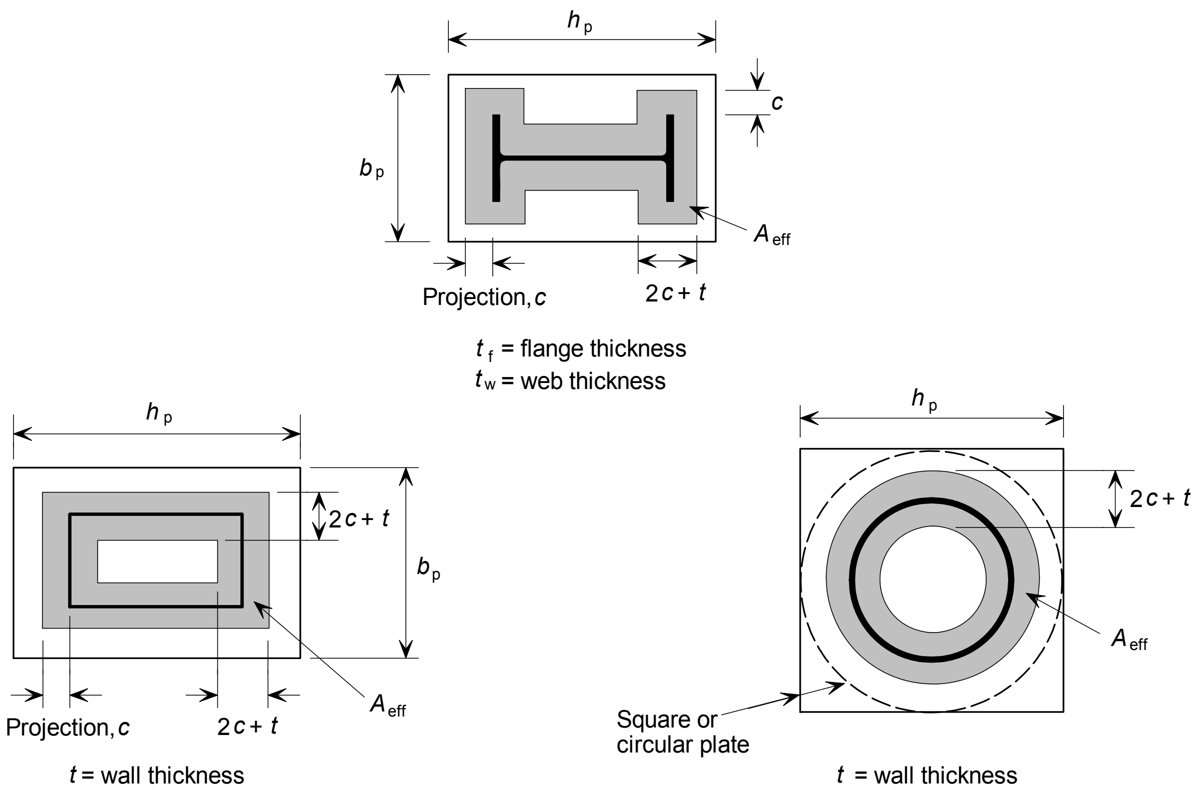
(1) In accordance with clause 6.2.5(7) of BS EN 1993-1-8, the use of β_j = 2/3 requires that:

a. The grout has a compressive strength at least equal to 0.2 f_{cd}, and:
   i. the grout is less than 50 mm thickness
   ii. the thickness of grout is less than 0.2 h_p and 0.2 b_p

b. The grout has a compressive strength at least equal to f_{cd} if over 50 mm thick

(2) If α = 1.5 is assumed, the foundation depth should be at least 50% of the larger base plate dimension, and all edge distances between the plate and the edge of the foundation should be at least 25% of the larger base plate dimension. If precision is required, $\alpha = \sqrt{A_{c1}/A_{c0}}$ as given in BS EN 1992-1-1 clause 6.7

CHECK 2	Effective area



Basic requirement:

$A_{req} \quad \leq \quad A_{eff}$

Provided there is no overlap, *c* may be calculated from the following equations:

For UKB, UKC $\quad A_{eff} = 4c^2 + P_{col}\,c + A_{col}$

For SHS or RHS column $\quad A_{eff} = P_{col}\,(t + 2c)$

For CHS column $\quad A_{eff} = \pi\,(d - t)\,(t + 2c)$

If there is an overlap, *c* may be calculated from the following equations:

For UKB, UKC, when $c \geq \dfrac{h - 2t_f}{2}$:

$$A_{eff} = 4c^2 + 2(h + b)c + h\,b$$

For SHS and RHS column, when $c \geq \dfrac{b - 2t}{2}$:

$$A_{eff} = 4c^2 + 2(h + b)c + h\,b$$

For CHS column, when $c \geq \dfrac{d - 2t}{2}$:

$$A_{eff} = 0.25\,\pi\,(d + 2c)^2$$

where:

A_{req} is the required area of base plate (from Check 1)

A_{col} is the cross sectional area of the column

h is the depth of the column

b is the width of the column

d is the diameter of the CHS column

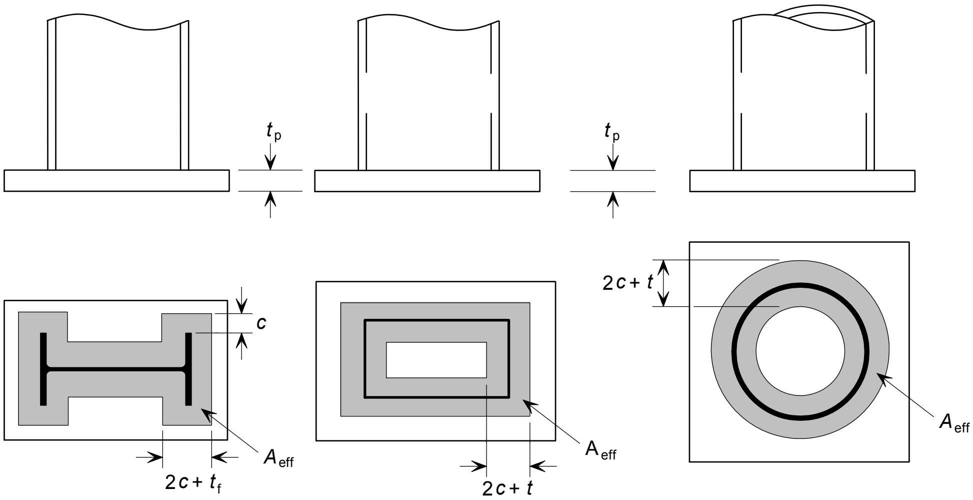
For a universal beam or column section:

P_{col} is the column perimeter

For a square or rectangular hollow section:

P_{col} is the perimeter of the centre line of the section wall of the hollow section

CHECK 3	Plate thickness



Basic requirement:

$t_p \geq t_{p,min}$

$$t_{p,min} = c\sqrt{\frac{3 f_{jd} \gamma_{M0}}{f_{yp}}}$$

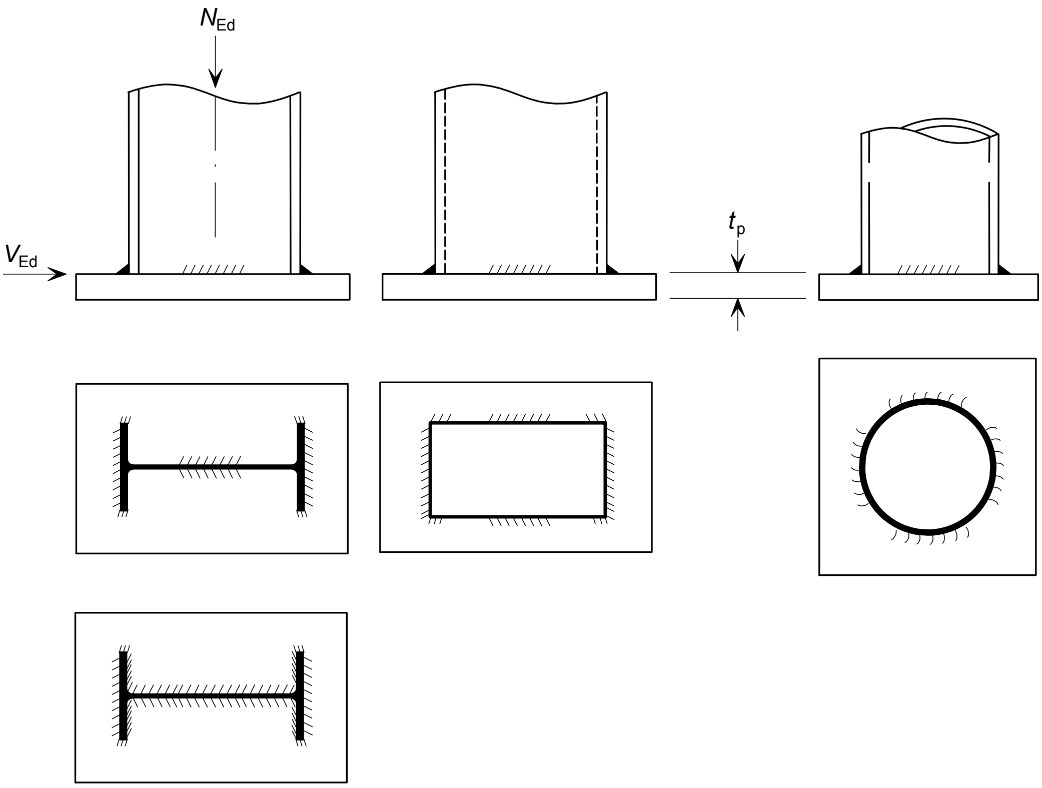
where:

f_{yp} is the yield strength of the base plate

f_{jd} is from Check 1

c is from Check 2

CHECK 4	**Welds**



Basic requirement:

For shear:

$V_{Ed} \leq F_{w,Rd}\, \ell_{w,eff}$

$F_{w,Rd}$ is the resistance of fillet weld per unit length

$= f_{vw,d}a$

$$f_{vw,d} = \frac{f_u / \sqrt{3}}{\beta_w \gamma_{M2}}$$

where:

a is the weld throat

$= 0.7s$

s is the weld leg length

β_w = 0.85 for S275 steel

= 0.9 for S355 steel

γ_{M2} is the partial factor for resistance of welds (γ_{M2} = 1.25 as given in the National Annex)

$\ell_{w,eff}$ is the total effective length of the welds in direction of shear

f_u is the ultimate strength of the weaker part joined

= 410 N/mm^2 for S275 steel

= 470 N/mm^2 for S355 steel

7.6 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 UKC Column base, using the effective area method, where the column is in direct bearing but welds must transfer 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 force, where the effective area calculation produces overlap and a recalculation of the outstand *c* has to be made.

Example 3

The design of an RHS column base, using the effective area method, where the column is in direct bearing but welds must resist the shear force at the base.

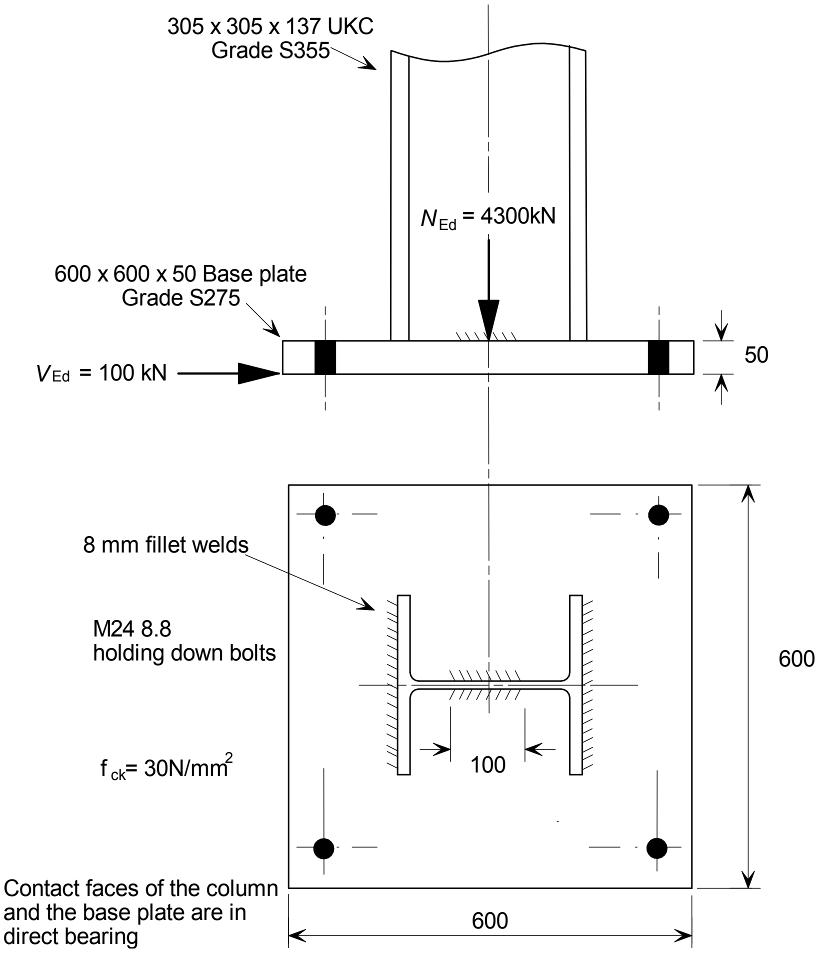
Example 4

The design of a CHS column base, using the effective area method, where the column is in direct bearing but welds must transfer the shear force at the base.

CALCULATION SHEET	Job	*Joints in Steel Construction – Simple Joints*	*Sheet 1 of 3*
	Title	*Example 1 – Column Base – I-section column – No overlap*	
	Client	*Connections Group*	
	Calcs by *ENM*	Checked by *CZT*	Date *Sept 2011*

DESIGN EXAMPLE 1

Check the column base for the design forces shown.



BASE DESIGN USING RESISTANCE TABLES

For a 600 × 600 × 50 base plate with C30/37 concrete, — Table G.32

Resistance given in the tables is 4461 kN > 4300 kN

Therefore the base is adequate

Title Example 1 – Column Base – *I*-section column – No overlap | Sheet 2 of 3

CONNECTION DESIGN FOLLOWING THE DESIGN PROCEDURES

Check 1: Required area

Basic requirement: $A_p \geq A_{req}$

Area of base plate: $A_p = h_p \times b_p = 600 \times 600 = 360000 \text{ mm}^2$

Design compressive strength of the concrete:

$$f_{cd} = \alpha_{cc} \frac{f_{ck}}{\gamma_c} = 0.85 \frac{30}{1.5} = 17 \text{ N/mm}^2$$

$f_{jd} = \alpha \beta_j f_{cd}$

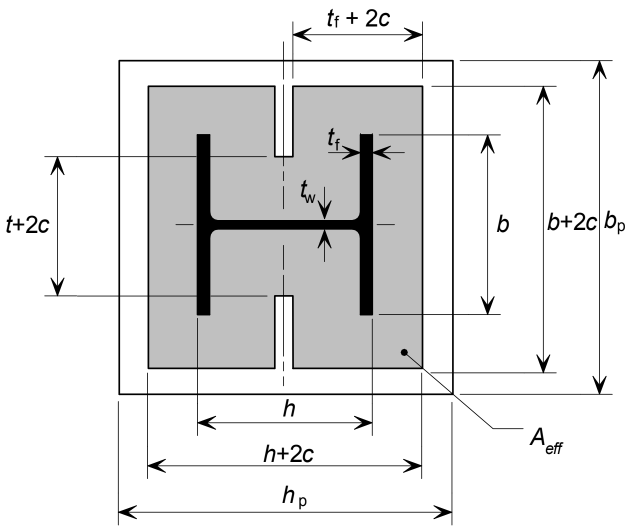
Assuming $\alpha = 1.5$, $\beta_j = 2/3$

$f_{jd} = 1.5 \times 2/3 \times 17 = 17 \text{ N/mm}^2$

Area required: $$A_{req} = \frac{N_{Ed}}{f_{jd}} = \frac{4300 \times 10^3}{17} = 253000 \text{ mm}^2$$

$A_p = 360000 \text{ mm}^2 > 253000 \text{ mm}^2$ **∴ O.K.**

Check 2: Effective area



Basic requirement: $A_{eff} = A_{req}$

To calculate the effective area, assume first that there is no overlap.

$A_{eff} = 4c^2 + c\,P_{col} + A_{col}$

Column perimeter $P_{col} = 1820$ mm

Area of column $A_{col} = 17400 \text{ mm}^2$

$A_{eff} = 4c^2 + 1820c + 17400 = 253000 = A_{req}$

∴ $c = 105$ mm

To check that there is no overlap c has to be less than half the depth between flanges:

$$\frac{h - 2t_f}{2} = \frac{320.5 - 2 \times 21.7}{2} = 138.6 \text{ mm} > 105 \text{ mm}$$

Therefore the assumption that there is no overlap is correct.

To check that the effective area fits on the base plate:

$h + 2c = 320.5 + 2 \times 105 = 530.5 \text{ mm} < 600 \text{ mm}$

$b + 2c = 309.2 + 2 \times 105 = 519.2 \text{ mm} < 600 \text{ mm}$

Therefore the calculated value of c is valid.

Title Example 1 – Column Base – I-section column – No overlap	Sheet 3 of 3

Check 3: Plate thickness

$t_{p,min} \quad = c\sqrt{\dfrac{3 f_{jd}\gamma_{M0}}{f_{yp}}}$

Yield strength of the 50 mm plate, f_{yp} = 255 N/mm²

$t_{p,min} \quad = 105\sqrt{\dfrac{3\times 17\times 1.0}{255}} \qquad = 47$ mm

$\therefore\ t_p \quad = 50$ mm $\quad > \quad 47$ mm — ∴ **O.K.**

Check 4: Welds

Basic requirement: $\quad V_{Ed} \ \leq \ F_{w,Rd}\,\ell_{w,eff}$

$F_{w,Rd} \quad = f_{vw,d}\,a = \dfrac{f_u / \sqrt{3}}{\beta_w \gamma_{M2}}\,a \quad = \dfrac{410/\sqrt{3}}{0.85\times 1.25}\times 0.7\times 8 \quad = 1248$ N/mm

$\ell_{w,eff} \quad = 2\,(\ell - 2s) \quad = 2\times(100 - 2\times 8) \quad = 168$ mm

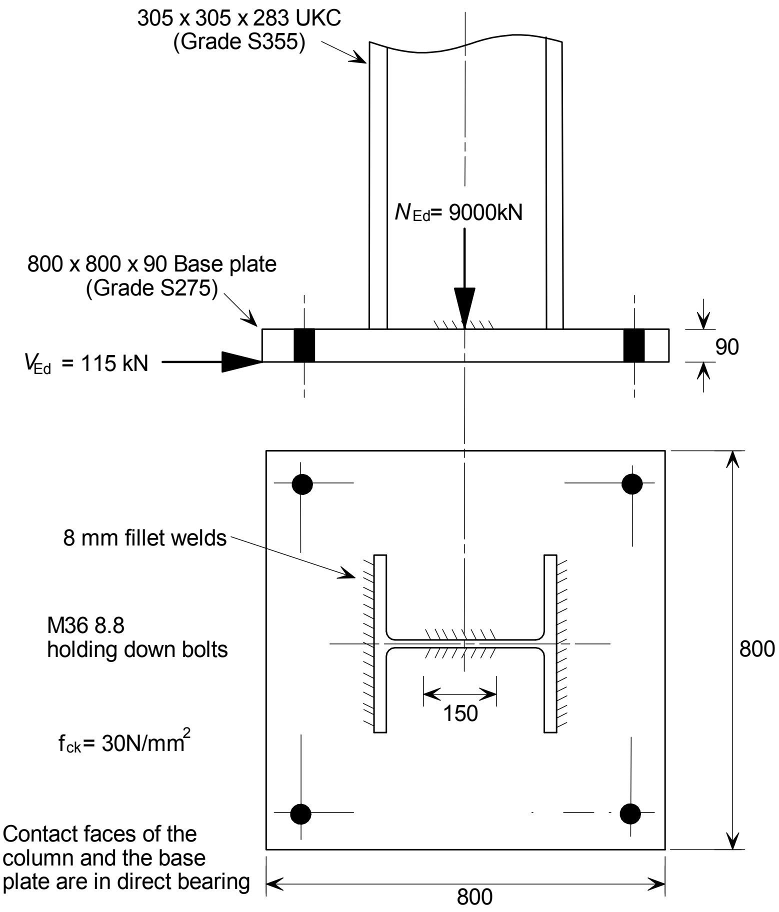
$F_{w,Rd}\times\ell_{w,eff} \quad = 1248\times 168\times 10^{-3} = 210$ kN

$V_{Ed} \quad = 100$ kN $\quad < \quad 210$ kN — ∴ **O.K.**

SCI Steel Knowledge CALCULATION SHEET BCSA	Job *Joints in Steel Construction – Simple Joints*		*Sheet 1 of 3*
	Title *Example 2 – Column base – I-section column – With overlap*		
	Client *Connections Group*		
	Calcs by *ENM*	Checked by *DGB*	Date *Sept 2011*

DESIGN EXAMPLE 2

Check the column base for the design forces shown.



BASE DESIGN USING RESISTANCE TABLES

For a 800 × 800 × 90 baseplate with C30/37 concrete, — Table G.32

Resistance given in the tables is 8164 kN, < 9000 kN, unsatisfactory.

Although the resistance tables indicate that the base is inadequate, the table values are calculated using the smallest section in any serial size and are therefore conservative for heavier columns in the range. A higher resistance may be obtained for a heavier column if the design is completed manually, as demonstrated in the following calculations.

Title *Example 2 – Column base – I-section column – With overlap* *Sheet 2 of 3*

CONNECTION DESIGN FOLLOWING THE DESIGN PROCEDURES

Check 1: Required area

Basic requirement: $A_p \geq A_{req}$

Area of base plate: $A_p = h_p \times b_p = 800 \times 800 = 640000 \text{ mm}^2$

Design compressive strength of the concrete: $f_{cd} = \alpha_{cc} \dfrac{f_{ck}}{\gamma_{M0}} = 0.85 \times \dfrac{30}{1.5} = 17 \text{ N/mm}^2$

$f_{jd} = \alpha\beta_j f_{cd}$

Assuming $\alpha = 1.5$, $\beta_j = 2/3$

$f_{jd} = 1.5 \times 2/3 \times 17 = 17 \text{ N/mm}^2$

Area required: $A_{req} = \dfrac{N_{Ed}}{f_{jd}} = \dfrac{9000 \times 10^3}{17} = 529000 \text{ mm}^2$

$A_p = 640000 \text{ mm}^2 > 529000 \text{ mm}^2$ ∴ **O.K.**

Check 2: Effective area

Basic requirement: $A_{eff} = A_{req}$

To calculate the effective area, assume first that there is no overlap.

$A_{eff} = 4c^2 + cP_{col} + A_{col}$

Column perimeter $P_{col} = 1940$ mm

Area of column $A_{col} = 36000 \text{ mm}^2$

$A_{eff} = 4c^2 + 1940c + 36000 = 529000 \text{ mm}^2 = A_{req}$

∴ $c = 184$ mm

To ensure that there is no overlap c has to be less than half the depth between flanges:

$$\frac{h - 2t_f}{2} = \frac{365.3 - 2 \times 44.1}{2} = 138.6 \text{ mm} < 184 \text{ mm}$$

Therefore assumption of no overlap is incorrect.

Recalculate c on the basis of a revised effective area as shown in Figure 7.3(iii)

$A_{eff} = (h + 2c)(b + 2c) = 4c^2 + 2(h + b)c + hb$

Equating the effective area, A_{eff} to the required area A_{req} (from Check 1) gives:

$$4c^2 + 2 \times (365.3 + 322.2)c + 365.3 \times 322.2 = 529000$$

∴ $c = 192$ mm

Also check that effective area fits on the base plate

$h + 2c = 365.3 + (2 \times 192) = 749.3 \text{ mm} < h_p = 800 \text{ mm}$

$b + 2c = 322.3 + (2 \times 192) = 706.3 \text{ mm} < b_p = 800 \text{ mm}$

Hence the new calculated value of c is valid

∴ $c = 192$ mm

Check 3: Plate thickness

$$t_{p,min} = c\sqrt{\frac{3 f_{jd} \gamma_{M0}}{f_{yp}}}$$

Yield strength of the 90 mm plate, $f_{y,p} = 235 \text{ N/mm}^2$

$$t_{p,min} = 192\sqrt{\frac{3 \times 17 \times 1.0}{235}} = 89 \text{ mm}$$

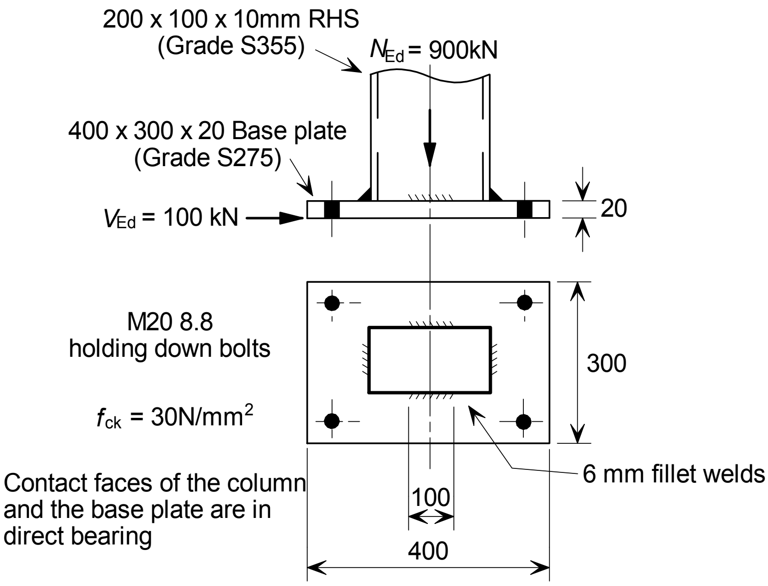
∴ $t_p = 90 \text{ mm} > 89 \text{ mm}$ ∴ **O.K.**

Title Example 2 – Column base – *I*-section column – With overlap	Sheet 3 of 3
Check 4: Welds	
Basic requirement: $V_{Ed} \leq F_{w,Rd}\,\ell_{w,eff}$	
$F_{w,Rd} = f_{vw,d}\,a = \dfrac{f_u/\sqrt{3}}{\beta_w \gamma_{M2}}a = \dfrac{410/\sqrt{3}}{0.85 \times 1.25} \times 0.7 \times 8 = 1248$ N/mm	
$\ell_{w,eff} = 2\,(\ell - 2s) = 2 \times (150 - 2 \times 8) = 268$ mm	
$F_{w,Rd}\,\ell_{w,eff} = 1248 \times 268 \times 10^{-3} = 334$ kN	
$\therefore\ V_{Ed} = 115$ kN $<$ 334 kN	$\therefore$ **O.K.**

Steel Knowledge **CALCULATION SHEET**	Job *Joints in Steel Construction – Simple Joints*		*Sheet 1 of 3*
	Title *Example 3 – Column Base – RHS*		
	Client *Connections Group*		
	Calcs by *ENM*	Checked by *DGB*	Date *Sept 2011*

DESIGN EXAMPLE 3

Check the RHS column base for the design forces shown.



BASE DESIGN USING RESISTANCE TABLES

For a 400 × 300 × 20 baseplate with C30/37 concrete, Table G.35

Resistance given in the tables is 1269 > 900 kN

Therefore the base is adequate

Title Example 3 – Column Base – RHS	Sheet 2 of 3

CONNECTION DESIGN FOLLOWING THE DESIGN PROCEDURES

Check 1: Required area

Basic requirement: $A_p \geq A_{req}$

Area of base plate: $A_p = h_p \times b_p = 400 \times 300 = 120000 \text{ mm}^2$

Design strength of the concrete: $f_{cd} = \alpha_{cc} \dfrac{f_{ck}}{\gamma_{M0}} = 0.85 \dfrac{30}{1.5} = 17 \text{ N/mm}^2$

$f_{jd} = \alpha \beta_j f_{cd}$

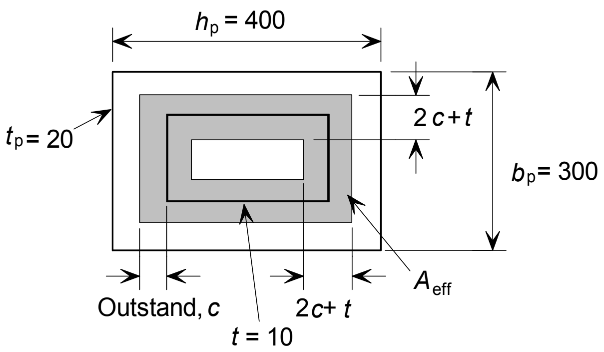
Assuming $\alpha = 1.5$, $\beta_j = 2/3$

$f_{jd} = 1.5 \times 2/3 \times 17 = 17 \text{ N/mm}^2$

Area required: $A_{req} = \dfrac{N_{Ed}}{f_{jd}} = \dfrac{900 \times 10^3}{17} = 52900 \text{ mm}^2$

$A_p = 120000 \text{ mm}^2 > 52900 \text{ mm}^2$ ∴ **O.K.**

Check 2: Effective area



Basic requirement: $A_{eff} = A_{req}$

To calculate the effective area, assume first that there is no overlap.

$A_{eff} = P_{col} (t + 2c)$

Mean wall perimeter:

$P_{col} = 2(h + b - 2t) = 2 \times (200 + 100 - 2 \times 10) = 560 \text{ mm}$

$A_{eff} = 560 \times (10 + 2c) = 52900 = A_{req}$

$5600 + 1120c = 52900$

$\therefore c = 42 \text{ mm}$

To ensure that there is no overlap c has to be less than half the width between flanges:

$\dfrac{b - 2t_f}{2} = \dfrac{100 - 2 \times 10}{2} = 40 \text{ mm} < 42 \text{ mm}$

Therefore assumption of no overlap is incorrect

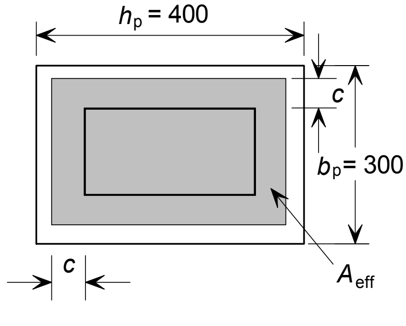
Recalculate c on the basis of a revised effective area

$A_{eff} = (h + 2c)(b + 2c) = 4c^2 + 2(h + b)c + hb$

Equating the effective area, A_{eff} to the required area A_{req} (from Check 1) gives:

$4c^2 + 2 \times (200 + 100)c + 200 \times 100 = 52900$

$\therefore c = 43 \text{ mm}$



Also check that effective area fits on the base plate:

$h_{fc} + 2c = 200 + (2 \times 43) = 286 \text{ mm} < h_p = 400 \text{ mm}$

$b_{fc} + 2c = 100 + (2 \times 43) = 186 \text{ mm} < b_p = 300 \text{ mm}$

Title Example 3 – Column Base – RHS *Sheet 3 of 3*

Hence the new calculated value of *c* is valid

$\therefore c = 43$ mm

Check 3: Plate thickness

$$t_{p,min} = c\sqrt{\frac{3 f_{jd} \gamma_{M0}}{f_{yp}}}$$

Yield strength of the 20 mm plate, $f_{yp} = 265$ N/mm^2

$$t_{p,min} = 43 \times \sqrt{\frac{3 \times 17 \times 1.0}{265}} = 19 \text{ mm}$$

$\therefore t_p = 20$ mm $>$ 19 mm $\therefore$ O.K.

Check 4: Welds

Basic requirement: $V_{Ed} \leq F_{w,Rd}\, \ell_{w,eff}$

$$F_{w,Rd} = f_{vw,d} a = \frac{f_u / \sqrt{3}}{\beta_w \gamma_{M2}} a = \frac{410/\sqrt{3}}{0.85 \times 1.25} \times 0.7 \times 6 = 936 \text{ N/mm}$$

$\ell_{w,eff} = 2(\ell - 2s) = 2 \times (100 - 2 \times 6) = 176$ mm

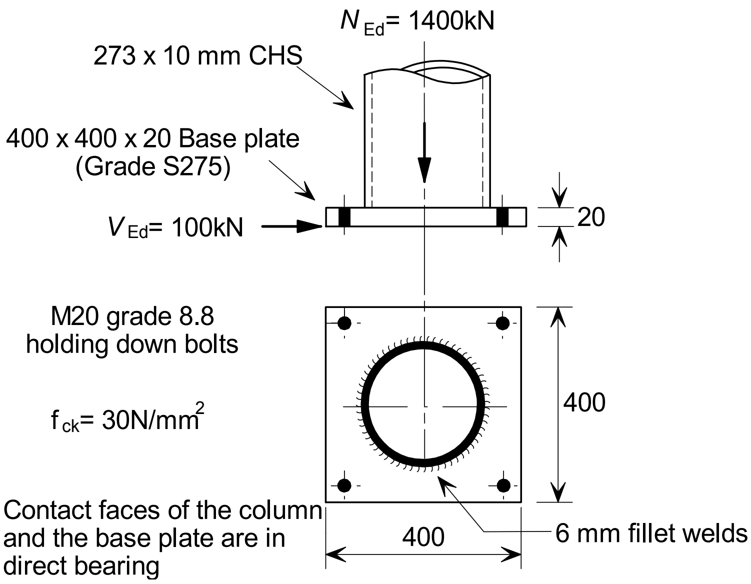
$F_{w,Rd}\, \ell_{w,eff} = 936 \times 176 \times 10^{-3} = 165$ kN

$V_{Ed} = 100$ kN $<$ 165 kN $\therefore$ O.K.

SCI Steel Knowledge CALCULATION SHEET BCSA	Job *Joints in Steel Construction – Simple Joints*		*Sheet 1 of 3*
	Title *Example 4 – Column Base – CHS*		
	Client *Connections Group*		
	Calcs by *ENM*	Checked by *DGB*	Date *Sept 2011*

DESIGN EXAMPLE 4

Check the CHS column base for the design forces shown.



BASE DESIGN USING RESISTANCE TABLES

For a 400 × 400 × 20 baseplate with C30 concrete, — Table G.33

Resistance given in the tables is 1376 kN > 1400 kN

Therefore the base is adequate — ∴ **OK**

CONNECTION DESIGN FOLLOWING THE DESIGN PROCEDURES

Check 1: Required area

Basic requirement: $A_p \geq A_{req}$

Area of base plate: $A_p = h_p \times b_p = 400 \times 400 = 160000 \text{ mm}^2$

Design compressive strength of the concrete: $f_{cd} = \alpha_{cc} \dfrac{f_{ck}}{\gamma_c} = 0.85 \times \dfrac{30}{1.5} = 17 \text{ N/mm}^2$

$f_{jd} = \alpha \beta_j f_{cd}$

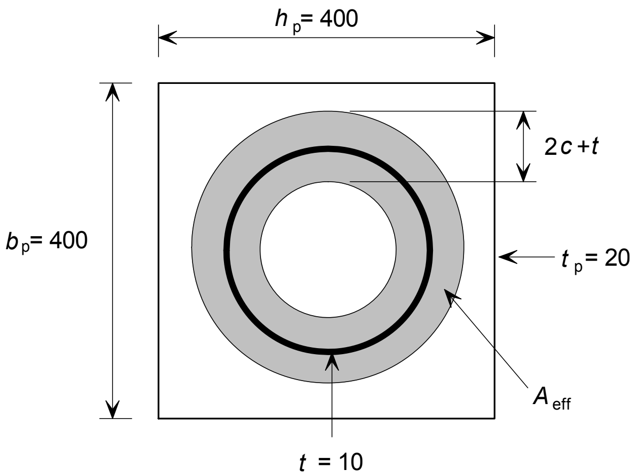
Assuming $\alpha = 1.5$, $\beta_j = 2/3$

$f_{jd} = 1.5 \times 2/3 \times 17 = 17 \text{ N/mm}^2$

Area required: $A_{req} = \dfrac{N_{Ed}}{f_{jd}} = \dfrac{1400 \times 10^3}{17} = 82400 \text{ mm}^2$

$A_p = 160000 \text{ mm}^2 > 82400 \text{ mm}^2$ $\therefore$ **O.K.**

Check 2: Effective area



Basic requirement: $A_{eff} = A_{req}$

To calculate the effective area, assume first that there is no overlap.

$A_{eff} = \pi(d - t)(t + 2c)$

$A_{eff} = \pi(273 - 10)(10 + 2c) = 82400 = A_{req}$

$526\pi c = 82400 - 2630\pi$

$\therefore c = 45$ mm

To ensure that there is no overlap c has to be less than half the internal diameter:

$\dfrac{d - 2t}{2} = \dfrac{273 - 2 \times 10}{2} = 126.5 \text{ mm} > 45 \text{ mm}$

Therefore the assumption that there is no overlap is correct.

To check that the effective area fits on the baseplate:

$d + 2c = 273 + 2 \times 45 = 363 \text{ mm} < 400 \text{ mm}$

Therefore the calculated value of c is valid.

Check 3: Plate thickness

$t_{p,min} = c\sqrt{\dfrac{3 f_{jd} \gamma_{M0}}{f_{yp}}}$

Title *Example 4 – Column Base – CHS*	*Sheet 3 of 3*
Yield strength of the 20 mm plate, f_{yp} = 265 N/mm^2	
$t_{p,min} = 45\sqrt{\dfrac{3 \times 17 \times 1.0}{265}} = 20$ mm	
$\therefore\ t_p = 20$ mm $\geq$ 20 mm	$\therefore$ O.K.
Check 4: Welds	
Basic requirement: $V_{Ed} \leq F_{w,Rd}\,\ell_{w,eff}$	
$F_{w,Rd} = f_{vw,d}a = \dfrac{f_u / \sqrt{3}}{\beta_w \gamma_{M2}} a = \dfrac{410/\sqrt{3}}{0.85 \times 1.25} \times 0.7 \times 6 = 936$ N/mm	
Conservatively assume 25% of the circumference is effective:	
$\ell_{w,eff} = 2\dfrac{\pi d}{4} = 2\dfrac{\pi \times 273}{4} = 429$ mm	
$F_{w,Rd}\,\ell_{w,eff} = 936 \times 429 \times 10^{-3} = 402$ kN	
$V_{Ed} = 100$ kN $<$ 402 kN	$\therefore$ **O.K.**

8 BRACING CONNECTIONS

8.1 INTRODUCTION

This Section gives general guidance on bracing connections and, where appropriate, refers to other publications for comprehensive detailed design.

Bracing members include flats, angles, channels, I sections, and hollow sections. Bracing arrangements may involve the bracing members working in tension alone, or in both tension and compression. In most cases, the bracing member is attached by bolting to a gusset plate, which is itself welded to the beam, to the column, or more commonly welded to the beam and its end connection as shown in Figure 8.1.

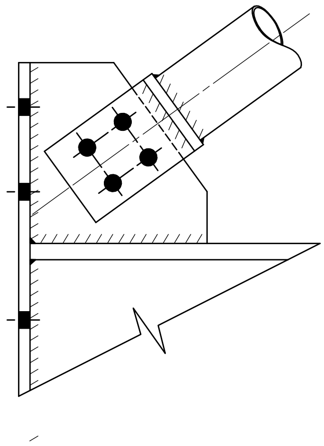


Figure 8.1 Typical bracing connection to gusset plate

Bracing systems are usually analysed assuming that all forces intersect on member centrelines. However, realising this assumption in the connection details may result in a connection with a very large gusset plate, especially if the bracing is shallow or steep. It is often more convenient to arrange the member intersections to make a more compact joint and check locally for the effects of eccentricities which are introduced.

Simple bracing connections usually involve a bolted lap joint with the gusset plate, as shown in Figure 8.2. The buckling resistance of the connection should be verified, as described in Section 8.5.

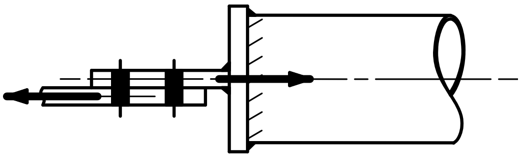


Figure 8.2 Lap joint between bracing and gusset

More significant eccentricities can arise with some details, as shown in Figure 8.3. In this case, the connecting elements must be checked for the moment introduced, or stiffened appropriately.

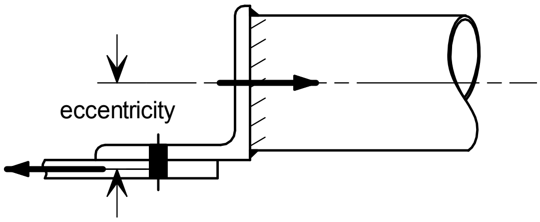


Figure 8.3 Eccentricity in connection to CHS

Bracing connections are generally made with non-preloaded bolts in clearance holes. In theory at least, this allows some movement in the connection, but in practice this is ignored in orthodox construction. In some cases it may be that movement on reversal is unacceptable – preloaded connections should be used in these circumstances.

The general design process is:

- identify the load path through the connection
- arrange the connection to ensure that the design intent of the members is realised (*e.g.* the beam connections remain nominally pinned)
- check the components in the connection and buckling of the lap and gusset plate.

Further comments on detailed design issues are given in the following Sections, firstly covering the bracing member and its connection, and then the considerations for the gusset and the main members.

8.2 ANGLES, CHANNELS AND FLATS

The design process is straightforward, as the axial force in the bracing member is transferred directly via the bolts. Members in tension should be checked in accordance with BS EN 1993-1-1 clause 6.2.3(2), carrying out a check of the gross section and of the net section. The net area of angles connected through one leg with one longitudinal line of bolts is covered by BS EN 1993-1-8 clause 3.10.3.

Channels may be used for larger tension forces, but the connection designer should recognise that the web of the member is relatively thin, and the bearing resistance may be the critical design check. If required, the web can be reinforced by welding a supplementary plate to the channel. A simple assumption is to assume that the entire load is transferred by the reinforcing plate, neglecting any contribution from the channel web, and specifying sufficient weld to ensure that the full force can be transferred between the channel and the reinforcing plate. The plate itself needs to be of sufficient size to distribute the load into the complete channel section.

8.3 HOLLOW SECTIONS

Hollow sections are often used as bracing members, being efficient in compression as well as tension. Typical connection details are shown in Figure 8.4. The "T" may be cut from a structural Tee section, or fabricated from plate.

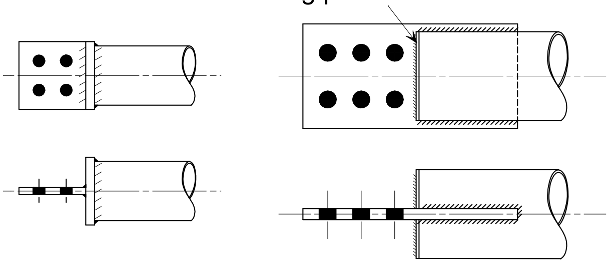


Figure 8.4 Connections to hollow sections

For lightly loaded members, the "T" shaped end connection is almost universally used. For larger forces, it is not uncommon to notch the connecting plate into the member, although this involves significant fabrication.

For the "T" shaped detail, in addition to determining the bolt resistance in shear and bearing, a check on the gross and net cross section will be required for members in tension, in accordance with BS EN 1993-1-1 clause 6.2.3(2).

The design of the end plate to circular hollow sections is covered in Reference 42, where the thickness of the end plate is based on the assumption of a dispersion of load at 2.5 to 1, as shown in Figure 8.5. Alternatively, the "effective breadth" can be calculated based on this dispersion, and only weld within this zone considered to be effective. The calculated weld size should be continued around the hollow section. The hollow section should also be checked to ensure that the design force can be accommodated, based on the area of the member within the effective breadth.

Connections to other types of hollow sections can be designed following these principles.

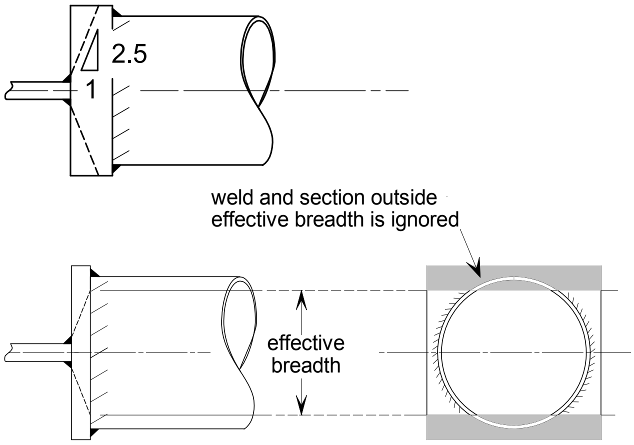


Figure 8.5 End plate for CHS

8.4 GUSSET PLATES

Preferably, gusset plates in compression should be supported on two edges and be reasonably compact. This may involve moving the point of intersection and checking for the eccentricity that this introduces. Plate thickness should be generous, as there is little economic benefit in minimising the plate thickness. Simple models are generally adequate for the design of the welds to the gusset plate; it is usually adequate to assume that the horizontal component of force is carried by the horizontal weld, and the vertical component to be carried by the vertical weld, as shown in Figure 8.6. The larger weld size should be used all round the gusset plate.

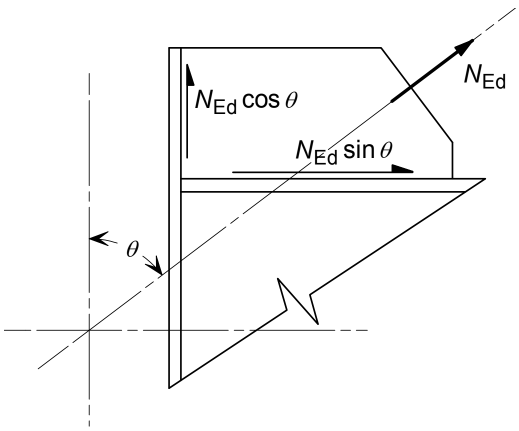


Figure 8.6 Simple gusset plate weld design

If more precision is justified, the size of the gusset plate can be chosen such that the centroid of the connection to both beam and end plate lie on the centroidal axis of the bracing member, as shown in Figure 8.7. This arrangement means that there are no bending moments on the connection between the gusset, beam and end plate. The welds to the gusset plate may be designed for the forces shown.

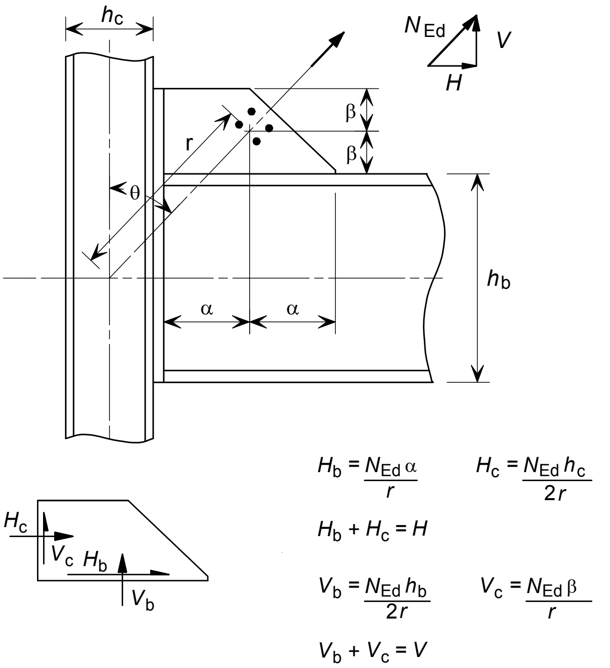


Figure 8.7 Force distribution to gusset plate welds

If the setting out point for the bracing intersection is adjusted to the intersection of the beam and column flanges, as shown in Figure 8.8, the forces in the welds are as described in the simple approach. Adopting this setting out means that the column must be designed for an additional moment M_c equal to the vertical component of the bracing force, *V*, multiplied by the eccentricity, e_c ($e_c = h_c/2$)¸ as shown in Figure 8.8. The column moment produced by this eccentricity is in addition to any nominal moments considered from the beam vertical reaction. The beam must also be designed for an additional moment M_b, which is the horizontal component of the bracing force, *H*, multiplied by the eccentricity e_b ($e_b = h_b/2$). The moment in the beam produces additional shear forces in the beam equal to M_b/L_b, where L_b is the length of the beam.

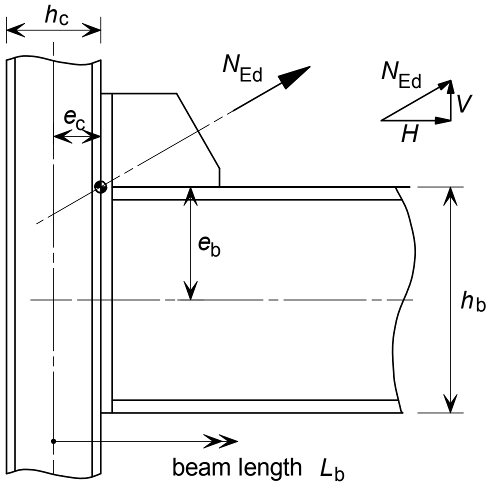


Figure 8.8 Force distribution for alternative setting out point at beam flange

If the setting out point is adjusted as shown in Figure 8.9, there is no additional moment in the beam to consider. The column should be designed for the additional moment M_c, as before. Clearly, the vertical component of the bracing force is now in addition to the shear force in the beam itself, and must be carried through the connection to the column. Care should be taken to check the shear resistance of the beam itself in this connection configuration. Where the gusset plate is supported on one edge only, the detail is only recommended for light loads. For heavier loads, an extended and plate and a gusset plate supported on two edges is recommended.

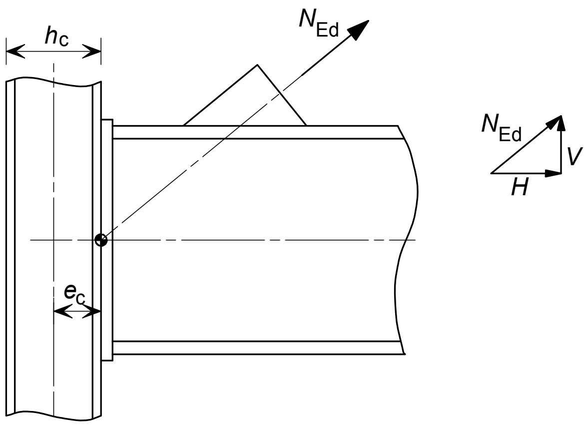


Figure 8.9 Force distribution for alternative setting out point on beam centre line

8.5 BUCKLING RESISTANCE

The following guidance has been developed following an extensive Finite Element study of members with lapped plate connections, which was calibrated against tests[43]. The study, and therefore the scope of the following rules, applies to:

- Connections with lapped plates,
- Tab plates with end plates, or tees, or tab plates notched into the bracing member (see Figure 8.10),
- Maximum 20 mm thick plates,
- Bolts in square or rectangular groups (not staggered patterns),
- Intersection angle θ (see Figure 8.11) greater than 20° and less than 160°
- Connections of normal proportion in realtion to the member, so that the member and the connections may be designed in isolation.

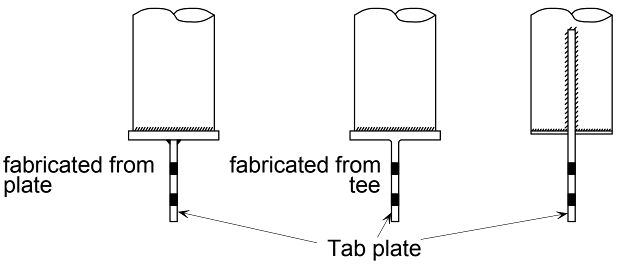


Figure 8.10 Bracing end connections

It is good practice to detail compact connections with the bolt group close to the support (close to the supporting member(s) and the bracing member). Elongated connections were not assessed as part of the FE study.

The buckling resistance of a gusset plate connection may be determined by assuming a yield line forms in both the tab plate and the gusset plate. The design model assumes that the support for the gusset plate and tab plate is 'fixed' (so that yield lines form in the gusset and tab plates) and this method is only appropriate for details that realise that assumption.

Connection resistance is verified by completing a combined axial load and moment interaction check on both the gusset plate and the tab plate. The moment resistance is based on a modulus part-way between elastic and plastic, which was found to give good correspondence with the FE study results. This partial plasticity is represented by the value of 5 which appears in the denominators of the expressions in Step 5.

The applied moment, which results from the axial force multiplied by the eccentricity between the plates, is distributed in inverse proportion to the plate stiffness. The initial eccentricity is increased to allow for second-order effects.

The design process (to calculate connection resistance) is first to determine the distribution of applied moment to each plate. The maximum axial force is then determined by rearranging the interaction check.

Step 1: Determine the length of each yield line

Tab plate

A yield line forms in the tab plate, adjacent to the end of the member. The maximum length is limited to $20t_{tab}$.

Gusset supported on one edge

A yield line forms, parallel to the yield line in the tab plate. If the gusset plate is supported perpendicular to the bracing member as shown in Figure 8.11(a), the yield line forms at the support. If the gusset plate is supported at an angle, the yield line is still parallel to the yield line in the tab plate, but from the nearest support point, as shown in Figure 8.11(b). For gusset plates supported on one edge only, the yield line is limited to $20t_{guss}$.

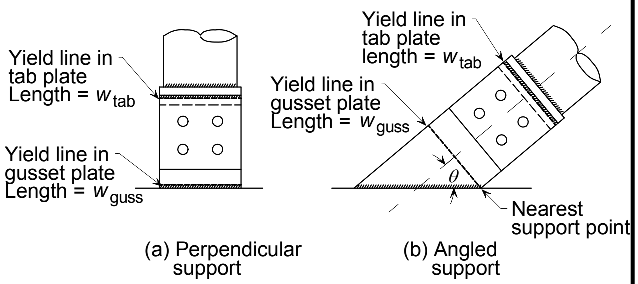


Figure 8.11 Yield lines – gusset plate supported on one edge only

Gusset supported on two edges

For gusset plates supported on two edges, the length of the yield line in the gusset plate is shown in Figure 8.12. If the 'point of nearest support' on both edges is 'in front' of the line of the last bolts, the yield line forms as shown in case (a). If only one 'point of nearest support' is 'in front of' the line of the last bolts, the yield line forms as case (b). If both 'points of nearest support' are 'behind' the line of the last bolts, the yield line forms parallel to that in the tab, from the 'point of nearest support', as case (c).

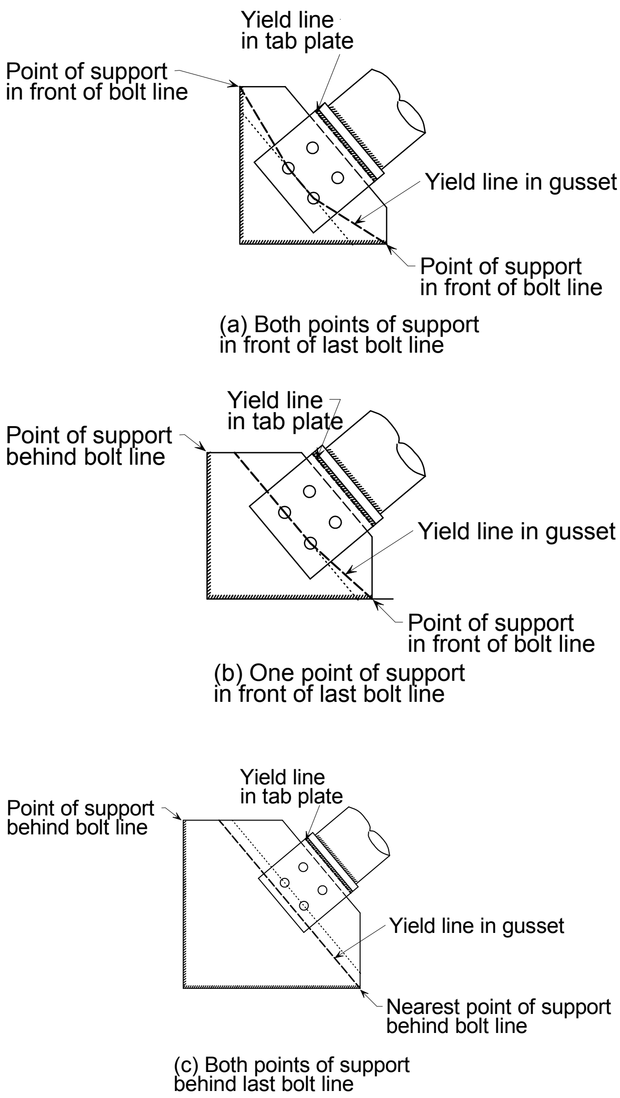


Figure 8.12 Yield lines in gusset plate supported on two edges

For gusset plates supported on two edges, the length of yield line is limited to $50t_{guss}$.

Step 2: Allow for second-order effects

The initial eccentricity, between centrelines of the plates is increased by k_{amp} to allow for second-order effects.

The initial eccentricity = $0.5(t_{tab} + t_{guss})$.

k_{amp} = 1.05 for gusset plates supported on two edges, case (a) only.

k_{amp} = 1.2 in all other cases.

Step 3: Calculate the inertia of each plate, and the lapped portion of the connection

The inertias of the plates and lapped portion of the connection are calculated, based on the length of yield line previously calculated. The inertia of the lapped portion is based on the combined thickness and the average length of the yield lines. Nomenclature is shown in Figure 8.13 for cases (a) and (b) and in Figure 8.14 for case (c).

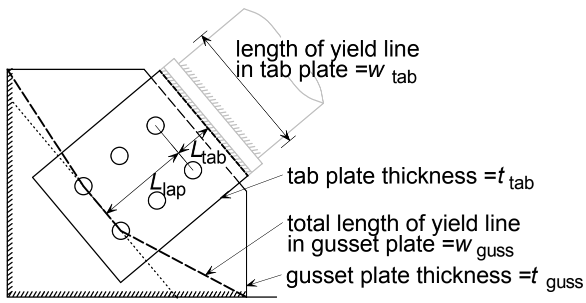


Figure 8.13 Nomenclature for gusset plate calculations – cases (a) and (b)

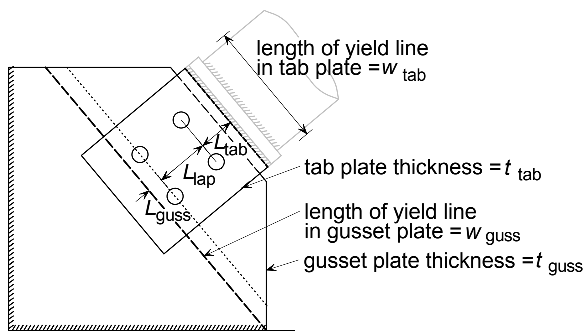


Figure 8.14 Nomenclature for gusset plate calculations – case (c)

The inertia of each component is given by:

$$I_{tab} = \frac{w_{tab} \times t_{tab}^3}{12}$$

$$I_{lap} = \frac{0.5 \times (w_{tab} + w_{guss}) \times (t_{tab} + t_{guss})^3}{12}$$

$$I_{guss} = \frac{w_{guss} \times t_{guss}^3}{12}$$

Step 4: Determine moment distribution factors

Notional moments:

$$M_{tab} = \frac{1}{\dfrac{L_{tab}}{EI_{tab}} + \dfrac{L_{lap}}{2EI_{lap}}} \quad \text{and} \quad M_{guss} = \frac{1}{\dfrac{L_{guss}}{EI_{guss}} + \dfrac{L_{lap}}{2EI_{lap}}}$$

Note that in cases (a) and (b) $L_{guss} = 0$.

The distribution factors are:

$$\mu_{tab} = \frac{M_{tab}}{M_{tab} + M_{guss}} \quad \text{and} \quad \mu_{guss} = \frac{M_{guss}}{M_{tab} + M_{guss}}$$

$\mu_{tab} + \mu_{guss} = 1.0$

For plates supported on two edges, and for cases (a) or (b) (see Figure 8.12), it may conservatively be assumed that μ_{guss} = 1.0 and μ_{tab} = 0.1 for preliminary design.

Step 5: Calculate connection resistance

The axial resistance of the connection is given by the minimum of the resistance of the tab and the gusset plate. The resistance of the tab and gusset is given by:

$$N_{Rd,tab} = \frac{w_{tab} f_{y,tab} t_{tab}^{2}}{\left(5 \times k_{amp} \times 0.5\left(t_{tab} + t_{guss}\right) \times \mu_{tab} + t_{tab}\right)\gamma_{M0}} \text{ and}$$

$$N_{Rd,guss} = \frac{w_{guss} f_{y,guss} t_{guss}^{2}}{\left(5 \times k_{amp} \times 0.5\left(t_{tab} + t_{guss}\right) \times \mu_{guss} + t_{guss}\right)\gamma_{M0}}$$

γ_{M0} may be taken as 1.0, based on a reliability analysis verified with full size physical tests.

Connections with two bolts only

If connections are detailed with two bolts only, the design rules are modified as follows.

If both 'points of nearest support' are 'behind' the line of the bolts, as shown in Figure 8.15, the expressions given above may still be used; L_{lap} is taken as zero.

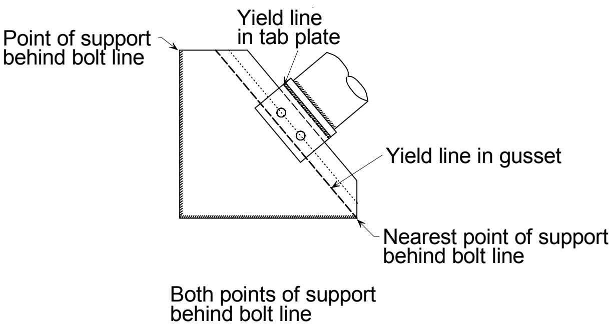


Figure 8.15 Two bolt connection – both points of support 'behind' the bolt line.

In the common case where at least one point of support is 'in front' of the bolt line, as shown in Figure 8.16, the distribution in Step 4 cannot be used; it may be assumed that μ_{guss} = 1.0 and μ_{tab} = 0.1.

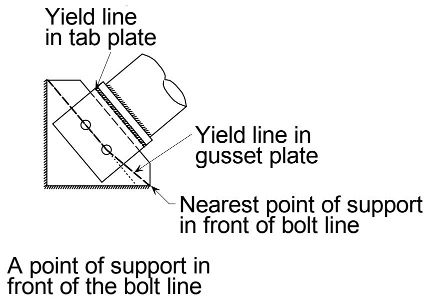


Figure 8.16 Two bolt connection – at least one point of support 'in front' of the bolt line.

Welds

The welds between the tab plate and the end plate must be full strength, to ensure that the tab plate yields, not the weld. For gusset plates supported on one edge only, the welds should be full strength.

For gusset plates supported on two edges, the welds to the gusset plate must ensure that the gusset plate yields before the weld. This may be achieved by providing full strength welds. The FE studies revealed that the stresses in the plate diminish rapidly towards the point where the two weld lines meet, and in this zone a weld size based on the guidance in Section 8.4 would be appropriate. Based on inspection of the FE results, it is recommended that the full strength welding should extend to a point at least $5t_{guss}$ past a line from the "futherst point of support", perpendicular to the axis of the bracing member as shown in Figure 8.17.

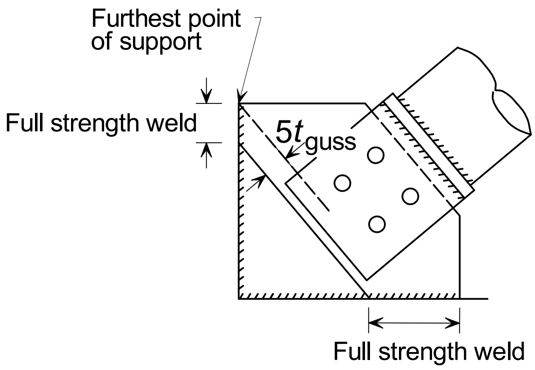


Figure 8.17 Extent of full strength welding – gusset plates supported on two edges.

Bolts

The resistance of the bolt group should be checked, in shear and bearing.

8.6 LARGE BRACING FORCES

Although the principles described in Section 8.5 can be applied to connections subject to large forces, alternative details may be considered. Stiffening either the gusset plate of the tab plate will increase the connection resistance considerably.

Reducing the initial eccentricity by detailing a ‘concentric’ connection with symmetric cover plates (Figure 8.18) is possible, but the ‘sway’ mechanism is not prevented in the detail shown in Figure 8.18.

The initial eccentricity cannot be entirely eliminated, as imperfect fit-up must be considered and the initial imperfection of the bracing member will introduce eccentricity into the connection as load is applied.

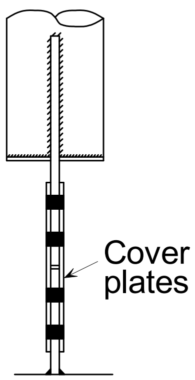


Figure 8.18 ‘concentric’ connection with cover plates.

8.7 EFFECTS OF BRACING CONNECTIONS ON JOINT PERFORMANCE

The general principle for the beam to column connections described in this publication is that connections assumed to be pinned in the analysis should realise that assumption in practice.

In general therefore, end plates are relatively thin, to try to ensure ductile behaviour. However, if substantial horizontal forces must be transferred from the bracing into the column, the end plates will inevitably be thicker and may need to be extended above or below the beams. The resulting connection will then be stiffer than assumed.

In orthodox construction the effect of bracing connections is often neglected but there will be instances where the structure should be reanalysed as a continuous frame.

9 SPECIAL CONNECTIONS

9.1 INTRODUCTION

Steelwork connections for simple construction, illustrated in Sections 1 to 8 of this publication, 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.

When designing special connections, it may be possible to use a modified version of one of the standardised connections, subject to additional design checks. Such connections should incorporate, as far as possible, the components sizes given in Section 2 and the design principles adopted in this publication.

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.

Table 9.1 deals with cases where connecting beams are at different levels. Table 9.2 considers beams which connect members not intersecting at 90°.

Table 9.3 and Table 9.4 show connections to hollow section columns where welding and conventional bolting are adopted and which provide an alternative to the use of Flowdrill and Hollo-Bolt connectors.

Table 9.5 takes account of eccentric connections necessary for off-grid beams.

A case where different size columns are spliced such that the outer faces are aligned is shown in Table 9.6. The special requirements for a column base in a braced bay are covered in Table 9.7.

The design of non-standard connections may require specialist advice. SCI offer a design service that may be consulted for such connections

Table 9.1 Beams at different levels

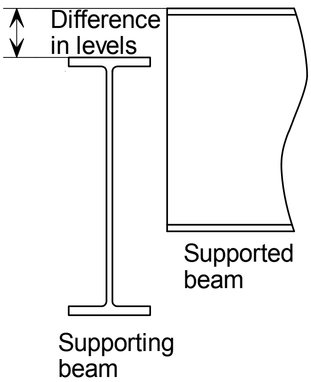
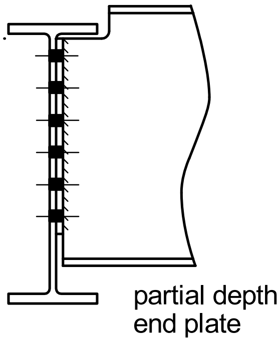
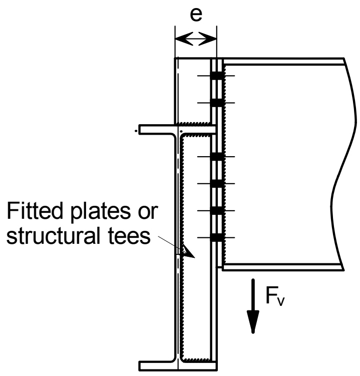
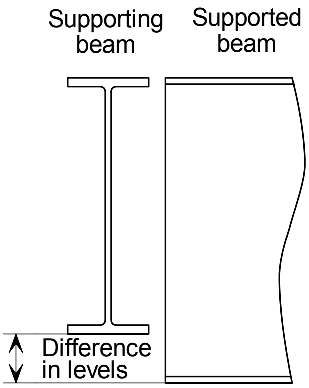
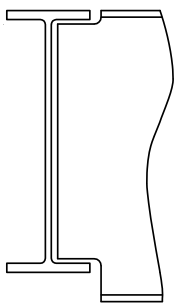
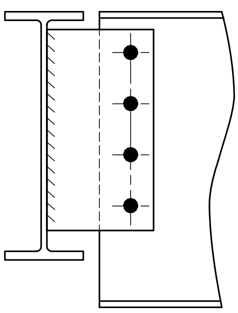
Requirement	Possible Solution	Special Considerations
(1) Beam to beam 	**(i) If level difference is small** 	• Reduced local shear and bending resistance of supported beam • Unrestrained top flange • Reduced number of bolts
	(ii) If level difference is large 	• Torsion on the supporting beam may be a consideration in one-sided connections. • Expensive to fabricate
(2) Beam to beam 	**(i) Increase depth of supporting member**	• Consider ease of connection at conceptual design stage
	(ii) Notched supported beam 	• Reduced shear and bending resistance 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 9.1 (continued)

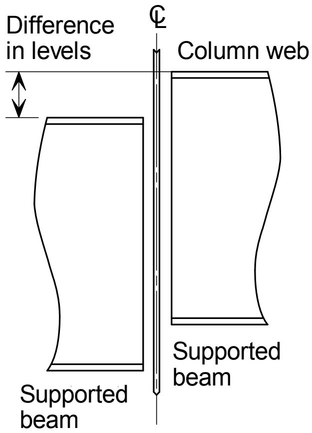
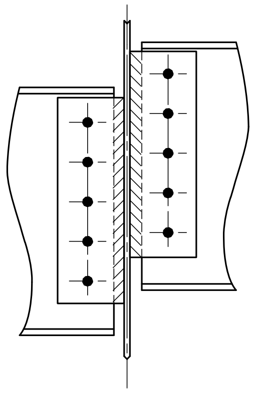
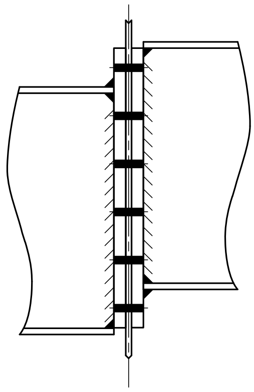
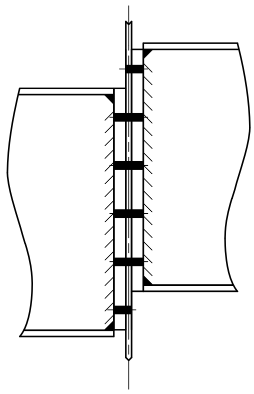
Requirement	Possible solution	Special considerations
(3) Beam to column web  	**(i) Fin plates** 	
	(ii) Extended end plates 	● Ease of erection ● May need to vary pitch to ensure adequate clearance
	(iii) Non-standard pitches 	● Bearing on column web from central, heavily loaded bolts ● May need to vary pitch to ensure adequate clearance

Table 9.2 Skewed connections

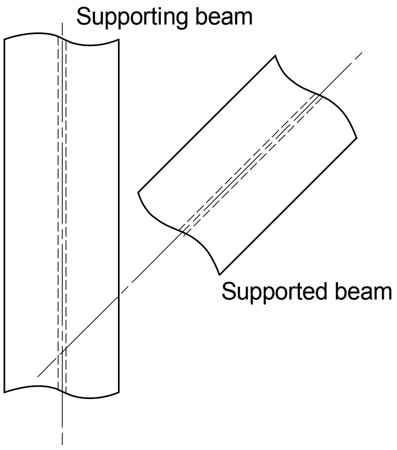
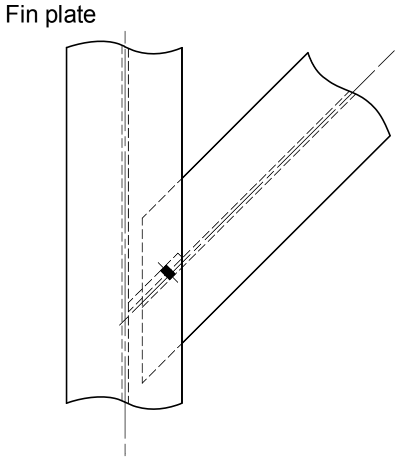
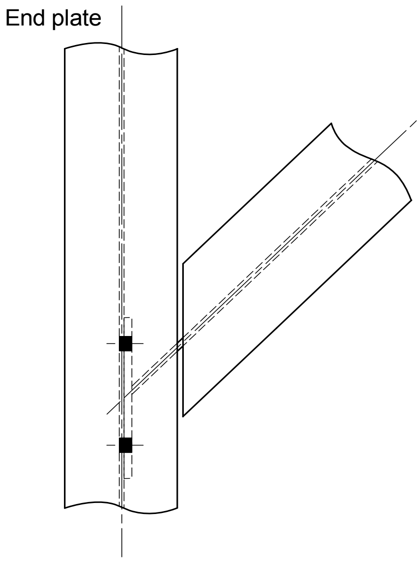
Requirement	Possible Solution	Special Considerations
(1) Beam to beam 	**(i) Supported beam depth less than supporting beam depth** 	• Reduced local shear and bending resistance of supported beam due to long top notch • Non-standard fin plate may be required to provide adequate bolt clearance
	(ii) Supported beam depth greater than supporting beam depth 	• Reduced local shear and bending resistance of supported beam due to long top and bottom notch • Non-standard end plate may be required to provide adequate bolt clearance

Table 9.2 (continued)

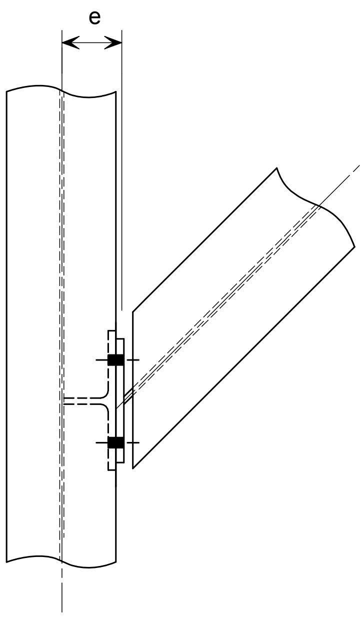
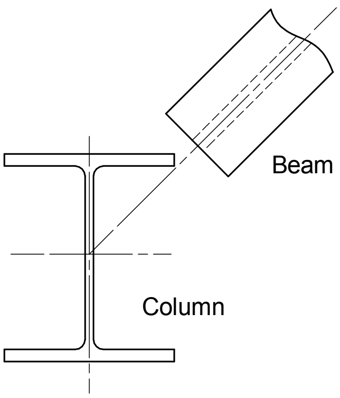
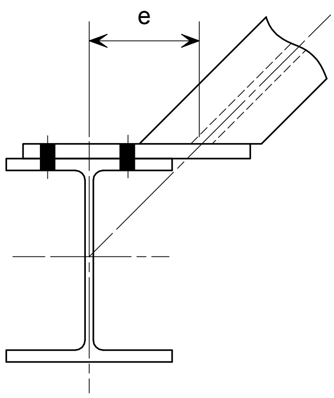
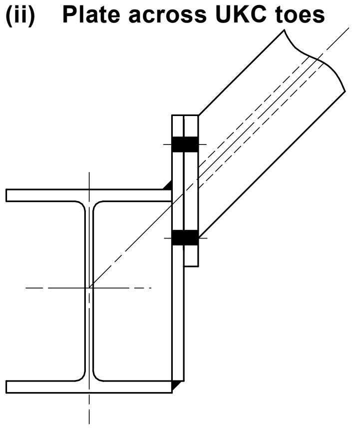
Requirement	Possible Solution	Special Considerations
	(iii) Beams of same depth 	● Expensive to fabricate ● Clearance for bolts ● Torsion of the supporting beam may be a consideration
(2) Beam to column 	**(i) Extended end plate** 	● Bolt group and end plate designed to resist moment due to eccentricity ● Structural integrity – large bolt tensions developed ● Structural integrity – thick end plate required ● Structural integrity – check column flange for bending
	(ii) Plate across UKC toes 	● Structural integrity – large tensions developed in weld ● Structural integrity – thick plates may be required

Table 9.3 I-Beam to hollow section column

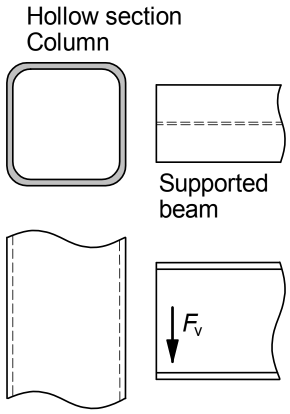
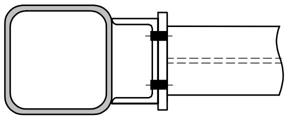
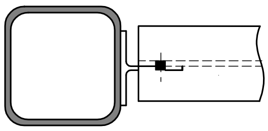
Requirement	Possible solution	Special considerations
	**(i) Channel Bracket** 	• Provides alternative to direct bolting by Flowdrill or Hollo-Bolt • Bracket transmits load direct to column side wall welds • Bracket and welds to resist moment due to eccentricity
	(ii) Tee connection 	• Tee stiffens column wall and can be used where a fin plate is insufficient

Table 9.4 Parallel beams to hollow section column

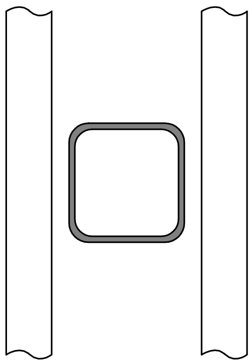
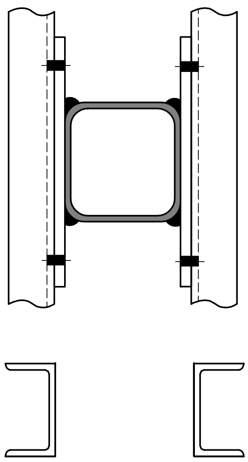
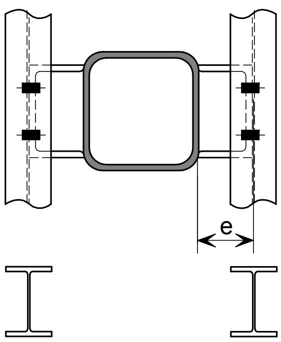
Requirement	Possible solution	Special considerations
	**(i) Side plates** 	• Bolts and welds to resist shear and any moment due to out of balance loading
	(ii) Bracket 	• Bracket transmits loads direct to column side wall welds • Bracket and welds to resist moment due to eccentricity

Table 9.5 Off-grid connections

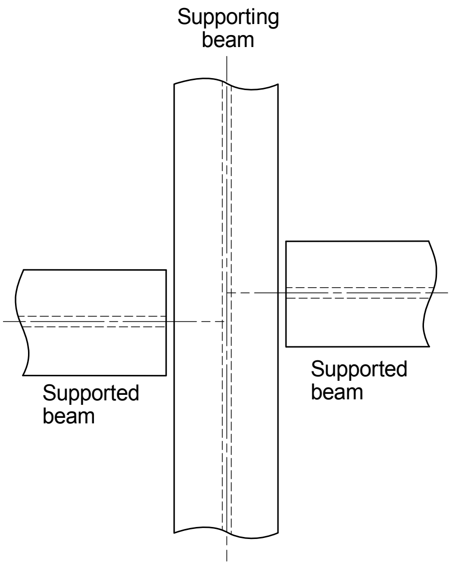
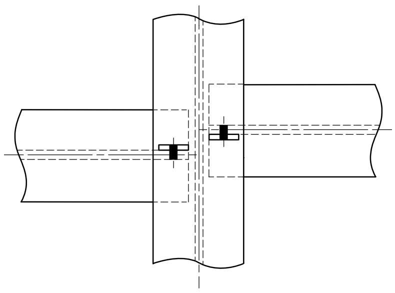
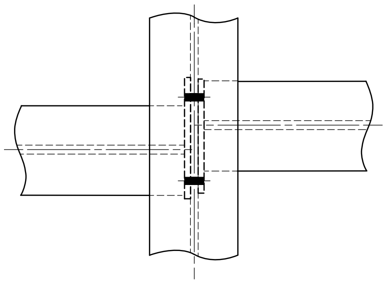
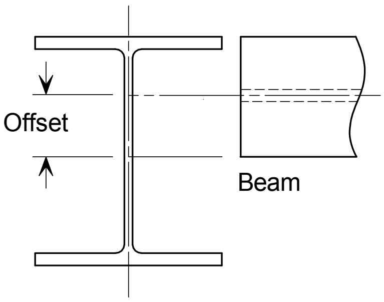
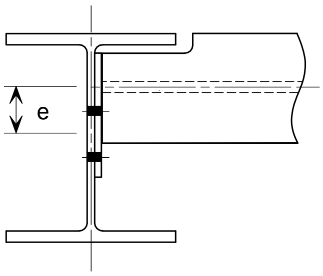
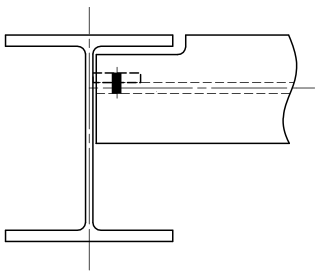
Requirement	**Possible solution**	**Special considerations**
(1) Beam to beam 	**(i) Fin plates** 	● Ensure beam connects to correct side of fin plate
	(ii) End plates (or non-standard cleats) 	● Non-standard bolt spacings and fittings required
(2) Beam to column small offsets 	**(i) End plate** 	● Bolt group and end plate designed to resist moment due to eccentricity ● Non standard bolt centres and end plate
	(ii) Fin plate 	● Bolt clearances and ease of installation on site

Table 9.5 (continued)

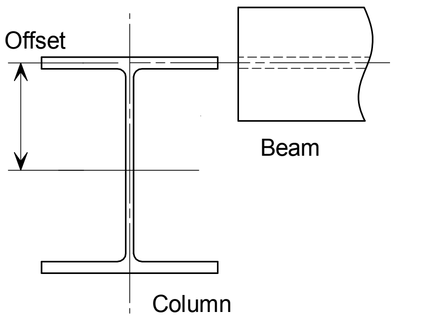
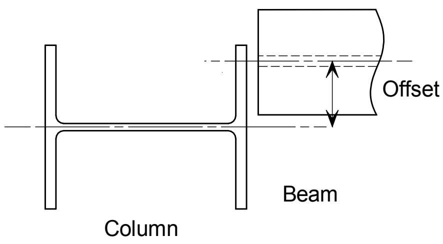
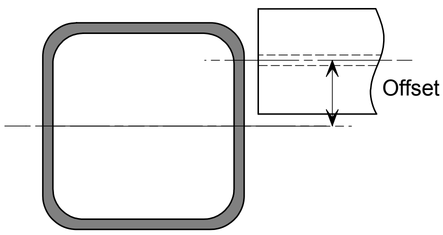
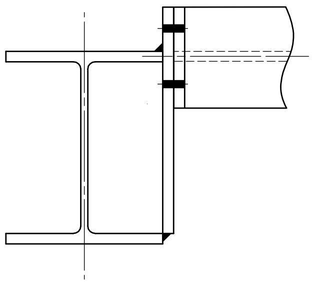
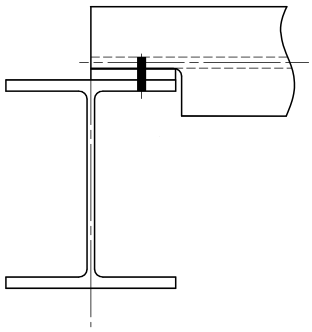
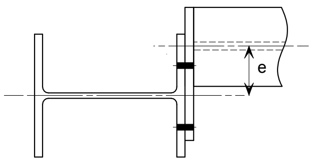
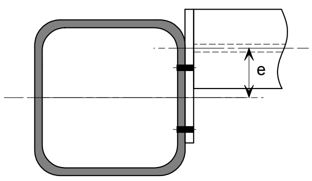
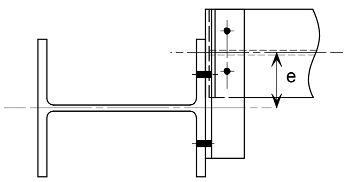
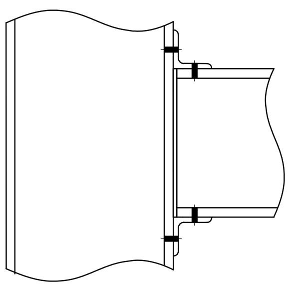
Requirement	Possible Solution	Special Considerations
(3) Beam to column large offsets   	**(i) Plate across UKC toes** 	● Non-standard fittings
	(ii) Packs to column flange 	● Reduced bolt shear capacity due to packs See BS EN 1993-1-8 clause 3.6.1(12)
	(iii) Extended end plate  	● Design bolt group and end plate for moment due to eccentricity
	(iv) Top and bottom cleats  	● Design bolts to column flanges and bottom cleat for moment due to eccentricity ● Design bottom cleat to carry the full vertical load ● Check the resistance of the supported beam web

Table 9.6 Column splices

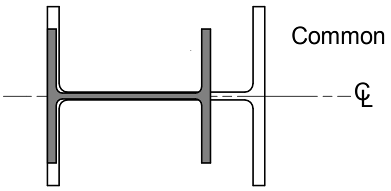
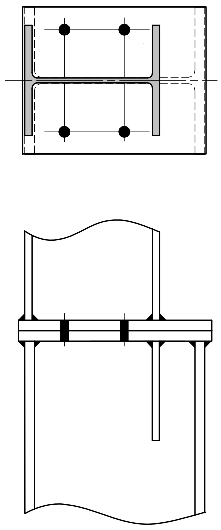
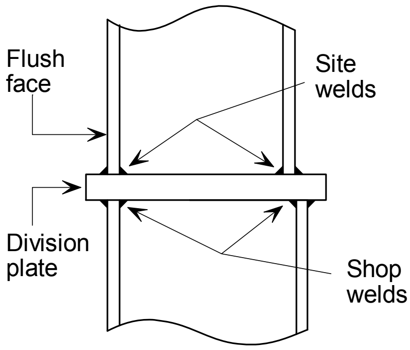
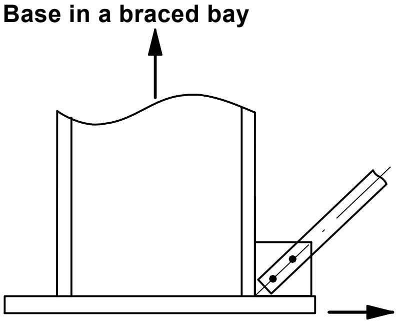
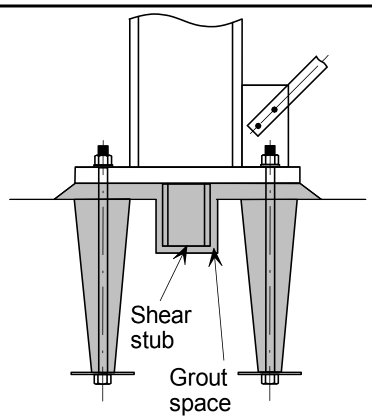
<table>
<tr><th>Requirement</th><th>Possible solution</th><th>Special considerations</th></tr>
<tr><td rowspan="2">There may be occasions when architectural or other requirements dictate that one of the column faces align, making the upper and lower columns eccentric.

</td><td>(i) Cap and base splice
</td><td>● Cap and base splices are especially suitable if there is a large change in sections 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 or where there are any structural integrity requirements</td></tr>
<tr><td>(ii) Site welded splice

</td><td>● Site welding can be expensive</td></tr>
</table>

Table 9.7 Column bases in braced bays

<table>
<tr><th>Requirement</th><th>Possible solution</th><th>Special considerations</th></tr>
<tr><td>Base in a braced bay
</td><td>
</td><td>● Welds to column designed to resist uplift and horizontal shear
● Design base plate for bending
● May need to provide shear stub to resist large horizontal shear</td></tr>
</table>

10 REFERENCES

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APPENDIX A STRUCTURAL INTEGRITY

A.1 GENERAL

BS EN 1991-1-7[7] covers accidental actions, including the requirements for structural integrity and the avoidance of disproportionate collapse.

Although this publication only relates to design of the steelwork, the structural integrity requirements are intended to ensure robustness of the complete 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 the requirements of BS EN 1991-1-7. Integrated solutions of this type will usually give greater economy than carrying the tying force through the steelwork alone.

The structural integrity requirements need to be satisfied for beam to column connections and for column splices. Methods to calculate the tying resistance of connections are given in Sections 4 to 6. It has been assumed that one-sided beam to beam connections are rarely designed for a tying force and thus no design methods are presented. Designers may adapt the beam to column design methodology for one-sided beam to beam connections.

The check for tying resistance is entirely separate to the check for vertical shear resistance – the two design forces do not occur in the same design situation and therefore should not be considered at the same time. It should also be recognised that when calculating the tying resistance, substantial permanent deformations are anticipated. The design checks are therefore generally based on ultimate strengths and use an appropriate material factor.

In general, the material factor γ_{Mu} is used when calculating resistances. Although not given in the Eurocode, the use of γ_{Mu} = 1.1 is recommended.

In the present publication, the methodology given in the Eurocode for the tension resistance of T-stubs has been adopted for the calculation of the tying resistance of end plates. This approach generally results in a higher resistance than that calculated by the previous model[16].

The tying resistance of fin plates can be determined by simple structural analysis; the structural model is not complex.

The tying resistance of splices with cover plates is also straightforward.

A.2 PRYING AND TYING FORCES

When calculating tying resistance it is accepted that the members and their connections will experience substantial permanent deformations. Several unusual features follow from taking account of the large displacements and strains:

a) The gross deformations can reduce the local eccentricities within the connection.
b) The local strains are so high that account can be taken of strain hardening, so that the design strength is based on f_u the minimum ultimate tensile strength, with a suitable material factor γ_{Mu}.
c) Bolt resistances are calculated using γ_{Mu}, not γ_{M2}.

APPENDIX B TYING RESISTANCE OF PARTIAL DEPTH AND FULL DEPTH END PLATES

B.1 PARTIAL DEPTH END PLATES

The tying resistance of partial depth end plates is based on a very simple model that assumes the end plates are in double curvature bending, with yield lines along the bolt lines and adjacent to the weld.

Three modes of failure are considered:

- Mode 1, where the critical component is the bending of the end plate (prying is considered)
- Mode 2, where the critical components are the bolts and the end plate in bending (prying is considered)
- Mode 3, where the critical components are the bolts in tension (no prying)

The resistance of each mode is verified generally in accordance with BS EN 1993-1-8 Table 6.2, but with f_u and γ_{Mu} instead of f_y and γ_{M2}, as noted in Appendix A.

B.2 FULL DEPTH END PLATES

When the end plate is welded to the flanges, the simple model used for partial depth end plates should not be used because of the enhancement offered by the connection to the flanges. The welding to the flange produces a complex series of yield lines in the area adjacent to the flange. To determine the tying resistance of a full depth end plate, this publication uses the methods given in BS EN 1993-1-8 Clause 6.2.4 and Table 6.2 to calculate the tension resistance of an equivalent T-stub. An equivalent T-stub has a resistance which can be readily calculated, and is equal to the resistance of the real connection component. This method was originally intended to calculate the resistance in the tension zone of moment-resisting connections under normal loading conditions. The method considers the 3 modes of failure referred to in Annex B1.

Whilst it is accepted that the method can be extended to cover the case of structural integrity, caution is recommended when applying this calculation outside the scope of this publication. Initial testing was carried out in 2010 on standard full depth end plate details, which demonstrated that the resistance was substantially higher than that calculated. As more testing is completed, it may be that an improved model can be developed, including revised welding requirements.

It is important that the weld to the flange is sufficient to ensure that yield lines develop in the end plate. At the time of writing (2011), the distribution of force to the flange and web welds cannot be determined with precision and for this reason full strength welds are specified. Based on tests, it may be possible to reduce the weld size, or indeed consider an alternative load path. The current model assumes that half the tying force is transferred through each of the flange to end plate welds.

APPENDIX C WELDS FOR END PLATE AND FIN PLATE CONNECTIONS

The design checks for end plates and fin plates provide simple equations to design the welds. The checks ensure that the welds are not the weakest part of the connection, so that brittle failure is avoided. The following sections describe how the simple equations are obtained and the reasoning behind the rules.

C.1 BASIC RULES

Symmetric fillet welds are provided on both sides of the beam web for end plates and on both sides of fin plates. Equation 4.1 from BS EN 1993-1-8 gives the directional method for the design of fillet welds as:

$$\left[\sigma_{\perp}^{2} + 3\left(\tau_{\perp}^{2} + \tau_{||}^{2}\right)\right]^{0.5} \leq \frac{f_{u}}{\beta_{w}\gamma_{M2}}$$

Welds subject to transverse force

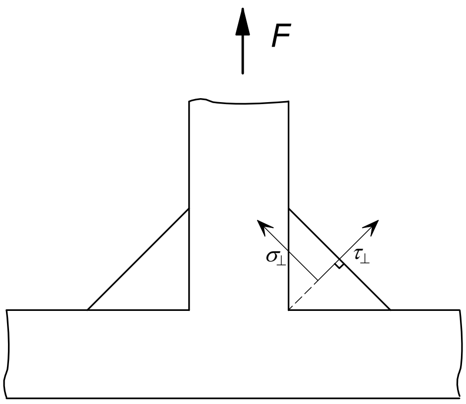


Figure C.1 ***Fillet weld subject to transverse force***

For a weld such as that shown in Figure C.1 each of the terms in the equation can be expressed as:

$$\sigma_{\perp} = \frac{F\sqrt{2}}{4a}$$

$$\tau_{\perp} = \frac{F\sqrt{2}}{4a}$$

$$\tau_{||} = 0$$

So that the expression above becomes:

$$\frac{F\sqrt{2}}{4a}\left(1+3\right)^{0.5} \leq \frac{f_{u}}{\beta_{w}\gamma_{M2}}\text{, or}$$

$$\frac{F\sqrt{2}}{2a} \leq \frac{f_{u}}{\beta_{w}\gamma_{M2}}$$

And therefore the minimum throat thickness required can be expressed as:

$$a \geq \frac{F\sqrt{2}}{2}\frac{\beta_{w}\gamma_{M2}}{f_{u}}$$

For a fillet weld to be full strength, it must be capable of carrying the maximum force that the beam web can carry, and therefore:

$$F = t_{w} f_{y,b}$$

This leads to the expression of the minimum weld throat required:

$$a \geq \frac{f_{y,b}}{\sqrt{2}}\frac{\beta_{w}\gamma_{M2}}{f_{u}} t_{w}$$

When considering tying, the design may be based on a material factor γ_{Mu}, which is taken as 1.1.

C.2 END PLATE WELDS SUBJECT TO SHEAR

The fillet weld is subject to shear. It is also subject to an (uncalculated) bending moment as the beam deflects.

A full strength weld will obviously be adequate, but the weld size will be significantly larger than previous UK practice, when the weld was designed only for for the applied vertical shear. ECCS Technical Committee TC10 Structural Connections considered that full strength is not always necessary - the minimum requirement for the weld is that the end plate should yield before the weld fractures. The Committee proposed in its draft guide that a factor of 0.8 could be applied when calculating the required weld size. Although this proposal was not included in the published guide,[24] it is recommended in the UK.

The rules described above result in the following weld sizes, which specify the minimum throat as a proportion of the web thickness.

	S275	S355
Normal design	0.40	0.48

In almost all cases, the tying force will be no more than the shear force and the weld size for normal design may be adopted. If the tying force were larger than the shear force, the weld should be designed in accordance with the Standard.

C.3 FIN PLATE WELDS

Full strength welds to a fin plate are required as the connection experiences shear and possibly significant bending (uncalculated) in the normal design condition. Previous UK practice was to make the welds larger than full strength to allow for the bending moment – this rule has been relaxed and full strength welds are specified. The 0.8 factor used in welds for end plates (Section C.2) is not used for fin plates as the bending moments are likely to be more significant.

When considering tying, full strength welds are provided but the design is based on a material factor γ_{Mu}, which is taken as 1.1.

The rules described above result in the following weld sizes, which specify the minimum throat as a proportion of the web thickness.

	S275	S355
Normal design	0.50	0.60
Tying resistance	0.44	0.53

For simplicity, the design rules and standard connection details in this publication adopt the more onerous requirement.

C.4 ALTERNATIVE WELD DESIGN

In some cases, where the loads are not significant, these simple equations might be too conservative. In such cases, it is possible to design the welds according to BS EN 1993-1-8, accounting for the vertical shear and the inevitable coexisting moment in the connection.

APPENDIX D THERMAL DRILLING OF HOLLOW SECTIONS

D.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.

D.2 DRILLING TOOL AND PROCESS

The initial hole is made by the thermal drilling tool which consists of a tungsten carbide bit held in a taper collet adaptor (Figure D.1). The tool can be used in a conventional drilling machine or CNC machine, as found in a steelwork contractor's works, provided that it has adequate power and spindle speed.

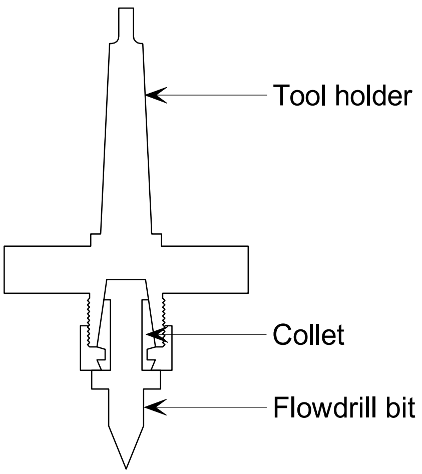


Figure D.1 Thermal drilling tool

The process is shown in Figure D.2. During the first stage the tungsten carbide bit is brought into contact with the hollow section 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 hollow section.

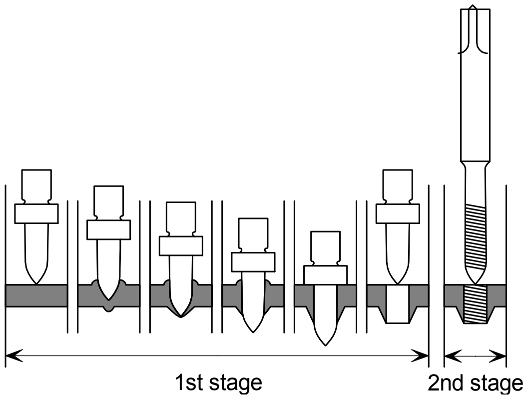


Figure D.2 Thermal drilling process

The second and final stage is to tap the formed bush. This is done by roll threading the bush with a coldform tap.

D.3 APPLICATION & LIMITATIONS

Flowdrill or Formdrill connections to hollow section columns can be made using either double angle cleats (not covered in this publication) or end plates, as shown in Figure D.3.

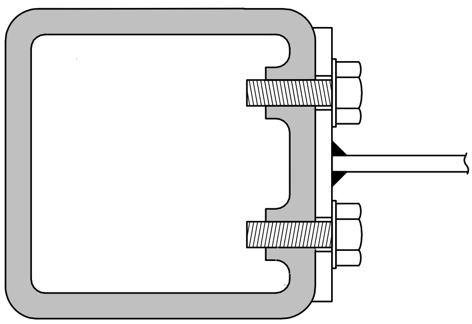


Figure D.3 End plate connection using Flowdrill

When using Flowdrill or Formdrill connections it must be noted that:

a) if used at locations exposed to the weather they should not be considered as watertight unless special protective measures are taken
b) they are not suitable for use with pre-galvanised materials.

D.4 FURTHER INFORMATION

Further information on thermal drilling, including drilling machine parameters and tool sizes, can be obtained from the companies given below:

Flowdrill (UK) Limited
Unit 7, Hopewell Business Centre
105 Hopewell Drive
Chatham
Kent ME5 7DX
Tel: 01634 309422
www.flowdrill.com

Formdrill
Robert Speck Ltd
Little Ridge
Whittlebury Road
Silverstone
Northants. NN12 8UD
Tel: 01327 857307
www.robertspeck.com

For detailing requirements and tension resistance of bolts see Tables G.58, G.59 and G.68 of the yellow pages.

APPENDIX E HOLLO-BOLT CONNECTIONS TO HOLLOW SECTIONS

E.1 INTRODUCTION

The Lindapter Hollo-Bolt is a patented method of fixing to hollow sections 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 as indicated in Table E.1.

Table E.1 Hollo-Bolt Type

Bolt Size	Hollo-bolt type
M8	3 Part
M10	3 Part
M12	3 Part
M16	5 Part
M20	5 Part

The pre-assembled Hollo-Bolt unit is inserted through normal tolerance holes in both the fixture and the hollow section. 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. Figure E.2 and Figure E.3 show the Hollo-Bolt in the installed condition.

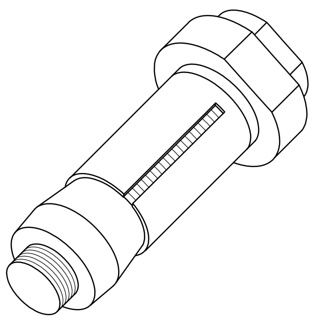


Figure E.1 Pre-assembled Hollo-Bolt unit

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 hollow section columns can be made using either double angle cleats (not covered in this publication) or flexible end plates, as shown in Figure E.4.

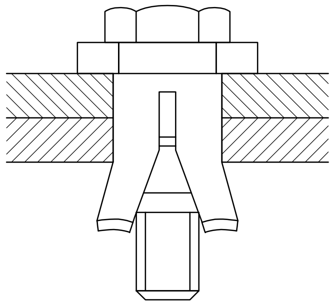


Figure E.2 Installed 3-part Hollo-Bolt (M8, M10 & M12)

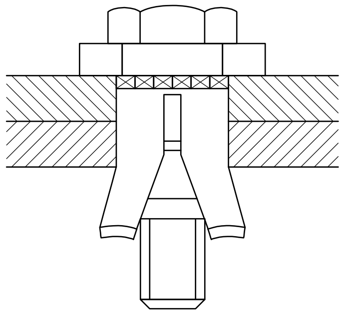


Figure E.3 Installed 5-part Hollo-Bolt (M16 & M20)

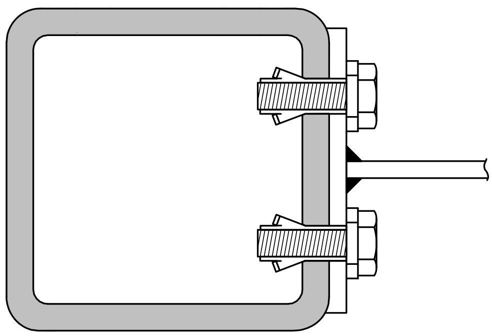


Figure E.4 Hollo-Bolts used with end plate connection

E.2 INSTALLATION

Hollo-Bolt uses a plain drilled hole which can be made on site or in the fabrication shop using 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 E.2.

Should the steelwork need to be adjusted, the fixing can simply be removed and the hole reused with another Hollo-Bolt.

Table E.2 Required tightening torque

Bolt Size	Tightening torque (Nm)
M8	21
M10	40
M12	78
M16	190
M20	300

E.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 austenitic stainless steel, with a grade A4-80 central bolt. This will not be a stocked item and would be manufactured to order.

E.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)

a) Stainless steel
b) Button head setscrew
c) Countersunk setscrew/body
d) Socket head capscrew
e) Special body length

E.5 FURTHER INFORMATION

Further information on Hollo-Bolt is available from:

Lindapter International
Lindsay House
Brackenbeck Road
Bradford
West Yorkshire
England
BD7 2NF

Telephone: 01274 521 4444
www.lindapter.com

For detailing requirements and resistance tables for the Hollo-Bolt, see Tables G.60, G.61 and G.69 of the yellow pages.

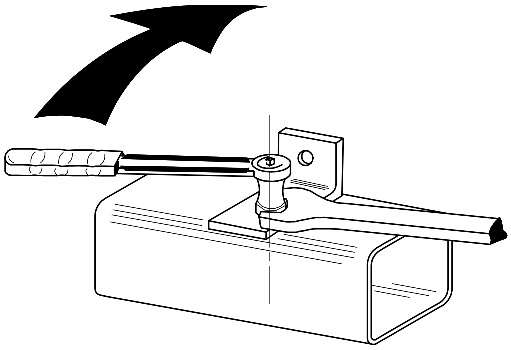


Figure E.5 Installation of Hollo-Bolt

APPENDIX F BLIND BOLT CONNECTIONS TO HOLLOW SECTIONS

F.1 INTRODUCTION

Blind bolts are a patented connection for fixing to hollow sections or when access is only available from one side. As illustrated in Figure F.1, the blind bolt has a weighted toggle which sits within the shaft of the bolt and which bears against the inside of the member. The bolt is inserted though the hole in the items to be connected, and then rotated, which allows the weighted toggle to drop into position.

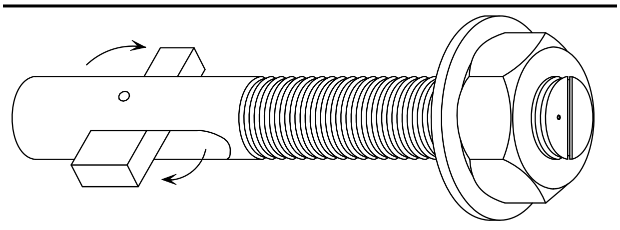


Figure F.1 Blind bolt

F.2 INSTALLATION

The installed bolt is shown in Figure F.2, after the bolt has been rotated to allow the toggle to drop into position. There is an indicator on the end of the bolt shank to indicate the orientation of the bolt. The nut is then tightened whilst holding the bolt shank in the same orientation.

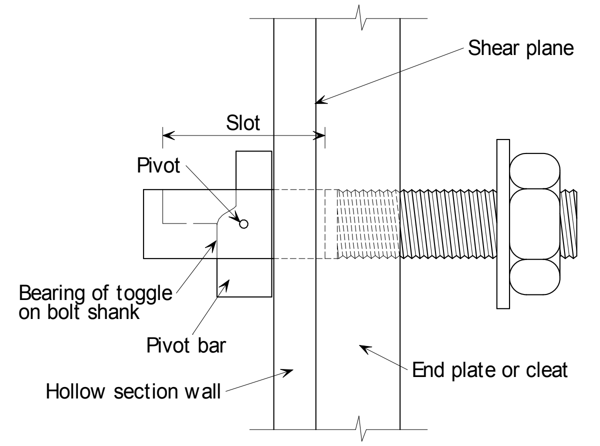


Figure F.2 Installation of a blind bolt in a hollow section

The bolts have grooves machined along the length of the shaft, so that a U-shaped gauge can be inserted to verify that the toggle is correctly positioned. Blind bolts can be removed and the components demounted.

F.3 MATERIAL

Blind bolts are manufactured in property class 10.9 material (yield strength of 900 N/mm^2, ultimate tensile strength of 1000 N/mm^2) and are available in sizes from M8 to M24. The bolts are available with a zinc or yellow finish, or in stainless steel.

F.4 BOLT RESISTANCES

The shear and tensile resistances of blind bolts are affected by the presence of the slot, so normal resistances cannot be assumed. A series of tests was undertaken, and design resistances determined by the Steel Construiction Institute.[44] covering shear, tension and combined resistance values.

F.5 FURTHER INFORMATION

Further information on blind bolts can be obtained from:

Blind Bolt Company Ltd
The Woodhouse
Hopton Wafers
Kidderminster
Worcs
DY14 0EE
Telephone +44 (07766) 003 165

www.blindbolt.co.uk

For resistance tables for the Blind Bolt, see Tables G.62, and G.63.

APPENDIX G RESISTANCE TABLES

MATERIAL STRENGTHS

FASTENER RESISTANCES

DIMENSIONS FOR DETAILING

SECTION DIMENSIONS AND PROPERTIES

LIST OF TABLES

LIST OF TABLES

Table G.1

Explanatory notes – PARTIAL DEPTH END PLATES
Use of Resistance Tables

The check numbers refer to those listed in Table G.3 and described in Section 4.5 Design procedures.

The resistance tables are based on the standard details given in Table G.2.

1 SHEAR RESISTANCE OF THE BEAM

The value given in { } below each beam designation is the shear resistance of the beam, given by $\frac{A_v f_y}{\sqrt{3}\gamma_{M0}}$.

A + symbol adjacent to the beam designation indicates that the serial size is additional to those specified in BS 4-1.

2 SHEAR RESISTANCE OF THE CONNECTION

This is the critical value of the design checks for the 'supported beam side' of the connection, i.e. the minimum resistance from Checks 2, 4, 5 (if applicable) 8 and 9.

For connections with Ordinary or Flowdrill bolts, connection shear resistances are given for un-notched, single notched and double notched beams.

For connections with Hollo-Bolts, connection shear resistances are tabulated for un-notched beams only, as Hollo-Bolts are generally only used with un-notched beams.

3 CRITICAL DESIGN CHECK

The check which gives the critical value of shear resistance. See Table G.3 for the description of the checks.

4 NOTCH LIMIT

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 resistance of the connection. The notch length is measured from the end of the beam. The calculated resistances allow for the thickness of the end plate.

It is assumed that the beam is fully restrained against lateral torsional buckling, and the limiting notch lengths are derived from Check 5.

To provide a simple check for double notched beams, it has been assumed that the remaining web depth is the same as the end plate length.

5 MINIMUM SUPPORT THICKNESS

This is the minimum thickness of supporting column or beam element that is needed to carry the given shear resistance (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.

If the applied shear force is less than the quoted resistance, the minimum support thickness reduces proportionally.

6 TYING RESISTANCE

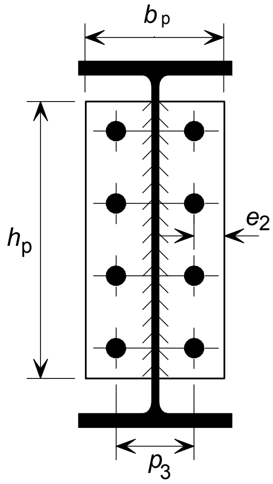
This is the critical value of the design checks for the 'supported beam' side of the connection, i.e. the minimum resistance from Checks 11, 12, and 13. The calculations assume that no washer is present and d_w is taken as the width across points of the bolt head.

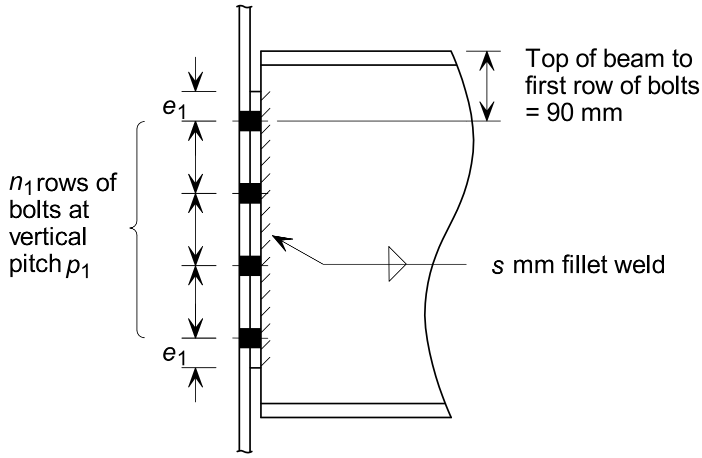
Separate checks will have to be carried out on the supporting members (see Checks 14 and 15).

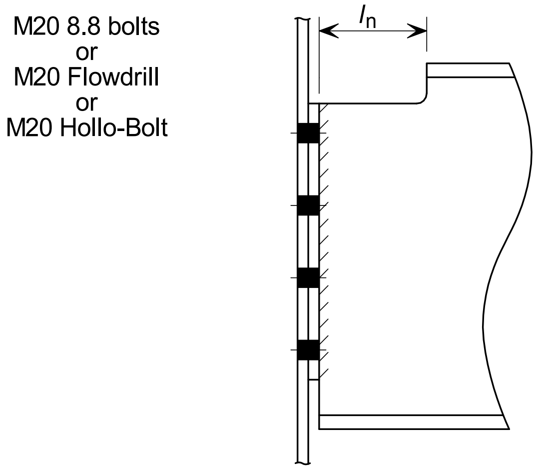
Table G.2

PARTIAL DEPTH END PLATES
Standard details used in Resistance Tables

Partial depth end plates







Ordinary and Flowdrill bolts:
- p_1 = 70 mm
- p_3 = 90 mm for beams ≤ 457 x 191 UB
- p_3 = 140 mm for beams > 457 x 191 UB
- e_1 = 40 mm; e_2 = 30 mm

Hollo-Bolts:
- p_1 = 80 mm
- p_3 = 90 mm for beams ≤ 457 x 191 UB
- p_3 = 110 mm for beams > 457 x 191 UB
- e_1 = 45 mm; e_2 = 45 mm

Table G.3

PARTIAL DEPTH END PLATES
Design Check List

		Check Number	Description
SHEAR RESISTANCE	Supported beam side	2	Welds
		4	Web in shear
		5	Resistance at a notch
		8	Bolt group
	Supporting member side	9	End plate
		10	Shear and bearing
TYING RESISTANCE		11	Plate and bolts
		12	Supported beam web
		13	Welds

Note: This table only lists the critical checks.
For a full list of design checks and further information, see Section 4.5.

BEAM: S275
END PLATES: S275
BOLTS: M20, 8.8

Table G.4

PARTIAL DEPTH END PLATES, ORDINARY or FLOWDRILL BOLTS
200 x 12 or 150 x 10 mm End Plate

Beam size {$V_{Rd,beam}$}	Bolt rows n_1	Un-notched Shear Resist V_{Rd} kN	Un-notched Crit chck	Single notch Shear Resist V_{Rd} kN	Single notch Max length l_n mm	Double notch Shear Resist V_{Rd} kN	Double notch Max length l_n mm	Fitting (End Plate) Width b_p mm	Fitting (End Plate) Thck t_p mm	Fitting (End Plate) Height h_p mm	Fitting (End Plate) Gauge p_3 mm	Fillet weld Leg s mm	Min support thickness S275 mm	Min support thickness S355 mm	Tying Resist $N_{Rd,u}$ kN	Tying Crit check
1016x305x487+	11	1660	8	1660	993	1660	563	200	12	780	140	18	5.7	4.9	1260	11
{4930 kN}	10	1510	8	1510	1090	1510	621	200	12	710	140	18	5.7	4.9	1150	11
	9	1350	8	1350	1220	1350	691	200	12	640	140	18	5.7	4.9	1040	11
	8	1200	8	1200	1370	1200	779	200	12	570	140	18	5.7	4.9	922	11
1016x305x437+	11	1660	8	1660	887	1660	503	200	12	780	140	18	5.7	4.9	1160	11
{4420 kN}	10	1510	8	1510	976	1510	558	200	12	710	140	18	5.7	4.9	1060	11
	9	1350	8	1350	1090	1350	621	200	12	640	140	18	5.7	4.9	952	11
	8	1200	8	1200	1220	1200	700	200	12	570	140	18	5.7	4.9	848	11
1016x305x393+	11	1660	8	1660	798	1660	444	200	12	780	140	15	5.7	4.9	1080	11
{3990 kN}	10	1510	8	1510	879	1510	504	200	12	710	140	15	5.7	4.9	984	11
	9	1350	8	1350	977	1350	565	200	12	640	140	15	5.7	4.9	887	11
	8	1200	8	1200	1100	1200	637	200	12	570	140	15	5.7	4.9	790	11
1016x305x349+	11	1660	8	1660	718	1660	371	200	12	780	140	12	5.7	4.9	1000	11
{3610 kN}	10	1510	8	1510	792	1510	437	200	12	710	140	12	5.7	4.9	913	11
	9	1350	8	1350	881	1350	504	200	12	640	140	12	5.7	4.9	823	11
	8	1200	8	1200	993	1200	571	200	12	570	140	12	5.7	4.9	733	11
1016x305x314+	11	1660	8	1660	632	1660	294	200	12	780	140	12	5.7	4.9	981	11
{3260 kN}	10	1510	8	1510	709	1510	367	200	12	710	140	12	5.7	4.9	893	11
	9	1350	8	1350	791	1350	441	200	12	640	140	12	5.7	4.9	805	11
	8	1200	8	1200	891	1200	514	200	12	570	140	12	5.7	4.9	717	11
1016x305x272+	11	1660	8	1660	492	1660	162	200	12	780	140	10	5.7	4.9	923	11
{2830 kN}	10	1510	8	1510	581	1510	247	200	12	710	140	10	5.7	4.9	840	11
	9	1350	8	1350	670	1350	332	200	12	640	140	10	5.7	4.9	757	11
	8	1200	8	1200	761	1200	417	200	12	570	140	10	5.7	4.9	675	11
1016x305x249+	11	1660	8	1660	471	1660	167	200	12	780	140	10	5.7	4.9	923	11
{2770 kN}	10	1510	8	1510	560	1510	252	200	12	710	140	10	5.7	4.9	840	11
	9	1350	8	1350	648	1350	337	200	12	640	140	10	5.7	4.9	757	11
	8	1200	8	1200	737	1200	422	200	12	570	140	10	5.7	4.9	675	11
1016x305x222+	11	1660	8	1660	418	1660	135	200	12	780	140	10	5.7	4.9	918	11
{2640 kN}	10	1510	8	1510	508	1510	223	200	12	710	140	10	5.7	4.9	836	11
	9	1350	8	1350	598	1350	311	200	12	640	140	10	5.7	4.9	754	11
	8	1200	8	1200	687	1200	398	200	12	570	140	10	5.7	4.9	671	11

For guidance on the use of tables see Explanatory notes in Table G.1

BEAM: S275
END PLATES: S275
Table G.4 *Continued* **BOLTS: M20, 8.8**

PARTIAL DEPTH END PLATES, ORDINARY or FLOWDRILL BOLTS
200 x 12 or 150 x 10 mm End Plate

Beam size	Bolt rows	Un-notched		Single notch		Double notch		Fitting (End Plate)				Fillet weld	Min support thickness		Tying	
		Shear Resist	Crit chck	Shear Resist	Max length	Shear Resist	Max length	Width	Thck	Height	Gauge	Leg	S275	S355	Resist	Crit check
$\{V_{Rd,beam}\}$	n_1	V_{Rd}		V_{Rd}	l_n	V_{Rd}	l_n	b_p	t_p	h_p	p_3	s			$N_{Rd,u}$	
		kN		kN	mm	kN	mm	mm	mm	mm	mm	mm	mm	mm	kN	
914x419x388	11	1660	8	1660	631	1660	290	200	12	780	140	15	5.7	4.9	1040	11
{3240 kN}	10	1510	8	1510	708	1510	355	200	12	710	140	15	5.7	4.9	950	11
	9	1350	8	1350	788	1350	421	200	12	640	140	15	5.7	4.9	856	11
	8	1200	8	1200	888	1200	486	200	12	570	140	15	5.7	4.9	763	11
914x419x343	11	1660	8	1660	537	1660	214	200	12	780	140	12	5.7	4.9	984	11
{2920 kN}	10	1510	8	1510	620	1510	286	200	12	710	140	12	5.7	4.9	896	11
	9	1350	8	1350	704	1350	358	200	12	640	140	12	5.7	4.9	808	11
	8	1200	8	1200	794	1200	431	200	12	570	140	12	5.7	4.9	719	11
914x305x289	11	1660	8	1660	542	1660	252	200	12	780	140	12	5.7	4.9	985	11
{2900 kN}	10	1510	8	1510	623	1510	324	200	12	710	140	12	5.7	4.9	897	11
	9	1350	8	1350	705	1350	396	200	12	640	140	12	5.7	4.9	809	11
	8	1200	8	1200	795	1200	468	200	12	570	140	12	5.7	4.9	720	11
914x305x253	11	1660	8	1660	420	1660	146	200	12	780	140	10	5.7	4.9	931	11
{2570 kN}	10	1510	8	1510	511	1510	227	200	12	710	140	10	5.7	4.9	847	11
	9	1350	8	1350	603	1350	308	200	12	640	140	10	5.7	4.9	764	11
	8	1200	8	1200	694	1200	389	200	12	570	140	10	5.7	4.9	680	11
914x305x224	11	1660	8	1660	317	1580	100	200	12	780	140	10	5.7	4.9	917	11
{2350 kN}	10	1510	8	1510	415	1510	146	200	12	710	140	10	5.7	4.9	835	11
	9	1350	8	1350	514	1350	234	200	12	640	140	10	5.7	4.9	753	11
	8	1200	8	1200	612	1200	322	200	12	570	140	10	5.7	4.9	670	11
914x305x201	11	1620	4	1620	266	1490	100	200	12	780	140	10	5.5	4.8	910	11
{2210 kN}	10	1480	4	1480	365	1480	107	200	12	710	140	10	5.6	4.8	828	11
	9	1330	4	1330	464	1330	197	200	12	640	140	10	5.6	4.9	747	11
	8	1190	4	1190	563	1190	287	200	12	570	140	10	5.6	4.9	665	11
838x292x226	10	1510	8	1510	336	1470	100	200	12	710	140	10	5.7	4.9	837	11
{2220 kN}	9	1350	8	1350	435	1350	167	200	12	640	140	10	5.7	4.9	754	11
	8	1200	8	1200	534	1200	254	200	12	570	140	10	5.7	4.9	672	11
838x292x194	10	1440	4	1440	270	1320	100	200	12	710	140	10	5.4	4.7	825	11
{2000 kN}	9	1300	4	1300	371	1300	117	200	12	640	140	10	5.4	4.7	744	11
	8	1150	4	1150	472	1150	207	200	12	570	140	10	5.4	4.7	662	11
838x292x176	10	1370	4	1370	257	1250	100	200	12	710	140	8	5.1	4.5	794	11
{1890 kN}	9	1230	4	1230	357	1230	110	200	12	640	140	8	5.2	4.5	716	11
	8	1100	4	1100	456	1100	200	200	12	570	140	8	5.2	4.5	637	11

For guidance on the use of tables see Explanatory notes in Table G.1

BEAM: S275
END PLATES: S275
BOLTS: M20, 8.8

Table G.4 *Continued*

PARTIAL DEPTH END PLATES, ORDINARY or FLOWDRILL BOLTS
200 x 12 or 150 x 10 mm End Plate

Beam size	Bolt rows	Un-notched		Single notch		Double notch		Fitting (End Plate)				Fillet weld	Min support thickness		Tying	
Beam size $\{V_{Rd,beam}\}$	n_1	Shear Resist V_{Rd} kN	Crit chck	Shear Resist V_{Rd} kN	Max length l_n mm	Shear Resist V_{Rd} kN	Max length l_n mm	Width b_p mm	Thck t_p mm	Height h_p mm	Gauge p_3 mm	Leg s mm	S275 mm	S355 mm	Resist $N_{Rd,u}$ kN	Crit check
762x267x197	9	1350	8	1350	270	1250	100	200	12	640	140	10	5.7	4.9	751	11
{1950 kN}	8	1200	8	1200	372	1200	128	200	12	570	140	10	5.7	4.9	668	11
	7	1050	8	1050	474	1050	218	200	12	500	140	10	5.7	4.9	586	11
762x267x173	9	1260	4	1260	242	1130	100	200	12	640	140	10	5.3	4.6	741	11
{1760 kN}	8	1120	4	1120	343	1120	106	200	12	570	140	10	5.3	4.6	660	11
	7	985	4	985	444	985	196	200	12	500	140	10	5.3	4.6	579	11
762x267x147	9	1130	4	1130	227	999	100	200	12	640	140	8	4.7	4.1	708	11
{1560 kN}	8	1000	4	1000	327	999	100	200	12	570	140	8	4.7	4.1	630	11
	7	881	4	881	426	881	186	200	12	500	140	8	4.7	4.1	553	11
762x267x134	9	1100	4	1100	219	965	100	200	12	640	140	8	4.6	4.0	702	11
{1520 kN}	8	977	4	977	318	965	100	200	12	570	140	8	4.6	4.0	625	11
	7	857	4	857	417	857	180	200	12	500	140	8	4.6	4.0	549	11
686x254x170	8	1140	4	1140	233	1010	100	200	12	570	140	10	5.4	4.7	661	11
{1630 kN}	7	998	4	998	335	998	107	200	12	500	140	10	5.4	4.7	580	11
	6	859	4	859	438	859	197	200	12	430	140	10	5.4	4.7	499	11
686x254x152	8	1040	4	1040	225	910	100	200	12	570	140	8	4.9	4.2	633	11
{1470 kN}	7	909	4	909	327	909	100	200	12	500	140	8	4.9	4.3	555	11
	6	782	4	782	430	782	190	200	12	430	140	8	4.9	4.3	477	11
686x254x140	8	973	4	973	218	848	100	200	12	570	140	8	4.6	4.0	628	11
{1370 kN}	7	854	4	854	320	848	100	200	12	500	140	8	4.6	4.0	551	11
	6	734	4	734	422	734	185	200	12	430	140	8	4.6	4.0	474	11
686x254x125	8	918	4	918	206	791	100	200	12	570	140	8	4.3	3.8	624	11
{1280 kN}	7	806	4	806	307	791	100	200	12	500	140	8	4.3	3.8	547	11
	6	693	4	693	407	693	178	200	12	430	140	8	4.3	3.8	470	11
610x305x238	7	1050	8	1050	377	1050	142	200	12	500	140	12	5.7	4.9	624	11
{1890 kN}	6	903	8	903	467	903	218	200	12	430	140	12	5.7	4.9	537	11
610x305x179	7	971	4	971	229	841	100	200	12	500	140	8	5.2	4.6	560	11
{1440 kN}	6	835	4	835	335	835	104	200	12	430	140	8	5.2	4.6	481	11
610x305x149	7	812	4	812	221	691	100	200	12	500	140	8	4.4	3.8	548	11
{1200 kN}	6	699	4	699	325	691	100	200	12	430	140	8	4.4	3.8	471	11

For guidance on the use of tables see Explanatory notes in Table G.1

BEAM: S275
END PLATES: S275
Table G.4 *Continued* **BOLTS: M20, 8.8**

PARTIAL DEPTH END PLATES, ORDINARY or FLOWDRILL BOLTS
200 x 12 or 150 x 10 mm End Plate

Beam size {$V_{Rd,beam}$}	Bolt rows	Un-notched		Single notch		Double notch		Fitting (End Plate)				Fillet weld	Min support thickness		Tying	
	n_1	Shear Resist V_{Rd} kN	Crit chck	Shear Resist V_{Rd} kN	Max length l_n mm	Shear Resist V_{Rd} kN	Max length l_n mm	Width b_p mm	Thck t_p mm	Height h_p mm	Gauge p_3 mm	Leg s mm	S275 mm	S355 mm	Resist $N_{Rd,u}$ kN	Crit check
610x229x140	7	902	4	902	208	776	100	200	12	500	140	8	4.8	4.2	554	11
{1300 kN}	6	776	4	776	312	776	100	200	12	430	140	8	4.9	4.2	477	11
610x229x125	7	819	4	819	201	697	100	200	12	500	140	8	4.4	3.8	548	11
{1170 kN}	6	705	4	705	305	697	100	200	12	430	140	8	4.4	3.9	471	11
610x229x113	7	764	4	764	193	643	100	200	12	500	140	8	4.1	3.6	544	11
{1090 kN}	6	657	4	657	296	643	100	200	12	430	140	8	4.1	3.6	468	11
610x229x101	7	750	4	750	182	624	100	200	12	500	140	6	4.0	3.5	525	11
{1060 kN}	6	645	4	645	284	624	100	200	12	430	140	6	4.0	3.5	452	11
610x178x100+	7	778	4	778	183	654	100	200	12	500	140	8	4.2	3.6	545	11
{1110 kN}	6	669	4	669	281	654	100	200	12	430	140	8	4.2	3.7	469	11
610x178x92+	7	779	4	779	173	648	100	200	12	500	140	8	4.2	3.7	543	11
{1090 kN}	6	670	4	670	270	648	100	200	12	430	140	8	4.2	3.7	467	11
610x178x82+	7	714	4	714	165	N/A	100	200	12	500	140	6	3.8	3.3	523	11
{1000 kN}	6	614	4	614	261	588	100	200	12	430	140	6	3.9	3.4	450	11
533x312x272+	6	903	8	903	438	903	202	200	12	430	140	12	5.7	4.9	553	11
{1910 kN}	5	753	8	753	529	753	268	200	12	360	140	12	5.7	4.9	463	11
533x312x219+	6	903	8	903	334	903	119	200	12	430	140	12	5.7	4.9	536	11
{1630 kN}	5	753	8	753	428	753	196	200	12	360	140	12	5.7	4.9	449	11
533x312x182+	6	900	4	900	209	762	100	200	12	430	140	10	5.6	4.9	502	11
{1340 kN}	5	753	8	753	320	753	106	200	12	360	140	10	5.7	4.9	420	11
533x312x150+	6	752	4	752	199	623	100	200	12	430	140	8	4.7	4.1	475	11
{1120 kN}	5	630	4	630	309	623	100	200	12	360	140	8	4.7	4.1	398	11

For guidance on the use of tables see Explanatory notes in Table G.1

BEAM: S275
END PLATES: S275
BOLTS: M20, 8.8

Table G.4 *Continued*

PARTIAL DEPTH END PLATES, ORDINARY or FLOWDRILL BOLTS
200 x 12 or 150 x 10 mm End Plate

Beam size {$V_{Rd,beam}$}	Bolt rows n_1	Un-notched Shear Resist V_{Rd} kN	Un-notched Crit chck	Single notch Shear Resist V_{Rd} kN	Single notch Max length l_n mm	Double notch Shear Resist V_{Rd} kN	Double notch Max length l_n mm	Fitting (End Plate) Width b_p mm	Fitting (End Plate) Thck t_p mm	Fitting (End Plate) Height h_p mm	Fitting (End Plate) Gauge p_3 mm	Fillet weld Leg s mm	Min support thickness S275 mm	Min support thickness S355 mm	Tying Resist $N_{Rd,u}$ kN	Tying Crit check
533x210x138+	6	870	4	870	194	733	100	200	12	430	140	10	5.5	4.8	500	11
{1290 kN}	5	729	4	729	298	729	102	200	12	360	140	10	5.5	4.8	418	11
533x210x122	6	752	4	752	191	625	100	200	12	430	140	8	4.7	4.1	475	11
{1110 kN}	5	630	4	630	295	625	100	200	12	360	140	8	4.7	4.1	398	11
533x210x109	6	687	4	687	184	563	100	200	12	430	140	8	4.3	3.8	470	11
{1020 kN}	5	575	4	575	287	563	100	200	12	360	140	8	4.3	3.8	394	11
533x210x101	6	639	4	639	180	520	100	200	12	430	140	8	4.0	3.5	466	11
{952 kN}	5	535	4	535	283	520	100	200	12	360	140	8	4.0	3.5	391	11
533x210x92	6	621	4	621	174	500	100	200	12	430	140	6	3.9	3.4	450	11
{909 kN}	5	520	4	520	276	500	100	200	12	360	140	6	3.9	3.4	377	11
533x210x82	6	590	4	590	163	N/A	100	200	12	430	140	6	3.7	3.2	448	11
{865 kN}	5	494	4	494	264	468	100	200	12	360	140	6	3.7	3.2	375	11
533x165x85+	6	610	4	610	171	494	100	150	10	430	90	6	3.8	3.3	572	11
{902 kN}	5	511	4	511	270	494	100	150	10	360	90	6	3.8	3.4	479	11
533x165x74+	6	596	4	596	160	N/A	100	150	10	430	90	6	3.7	3.3	566	11
{871 kN}	5	499	4	499	258	477	100	150	10	360	90	6	3.8	3.3	474	11
533x165x66+	6	547	4	547	152	N/A	100	150	10	430	90	6	3.4	3.0	559	11
{793 kN}	5	458	4	458	248	432	100	150	10	360	90	6	3.4	3.0	468	11

For guidance on the use of tables see Explanatory notes in Table G.1

BEAM: S275
END PLATES: S275
Table G.4 *Continued* **BOLTS: M20, 8.8**

PARTIAL DEPTH END PLATES, ORDINARY or FLOWDRILL BOLTS
200 x 12 or 150 x 10 mm End Plate

Beam size $\{V_{Rd,beam}\}$	Bolt rows n_1	Un-notched Shear Resist V_{Rd} kN	Un-notched Crit chck	Single notch Shear Resist V_{Rd} kN	Single notch Max length l_n mm	Double notch Shear Resist V_{Rd} kN	Double notch Max length l_n mm	Fitting (End Plate) Width b_p mm	Fitting (End Plate) Thck t_p mm	Fitting (End Plate) Height h_p mm	Fitting (End Plate) Gauge p_3 mm	Fillet weld Leg s mm	Min support thickness S275 mm	Min support thickness S355 mm	Tying Resist $N_{Rd,u}$ kN	Tying Crit check
457x191x161+	5	753	8	753	285	753	103	150	10	360	90	12	5.7	4.9	670	11
{1390 kN}	4	602	8	602	378	602	181	150	10	290	90	12	5.7	4.9	540	11
457x191x133+	5	753	8	753	187	622	100	150	10	360	90	10	5.7	4.9	590	11
{1160 kN}	4	602	8	602	295	602	112	150	10	290	90	10	5.7	4.9	475	11
457x191x106+	5	625	4	625	166	492	100	150	10	360	90	8	4.7	4.1	526	11
{947 kN}	4	503	4	503	272	492	100	150	10	290	90	8	4.7	4.1	424	11
457x191x98	5	565	4	565	166	442	100	150	10	360	90	8	4.3	3.7	515	11
{852 kN}	4	455	4	455	272	442	100	150	10	290	90	8	4.3	3.7	415	11
457x191x89	5	520	4	520	161	402	100	150	10	360	90	6	3.9	3.4	480	11
{789 kN}	4	419	4	419	266	402	100	150	10	290	90	6	3.9	3.4	387	11
457x191x82	5	509	4	509	155	388	100	150	10	360	90	6	3.8	3.3	476	11
{756 kN}	4	410	4	410	260	388	100	150	10	290	90	6	3.9	3.4	383	11
457x191x74	5	463	4	463	151	N/A	100	150	10	360	90	6	3.5	3.0	469	11
{693 kN}	4	373	4	373	256	349	100	150	10	290	90	6	3.5	3.1	377	11
457x191x67	5	437	4	437	144	N/A	100	150	10	360	90	6	3.3	2.9	465	11
{650 kN}	4	352	4	352	248	326	100	150	10	290	90	6	3.3	2.9	374	11
457x152x82	5	520	4	520	161	405	100	150	10	360	90	6	3.9	3.4	480	11
{798 kN}	4	419	4	419	263	405	100	150	10	290	90	6	3.9	3.4	387	11
457x152x74	5	476	4	476	156	366	100	150	10	360	90	6	3.6	3.1	473	11
{721 kN}	4	383	4	383	258	366	100	150	10	290	90	6	3.6	3.1	381	11
457x152x67	5	463	4	463	148	N/A	100	150	10	360	90	6	3.5	3.0	469	11
{697 kN}	4	373	4	373	250	351	100	150	10	290	90	6	3.5	3.1	377	11
457x152x60	5	417	4	417	144	N/A	100	150	10	360	90	6	3.1	2.7	462	11
{624 kN}	4	336	4	336	245	312	100	150	10	290	90	6	3.2	2.8	372	11
457x152x52	5	391	4	391	134	N/A	100	150	10	360	90	6	2.9	2.6	458	11
{578 kN}	4	315	4	315	233	287	100	150	10	290	90	6	3.0	2.6	369	11

For guidance on the use of tables see Explanatory notes in Table G.1

BEAM: S275
END PLATES: S275
BOLTS: M20, 8.8

Table G.4 *Continued*

PARTIAL DEPTH END PLATES, ORDINARY or FLOWDRILL BOLTS
200 x 12 or 150 x 10 mm End Plate

Beam size	Bolt rows	Un-notched		Single notch		Double notch		Fitting (End Plate)				Fillet weld	Min support thickness		Tying	
Beam size $\{V_{Rd,beam}\}$	n_1	Shear Resist V_{Rd} kN	Crit chck	Shear Resist V_{Rd} kN	Max length l_n mm	Shear Resist V_{Rd} kN	Max length l_n mm	Width b_p mm	Thck t_p mm	Height h_p mm	Gauge p_3 mm	Leg s mm	S275 mm	S355 mm	Resist $N_{Rd,u}$ kN	Crit check
406x178x85+ {742 kN}	4	435	4	435	187	348	100	150	10	290	90	8	4.1	3.6	411	11
406x178x74 {664 kN}	4	394	4	394	182	309	100	150	10	290	90	6	3.7	3.2	381	11
406x178x67 {612 kN}	4	365	4	365	177	282	100	150	10	290	90	6	3.4	3.0	376	11
406x178x60 {549 kN}	4	327	4	327	173	250	100	150	10	290	90	6	3.1	2.7	371	11
406x178x54 {529 kN}	4	319	4	319	163	239	100	150	10	290	90	6	3.0	2.6	370	11
406x140x53+ {529 kN}	4	327	4	327	168	278	100	150	10	290	90	6	3.1	2.7	371	11
406x140x46 {473 kN}	4	282	4	282	164	236	100	150	10	290	90	6	2.6	2.3	365	11
406x140x39 {438 kN}	4	265	4	265	151	217	100	150	10	290	90	6	2.5	2.2	362	11
356x171x67 {568 kN}	3	286	4	286	206	264	100	150	10	220	90	6	3.6	3.1	287	11
356x171x57 {501 kN}	3	255	4	255	196	229	100	150	10	220	90	6	3.2	2.8	282	11
356x171x51 {455 kN}	3	233	4	233	191	206	100	150	10	220	90	6	2.9	2.5	279	11
356x171x45 {425 kN}	3	220	4	220	183	191	100	150	10	220	90	6	2.8	2.4	277	11
356x127x39 {408 kN}	3	207	4	207	182	182	100	150	10	220	90	6	2.6	2.3	276	11
356x127x33 {366 kN}	3	189	4	189	172	162	100	150	10	220	90	6	2.4	2.1	273	11

For guidance on the use of tables see Explanatory notes in Table G.1

BEAM: S275
END PLATES: S275
BOLTS: M20, 8.8

Table G.4 *Continued*

PARTIAL DEPTH END PLATES, ORDINARY or FLOWDRILL BOLTS
200 x 12 or 150 x 10 mm End Plate

Beam size	Bolt rows	Un-notched		Single notch		Double notch		Fitting (End Plate)				Fillet weld	Min support thickness		Tying	
{$V_{Rd,beam}$}	n_1	Shear Resist V_{Rd} kN	Crit chck	Shear Resist V_{Rd} kN	Max length l_n mm	Shear Resist V_{Rd} kN	Max length l_n mm	Width b_p mm	Thck t_p mm	Height h_p mm	Gauge p_3 mm	Leg s mm	S275 mm	S355 mm	Resist $N_{Rd,u}$ kN	Crit check
305x165x54 {422 kN}	3	248	4	248	116	169	100	150	10	220	90	6	3.1	2.7	281	11
305x165x46 {357 kN}	3	211	4	211	113	140	100	150	10	220	90	6	2.6	2.3	276	11
305x165x40 {319 kN}	3	189	4	189	108	123	100	150	10	220	90	6	2.4	2.1	273	11
305x127x48 {474 kN}	3	283	4	283	109	194	100	150	10	220	90	6	3.5	3.1	286	11
305x127x42 {420 kN}	3	251	4	251	103	168	100	150	10	220	90	6	3.2	2.8	282	11
305x127x37 {372 kN}	3	223	4	223	100	146	100	150	10	220	90	6	2.8	2.4	278	11
305x102x33 {350 kN}	3	207	4	207	110	144	100	150	10	220	90	6	2.6	2.3	276	11
305x102x28 {315 kN}	3	189	4	189	102	127	100	150	10	220	90	6	2.4	2.1	273	11
305x102x25 {299 kN}	3	182	4	179	100	120	100	150	10	220	90	6	2.3	2.0	272	11
254x146x43 {321 kN}	2	154	4	154	134	102	100	150	10	150	90	6	2.9	2.5	190	11
254x146x37 {280 kN}	2	135	4	135	129	86	100	150	10	150	90	6	2.5	2.2	187	11
254x146x31 {260 kN}	2	129	4	129	119	78	100	150	10	150	90	6	2.4	2.1	186	11
254x102x28 {283 kN}	2	135	4	135	127	90	100	150	10	150	90	6	2.5	2.2	187	11
254x102x25 {265 kN}	2	129	4	129	121	83	100	150	10	150	90	6	2.4	2.1	186	11
254x102x22 {248 kN}	2	122	4	122	113	76	100	150	10	150	90	6	2.3	2.0	185	11

For guidance on the use of tables see Explanatory notes in Table G.1

BEAM: S355
END PLATES: S275
BOLTS: M20, 8.8

Table G.5

PARTIAL DEPTH END PLATES, ORDINARY or FLOWDRILL BOLTS
200 x 12 or 150 x 10 mm End Plate

	Bolt rows	Un-notched		Single notch		Double notch		Fitting (End Plate)				Fillet weld	Min support thickness		Tying	
Beam size {$V_{Rd,beam}$}	n_1	Shear Resist V_{Rd} kN	Crit chck	Shear Resist V_{Rd} kN	Max length l_n mm	Shear Resist V_{Rd} kN	Max length l_n mm	Width b_p mm	Thck t_p mm	Height h_p mm	Gauge p_3 mm	Leg s mm	S275 mm	S355 mm	Resist $N_{Rd,u}$ kN	Crit check
1016x305x487+	11	1660	8	1660	1310	1660	744	200	12	780	140	22	5.7	4.9	1390	11
{6480 kN}	10	1510	8	1510	1440	1510	819	200	12	710	140	22	5.7	4.9	1260	11
	9	1350	8	1350	1600	1350	912	200	12	640	140	22	5.7	4.9	1140	11
	8	1200	8	1200	1800	1200	1030	200	12	570	140	22	5.7	4.9	1010	11
1016x305x437+	11	1660	8	1660	1170	1660	668	200	12	780	140	20	5.7	4.9	1260	11
{5810 kN}	10	1510	8	1510	1290	1510	736	200	12	710	140	20	5.7	4.9	1150	11
	9	1350	8	1350	1430	1350	820	200	12	640	140	20	5.7	4.9	1040	11
	8	1200	8	1200	1610	1200	924	200	12	570	140	20	5.7	4.9	924	11
1016x305x393+	11	1660	8	1660	1050	1660	608	200	12	780	140	18	5.7	4.9	1170	11
{5240 kN}	10	1510	8	1510	1160	1510	670	200	12	710	140	18	5.7	4.9	1070	11
	9	1350	8	1350	1290	1350	745	200	12	640	140	18	5.7	4.9	960	11
	8	1200	8	1200	1450	1200	840	200	12	570	140	18	5.7	4.9	855	11
1016x305x349+	11	1660	8	1660	939	1660	540	200	12	780	140	15	5.7	4.9	1080	11
{4700 kN}	10	1510	8	1510	1030	1510	595	200	12	710	140	15	5.7	4.9	983	11
	9	1350	8	1350	1150	1350	663	200	12	640	140	15	5.7	4.9	886	11
	8	1200	8	1200	1300	1200	747	200	12	570	140	15	5.7	4.9	789	11
1016x305x314+	11	1660	8	1660	843	1660	481	200	12	780	140	15	5.7	4.9	1020	11
{4240 kN}	10	1510	8	1510	929	1510	537	200	12	710	140	15	5.7	4.9	925	11
	9	1350	8	1350	1030	1350	599	200	12	640	140	15	5.7	4.9	834	11
	8	1200	8	1200	1160	1200	675	200	12	570	140	15	5.7	4.9	743	11
1016x305x272+	11	1660	8	1660	720	1660	378	200	12	780	140	12	5.7	4.9	954	11
{3680 kN}	10	1510	8	1510	793	1510	444	200	12	710	140	12	5.7	4.9	869	11
	9	1350	8	1350	883	1350	509	200	12	640	140	12	5.7	4.9	783	11
	8	1200	8	1200	994	1200	576	200	12	570	140	12	5.7	4.9	697	11
1016x305x249+	11	1660	8	1660	697	1660	384	200	12	780	140	12	5.7	4.9	954	11
{3600 kN}	10	1510	8	1510	768	1510	449	200	12	710	140	12	5.7	4.9	869	11
	9	1350	8	1350	855	1350	514	200	12	640	140	12	5.7	4.9	783	11
	8	1200	8	1200	964	1200	582	200	12	570	140	12	5.7	4.9	697	11
1016x305x222+	11	1660	8	1660	647	1660	359	200	12	780	140	12	5.7	4.9	949	11
{3430 kN}	10	1510	8	1510	716	1510	426	200	12	710	140	12	5.7	4.9	864	11
	9	1350	8	1350	797	1350	493	200	12	640	140	12	5.7	4.9	779	11
	8	1200	8	1200	899	1200	561	200	12	570	140	12	5.7	4.9	694	11

For guidance on the use of tables see Explanatory notes in Table G.1

BEAM: S355
END PLATES: S275
BOLTS: M20, 8.8

Table G.5 *Continued*

PARTIAL DEPTH END PLATES, ORDINARY or FLOWDRILL BOLTS
200 x 12 or 150 x 10 mm End Plate

Beam size	Bolt rows	Un-notched		Single notch		Double notch		Fitting (End Plate)				Fillet weld	Min support thickness		Tying	
Beam size {$V_{Rd,beam}$}	n_1	Shear Resist V_{Rd} kN	Crit chck	Shear Resist V_{Rd} kN	Max length l_n mm	Shear Resist V_{Rd} kN	Max length l_n mm	Width b_p mm	Thck t_p mm	Height h_p mm	Gauge p_3 mm	Leg s mm	S275 mm	S355 mm	Resist $N_{Rd,u}$ kN	Crit check
914x419x388	11	1660	8	1660	840	1660	457	200	12	780	140	15	5.7	4.9	1080	11
{4220 kN}	10	1510	8	1510	926	1510	507	200	12	710	140	15	5.7	4.9	986	11
	9	1350	8	1350	1030	1350	565	200	12	640	140	15	5.7	4.9	889	11
	8	1200	8	1200	1160	1200	637	200	12	570	140	15	5.7	4.9	792	11
914x419x343	11	1660	8	1660	751	1660	398	200	12	780	140	15	5.7	4.9	1020	11
{3800 kN}	10	1510	8	1510	827	1510	454	200	12	710	140	15	5.7	4.9	928	11
	9	1350	8	1350	921	1350	509	200	12	640	140	15	5.7	4.9	837	11
	8	1200	8	1200	1040	1200	575	200	12	570	140	15	5.7	4.9	745	11
914x305x289	11	1660	8	1660	752	1660	436	200	12	780	140	15	5.7	4.9	1020	11
{3780 kN}	10	1510	8	1510	828	1510	491	200	12	710	140	15	5.7	4.9	929	11
	9	1350	8	1350	921	1350	548	200	12	640	140	15	5.7	4.9	838	11
	8	1200	8	1200	1040	1200	618	200	12	570	140	15	5.7	4.9	746	11
914x305x253	11	1660	8	1660	653	1660	353	200	12	780	140	12	5.7	4.9	962	11
{3350 kN}	10	1510	8	1510	724	1510	415	200	12	710	140	12	5.7	4.9	876	11
	9	1350	8	1350	805	1350	478	200	12	640	140	12	5.7	4.9	790	11
	8	1200	8	1200	908	1200	541	200	12	570	140	12	5.7	4.9	703	11
914x305x224	11	1660	8	1660	568	1660	282	200	12	780	140	12	5.7	4.9	948	11
{3060 kN}	10	1510	8	1510	644	1510	350	200	12	710	140	12	5.7	4.9	863	11
	9	1350	8	1350	720	1350	418	200	12	640	140	12	5.7	4.9	778	11
	8	1200	8	1200	812	1200	486	200	12	570	140	12	5.7	4.9	693	11
914x305x201	11	1660	8	1660	504	1660	233	200	12	780	140	12	5.7	4.9	940	11
{2870 kN}	10	1510	8	1510	582	1510	305	200	12	710	140	12	5.7	4.9	856	11
	9	1350	8	1350	661	1350	376	200	12	640	140	12	5.7	4.9	772	11
	8	1200	8	1200	746	1200	447	200	12	570	140	12	5.7	4.9	687	11
838x292x226	10	1510	8	1510	566	1510	282	200	12	710	140	12	5.7	4.9	865	11
{2900 kN}	9	1350	8	1350	642	1350	349	200	12	640	140	12	5.7	4.9	780	11
	8	1200	8	1200	724	1200	416	200	12	570	140	12	5.7	4.9	694	11
838x292x194	10	1510	8	1510	470	1510	205	200	12	710	140	10	5.7	4.9	825	11
{2610 kN}	9	1350	8	1350	552	1350	279	200	12	640	140	10	5.7	4.9	744	11
	8	1200	8	1200	634	1200	352	200	12	570	140	10	5.7	4.9	662	11
838x292x176	10	1510	8	1510	414	1510	161	200	12	710	140	10	5.7	4.9	819	11
{2460 kN}	9	1350	8	1350	499	1350	238	200	12	640	140	10	5.7	4.9	739	11
	8	1200	8	1200	585	1200	315	200	12	570	140	10	5.7	4.9	658	11

For guidance on the use of tables see Explanatory notes in Table G.1

BEAM: S355
END PLATES: S275
BOLTS: M20, 8.8

Table G.5 *Continued*

PARTIAL DEPTH END PLATES, ORDINARY or FLOWDRILL BOLTS
200 x 12 or 150 x 10 mm End Plate

Beam size	Bolt rows	Un-notched		Single notch		Double notch		Fitting (End Plate)				Fillet weld	Min support thickness		Tying	
$\{V_{Rd,beam}\}$	n_1	Shear Resist V_{Rd} kN	Crit chck	Shear Resist V_{Rd} kN	Max length l_n mm	Shear Resist V_{Rd} kN	Max length l_n mm	Width b_p mm	Thck t_p mm	Height h_p mm	Gauge p_3 mm	Leg s mm	S275 mm	S355 mm	Resist $N_{Rd,u}$ kN	Crit check
762x267x197	9	1350	8	1350	483	1350	226	200	12	640	140	12	5.7	4.9	776	11
{2530 kN}	8	1200	8	1200	561	1200	295	200	12	570	140	12	5.7	4.9	691	11
	7	1050	8	1050	644	1050	364	200	12	500	140	12	5.7	4.9	606	11
762x267x173	9	1350	8	1350	403	1350	160	200	12	640	140	10	5.7	4.9	741	11
{2290 kN}	8	1200	8	1200	488	1200	235	200	12	570	140	10	5.7	4.9	660	11
	7	1050	8	1050	573	1050	310	200	12	500	140	10	5.7	4.9	579	11
762x267x147	9	1350	8	1350	298	1300	100	200	12	640	140	10	5.7	4.9	730	11
{2040 kN}	8	1200	8	1200	391	1200	154	200	12	570	140	10	5.7	4.9	650	11
	7	1050	8	1050	485	1050	238	200	12	500	140	10	5.7	4.9	570	11
762x267x134	9	1350	8	1350	259	1250	100	200	12	640	140	10	5.7	4.9	724	11
{1970 kN}	8	1200	8	1200	355	1200	124	200	12	570	140	10	5.7	4.9	645	11
	7	1050	8	1050	451	1050	211	200	12	500	140	10	5.7	4.9	566	11
686x254x170	8	1200	8	1200	389	1200	154	200	12	570	140	10	5.7	4.9	661	11
{2120 kN}	7	1050	8	1050	475	1050	229	200	12	500	140	10	5.7	4.9	580	11
	6	903	8	903	562	903	303	200	12	430	140	10	5.7	4.9	499	11
686x254x152	8	1200	8	1200	314	1180	100	200	12	570	140	10	5.7	4.9	653	11
{1920 kN}	7	1050	8	1050	407	1050	170	200	12	500	140	10	5.7	4.9	572	11
	6	903	8	903	500	903	252	200	12	430	140	10	5.7	4.9	492	11
686x254x140	8	1200	8	1200	259	1100	100	200	12	570	140	10	5.7	4.9	647	11
{1790 kN}	7	1050	8	1050	358	1050	128	200	12	500	140	10	5.7	4.9	568	11
	6	903	8	903	456	903	215	200	12	430	140	10	5.7	4.9	488	11
686x254x125	8	1200	4	1200	206	1030	100	200	12	570	140	8	5.6	4.9	624	11
{1670 kN}	7	1050	4	1050	307	1030	100	200	12	500	140	8	5.6	4.9	547	11
	6	902	4	902	407	902	178	200	12	430	140	8	5.7	4.9	470	11
610x305x238	7	1050	8	1050	525	1050	266	200	12	500	140	15	5.7	4.9	647	11
{2460 kN}	6	903	8	903	614	903	324	200	12	430	140	15	5.7	4.9	556	11
610x305x179	7	1050	8	1050	354	1050	121	200	12	500	140	10	5.7	4.9	578	11
{1880 kN}	6	903	8	903	444	903	197	200	12	430	140	10	5.7	4.9	497	11
610x305x149	7	1050	8	1050	223	900	100	200	12	500	140	10	5.7	4.9	565	11
{1570 kN}	6	903	8	903	330	900	100	200	12	430	140	10	5.7	4.9	486	11

For guidance on the use of tables see Explanatory notes in Table G.1

BEAM: S355
END PLATES: S275
BOLTS: M20, 8.8

Table G.5 *Continued*

PARTIAL DEPTH END PLATES, ORDINARY or FLOWDRILL BOLTS
200 x 12 or 150 x 10 mm End Plate

Beam size {$V_{Rd,beam}$}	Bolt rows n_1	Un-notched Shear Resist V_{Rd} kN	Un-notched Crit chck	Single notch Shear Resist V_{Rd} kN	Single notch Max length l_n mm	Double notch Shear Resist V_{Rd} kN	Double notch Max length l_n mm	Fitting (End Plate) Width b_p mm	Fitting (End Plate) Thck t_p mm	Fitting (End Plate) Height h_p mm	Fitting (End Plate) Gauge p_3 mm	Fillet weld Leg s mm	Min support thickness S275 mm	Min support thickness S355 mm	Tying Resist $N_{Rd,u}$ kN	Tying Crit check
610x229x140	7	1050	8	1050	284	1010	100	200	12	500	140	10	5.7	4.9	572	11
{1690 kN}	6	903	8	903	380	903	158	200	12	430	140	10	5.7	4.9	492	11
610x229x125	7	1050	8	1050	210	907	100	200	12	500	140	10	5.7	4.9	565	11
{1520 kN}	6	903	8	903	314	903	102	200	12	430	140	10	5.7	4.9	486	11
610x229x113	7	995	4	995	193	837	100	200	12	500	140	8	5.3	4.7	544	11
{1420 kN}	6	856	4	856	296	837	100	200	12	430	140	8	5.4	4.7	468	11
610x229x101	7	968	4	968	182	805	100	200	12	500	140	8	5.2	4.5	541	11
{1370 kN}	6	833	4	833	284	805	100	200	12	430	140	8	5.2	4.6	465	11
610x178x100+	7	1010	4	1010	183	852	100	200	12	500	140	8	5.4	4.7	545	11
{1450 kN}	6	871	4	871	281	852	100	200	12	430	140	8	5.5	4.8	469	11
610x178x92+	7	1010	4	1010	173	836	100	200	12	500	140	8	5.4	4.7	543	11
{1410 kN}	6	865	4	865	270	836	100	200	12	430	140	8	5.4	4.7	467	11
610x178x82+	7	922	4	922	165	N/A	100	200	12	500	140	8	5.0	4.3	538	11
{1290 kN}	6	793	4	793	261	759	100	200	12	430	140	8	5.0	4.3	463	11
533x312x272+	6	903	8	903	575	903	294	200	12	430	140	15	5.7	4.9	595	11
{2480 kN}	5	753	8	753	693	753	356	200	12	360	140	15	5.7	4.9	498	11
533x312x219+	6	903	8	903	466	903	226	200	12	430	140	15	5.7	4.9	555	11
{2120 kN}	5	753	8	753	562	753	285	200	12	360	140	15	5.7	4.9	465	11
533x312x182+	6	903	8	903	364	903	142	200	12	430	140	12	5.7	4.9	519	11
{1740 kN}	5	753	8	753	451	753	213	200	12	360	140	12	5.7	4.9	434	11
533x312x150+	6	903	8	903	251	810	100	200	12	430	140	10	5.7	4.9	490	11
{1460 kN}	5	753	8	753	354	753	132	200	12	360	140	10	5.7	4.9	410	11

For guidance on the use of tables see Explanatory notes in Table G.1

BEAM: S355
END PLATES: S275
BOLTS: M20, 8.8

Table G.5 *Continued*

PARTIAL DEPTH END PLATES, ORDINARY or FLOWDRILL BOLTS
200 x 12 or 150 x 10 mm End Plate

Beam size $\{V_{Rd,beam}\}$	Bolt rows n_1	Un-notched Shear Resist V_{Rd} kN	Un-notched Crit chck	Single notch Shear Resist V_{Rd} kN	Single notch Max length l_n mm	Double notch Shear Resist V_{Rd} kN	Double notch Max length l_n mm	Fitting (End Plate) Width b_p mm	Fitting (End Plate) Thck t_p mm	Fitting (End Plate) Height h_p mm	Fitting (End Plate) Gauge p_3 mm	Fillet weld Leg s mm	Min support thickness S275 mm	Min support thickness S355 mm	Tying Resist $N_{Rd,u}$ kN	Tying Crit check
533x210x138+	6	903	8	903	324	903	124	200	12	430	140	10	5.7	4.9	500	11
{1680 kN}	5	753	8	753	408	753	198	200	12	360	140	10	5.7	4.9	418	11
533x210x122	6	903	8	903	240	814	100	200	12	430	140	10	5.7	4.9	490	11
{1450 kN}	5	753	8	753	338	753	134	200	12	360	140	10	5.7	4.9	410	11
533x210x109	6	894	4	894	184	733	100	200	12	430	140	8	5.6	4.9	470	11
{1330 kN}	5	749	4	749	287	733	100	200	12	360	140	8	5.6	4.9	394	11
533x210x101	6	833	4	833	180	677	100	200	12	430	140	8	5.2	4.6	466	11
{1240 kN}	5	697	4	697	283	677	100	200	12	360	140	8	5.2	4.6	391	11
533x210x92	6	801	4	801	174	645	100	200	12	430	140	8	5.0	4.4	463	11
{1170 kN}	5	671	4	671	276	645	100	200	12	360	140	8	5.0	4.4	388	11
533x210x82	6	761	4	761	163	N/A	100	200	12	430	140	8	4.8	4.2	461	11
{1120 kN}	5	638	4	638	264	604	100	200	12	360	140	8	4.8	4.2	386	11
533x165x85+	6	794	4	794	171	644	100	150	10	430	90	8	5.0	4.3	604	11
{1170 kN}	5	665	4	665	270	644	100	150	10	360	90	8	5.0	4.4	505	11
533x165x74+	6	769	4	769	160	N/A	100	150	10	430	90	8	4.8	4.2	597	11
{1120 kN}	5	644	4	644	258	615	100	150	10	360	90	8	4.8	4.2	500	11
533x165x66+	6	706	4	706	152	N/A	100	150	10	430	90	8	4.4	3.9	589	11
{1020 kN}	5	591	4	591	248	557	100	150	10	360	90	8	4.4	3.9	493	11

For guidance on the use of tables see Explanatory notes in Table G.1

BEAM: S355
END PLATES: S275
BOLTS: M20, 8.8

Table G.5 *Continued*

PARTIAL DEPTH END PLATES, ORDINARY or FLOWDRILL BOLTS
200 x 12 or 150 x 10 mm End Plate

Beam size $\{V_{Rd,beam}\}$	Bolt rows n_1	Un-notched Shear Resist V_{Rd} kN	Un-notched Crit chck	Single notch Shear Resist V_{Rd} kN	Single notch Max length l_n mm	Double notch Shear Resist V_{Rd} kN	Double notch Max length l_n mm	Fitting (End Plate) Width b_p mm	Fitting (End Plate) Thck t_p mm	Fitting (End Plate) Height h_p mm	Fitting (End Plate) Gauge p_3 mm	Fillet weld Leg s mm	Min support thickness S275 mm	Min support thickness S355 mm	Tying Resist $N_{Rd,u}$ kN	Tying Crit check
457x191x161+	5	753	8	753	394	753	194	150	10	360	90	15	5.7	4.9	724	11
{1810 kN}	4	602	8	602	495	602	254	150	10	290	90	15	5.7	4.9	583	11
457x191x133+	5	753	8	753	313	753	126	150	10	360	90	12	5.7	4.9	631	11
{1510 kN}	4	602	8	602	400	602	197	150	10	290	90	12	5.7	4.9	508	11
457x191x106+	5	753	8	753	207	641	100	150	10	360	90	10	5.7	4.9	559	11
{1230 kN}	4	602	8	602	307	602	122	150	10	290	90	10	5.7	4.9	450	11
457x191x98	5	736	4	736	166	576	100	150	10	360	90	8	5.5	4.8	515	11
{1110 kN}	4	593	4	593	272	576	100	150	10	290	90	8	5.6	4.9	415	11
457x191x89	5	678	4	678	161	523	100	150	10	360	90	8	5.1	4.4	507	11
{1030 kN}	4	546	4	546	266	523	100	150	10	290	90	8	5.1	4.5	409	11
457x191x82	5	657	4	657	155	501	100	150	10	360	90	8	4.9	4.3	502	11
{976 kN}	4	530	4	530	260	501	100	150	10	290	90	8	5.0	4.3	404	11
457x191x74	5	598	4	598	151	N/A	100	150	10	360	90	8	4.5	3.9	494	11
{895 kN}	4	481	4	481	256	451	100	150	10	290	90	8	4.5	3.9	398	11
457x191x67	5	564	4	564	144	N/A	100	150	10	360	90	6	4.2	3.7	465	11
{839 kN}	4	455	4	455	248	420	100	150	10	290	90	6	4.3	3.7	374	11
457x152x82	5	678	4	678	161	528	100	150	10	360	90	8	5.1	4.4	507	11
{1040 kN}	4	546	4	546	263	528	100	150	10	290	90	8	5.1	4.5	409	11
457x152x74	5	620	4	620	156	476	100	150	10	360	90	8	4.7	4.1	499	11
{938 kN}	4	499	4	499	258	476	100	150	10	290	90	8	4.7	4.1	402	11
457x152x67	5	598	4	598	148	N/A	100	150	10	360	90	8	4.5	3.9	494	11
{899 kN}	4	481	4	481	250	453	100	150	10	290	90	8	4.5	3.9	398	11
457x152x60	5	538	4	538	144	N/A	100	150	10	360	90	6	4.0	3.5	462	11
{806 kN}	4	433	4	433	245	402	100	150	10	290	90	6	4.1	3.6	372	11
457x152x52	5	505	4	505	134	N/A	100	150	10	360	90	6	3.8	3.3	458	11
{747 kN}	4	407	4	407	233	371	100	150	10	290	90	6	3.8	3.3	369	11

For guidance on the use of tables see Explanatory notes in Table G.1

BEAM: S355
END PLATES: S275
BOLTS: M20, 8.8

Table G.5 *Continued*

PARTIAL DEPTH END PLATES, ORDINARY or FLOWDRILL BOLTS
200 x 12 or 150 x 10 mm End Plate

Beam size {$V_{Rd,beam}$}	Bolt rows	Un-notched		Single notch		Double notch		Fitting (End Plate)				Fillet weld	Min support thickness		Tying	
	n_1	Shear Resist V_{Rd} kN	Crit chck	Shear Resist V_{Rd} kN	Max length l_n mm	Shear Resist V_{Rd} kN	Max length l_n mm	Width b_p mm	Thck t_p mm	Height h_p mm	Gauge p_3 mm	Leg s mm	S275 mm	S355 mm	Resist $N_{Rd,u}$ kN	Crit check
406x178x85+ {966 kN}	4	567	4	567	187	453	100	150	10	290	90	8	5.3	4.6	411	11
406x178x74 {858 kN}	4	508	4	508	182	398	100	150	10	290	90	8	4.8	4.2	402	11
406x178x67 {790 kN}	4	471	4	471	177	364	100	150	10	290	90	6	4.4	3.9	376	11
406x178x60 {709 kN}	4	423	4	423	173	322	100	150	10	290	90	6	4.0	3.5	371	11
406x178x54 {683 kN}	4	412	4	412	163	309	100	150	10	290	90	6	3.9	3.4	370	11
406x140x53+ {683 kN}	4	423	4	423	168	359	100	150	10	290	90	6	4.0	3.5	371	11
406x140x46 {611 kN}	4	364	4	364	164	305	100	150	10	290	90	6	3.4	3.0	365	11
406x140x39 {566 kN}	4	342	4	342	151	281	100	150	10	290	90	6	3.2	2.8	362	11
356x171x67 {733 kN}	3	369	4	369	206	341	100	150	10	220	90	8	4.6	4.0	303	11
356x171x57 {646 kN}	3	329	4	329	196	295	100	150	10	220	90	6	4.1	3.6	282	11
356x171x51 {587 kN}	3	300	4	300	191	266	100	150	10	220	90	6	3.8	3.3	279	11
356x171x45 {549 kN}	3	284	4	284	183	247	100	150	10	220	90	6	3.6	3.1	277	11
356x127x39 {527 kN}	3	268	4	268	182	235	100	150	10	220	90	6	3.4	2.9	276	11
356x127x33 {472 kN}	3	243	4	243	172	209	100	150	10	220	90	6	3.1	2.7	273	11

For guidance on the use of tables see Explanatory notes in Table G.1

BEAM: S355
END PLATES: S275
BOLTS: M20, 8.8

Table G.5 *Continued*

PARTIAL DEPTH END PLATES, ORDINARY or FLOWDRILL BOLTS
200 x 12 or 150 x 10 mm End Plate

Beam size $\{V_{Rd,beam}\}$	Bolt rows	Un-notched		Single notch		Double notch		Fitting (End Plate)				Fillet weld	Min support thickness		Tying	
	n_1	Shear Resist V_{Rd} kN	Crit chck	Shear Resist V_{Rd} kN	Max length l_n mm	Shear Resist V_{Rd} kN	Max length l_n mm	Width b_p mm	Thck t_p mm	Height h_p mm	Gauge p_3 mm	Leg s mm	S275 mm	S355 mm	Resist $N_{Rd,u}$ kN	Crit check
305x165x54 {545 kN}	3	321	4	321	116	219	100	150	10	220	90	6	4.0	3.5	281	11
305x165x46 {461 kN}	3	272	4	272	113	181	100	150	10	220	90	6	3.4	3.0	276	11
305x165x40 {411 kN}	3	243	4	243	108	158	100	150	10	220	90	6	3.1	2.7	273	11
305x127x48 {612 kN}	3	365	4	365	109	250	100	150	10	220	90	8	4.6	4.0	302	11
305x127x42 {542 kN}	3	325	4	325	103	217	100	150	10	220	90	6	4.1	3.6	282	11
305x127x37 {481 kN}	3	288	4	288	100	189	100	150	10	220	90	6	3.6	3.2	278	11
305x102x33 {452 kN}	3	268	4	268	110	186	100	150	10	220	90	6	3.4	2.9	276	11
305x102x28 {407 kN}	3	243	4	243	102	164	100	150	10	220	90	6	3.1	2.7	273	11
305x102x25 {386 kN}	3	235	4	231	100	155	100	150	10	220	90	6	3.0	2.6	272	11
254x146x43 {415 kN}	2	199	4	199	134	132	100	150	10	150	90	6	3.7	3.3	190	11
254x146x37 {361 kN}	2	174	4	174	129	111	100	150	10	150	90	6	3.3	2.9	187	11
254x146x31 {336 kN}	2	166	4	166	119	101	100	150	10	150	90	6	3.1	2.7	186	11
254x102x28 {365 kN}	2	174	4	174	127	116	100	150	10	150	90	6	3.3	2.9	187	11
254x102x25 {341 kN}	2	166	4	166	121	107	100	150	10	150	90	6	3.1	2.7	186	11
254x102x22 {320 kN}	2	158	4	158	113	99	100	150	10	150	90	6	3.0	2.6	185	11

For guidance on the use of tables see Explanatory notes in Table G.1

BEAM: S275
END PLATES: S275
HOLLO-BOLTS: M20, 8.8

Table G.6

PARTIAL DEPTH END PLATES, HOLLO-BOLTS
200 x 12 or 180 x 10 mm End Plate

	Bolt rows	Un-notched		Fitting (End Plate)				Web weld	Min support thickness		Tying	
Beam size {$V_{Rd,beam}$}	n_1	Shear Resistance V_{Rd} kN	Critical Check	Width b_p mm	Thck t_p mm	Height h_p mm	Gauge p_3 mm	Leg s mm	S275 mm	S355 mm	Resistance $N_{Rd,u}$ kN	Critical Check
1016x305x487+	10	1280	8	200	12	810	110	18	4.1	3.5	1630	11
{4930 kN}	9	1150	8	200	12	730	110	18	4.1	3.5	1470	11
	8	1030	8	200	12	650	110	18	4.1	3.5	1310	11
1016x305x437+	10	1280	8	200	12	810	110	18	4.1	3.5	1590	11
{4420 kN}	9	1150	8	200	12	730	110	18	4.1	3.5	1430	11
	8	1030	8	200	12	650	110	18	4.1	3.5	1270	11
1016x305x393+	10	1280	8	200	12	810	110	15	4.1	3.5	1560	11
{3990 kN}	9	1150	8	200	12	730	110	15	4.1	3.5	1410	11
	8	1030	8	200	12	650	110	15	4.1	3.5	1250	11
1016x305x349+	10	1280	8	200	12	810	110	12	4.1	3.5	1530	11
{3610 kN}	9	1150	8	200	12	730	110	12	4.1	3.5	1380	11
	8	1030	8	200	12	650	110	12	4.1	3.5	1230	11
1016x305x314+	10	1280	8	200	12	810	110	12	4.1	3.5	1530	11
{3260 kN}	9	1150	8	200	12	730	110	12	4.1	3.5	1370	11
	8	1030	8	200	12	650	110	12	4.1	3.5	1220	11
1016x305x272+	10	1280	8	200	12	810	110	10	4.1	3.5	1450	11
{2830 kN}	9	1150	8	200	12	730	110	10	4.1	3.5	1310	11
	8	1030	8	200	12	650	110	10	4.1	3.5	1160	11
1016x305x249+	10	1280	8	200	12	810	110	10	4.1	3.5	1450	11
{2770 kN}	9	1150	8	200	12	730	110	10	4.1	3.5	1310	11
	8	1030	8	200	12	650	110	10	4.1	3.5	1160	11
1016x305x222+	10	1280	8	200	12	810	110	10	4.1	3.5	1440	11
{2640 kN}	9	1150	8	200	12	730	110	10	4.1	3.5	1300	11
	8	1030	8	200	12	650	110	10	4.1	3.5	1160	11

For guidance on the use of tables see Explanatory notes in Table G.1

BEAM: S275
END PLATES: S275
HOLLO-BOLTS: M20, 8.8

Table G.6 *Continued*

PARTIAL DEPTH END PLATES, HOLLO-BOLTS
200 x 12 or 180 x 10 mm End Plate

Beam size	Bolt rows	Un-notched		Fitting (End Plate)				Web weld	Min support thickness		Tying	
		Shear Resistance	Critical Check	Width	Thck	Height	Gauge	Leg	S275	S355	Resistance	Critical Check
{$V_{Rd,beam}$}	n_1	V_{Rd} kN		b_p mm	t_p mm	h_p mm	p_3 mm	s mm	mm	mm	$N_{Rd,u}$ kN	
914x419x388	9	1150	8	200	12	730	110	15	4.1	3.5	1390	11
{3240 kN}	8	1030	8	200	12	650	110	15	4.1	3.5	1240	11
914x419x343	9	1150	8	200	12	730	110	12	4.1	3.5	1370	11
{2920 kN}	8	1030	8	200	12	650	110	12	4.1	3.5	1220	11
914x305x289	10	1280	8	200	12	810	110	12	4.1	3.5	1530	11
{2900 kN}	9	1150	8	200	12	730	110	12	4.1	3.5	1370	11
	8	1030	8	200	12	650	110	12	4.1	3.5	1220	11
914x305x253	10	1280	8	200	12	810	110	10	4.1	3.5	1470	11
{2570 kN}	9	1150	8	200	12	730	110	10	4.1	3.5	1320	11
	8	1030	8	200	12	650	110	10	4.1	3.5	1180	11
914x305x224	10	1280	8	200	12	810	110	10	4.1	3.5	1440	11
{2350 kN}	9	1150	8	200	12	730	110	10	4.1	3.5	1300	11
	8	1030	8	200	12	650	110	10	4.1	3.5	1150	11
914x305x201	10	1280	8	200	12	810	110	10	4.1	3.5	1420	11
{2210 kN}	9	1150	8	200	12	730	110	10	4.1	3.5	1280	11
	8	1030	8	200	12	650	110	10	4.1	3.5	1140	11
838x292x226	9	1150	8	200	12	730	110	10	4.1	3.5	1300	11
{2220 kN}	8	1030	8	200	12	650	110	10	4.1	3.5	1160	11
838x292x194	9	1150	8	200	12	730	110	10	4.1	3.5	1270	11
{2000 kN}	8	1030	8	200	12	650	110	10	4.1	3.5	1130	11
838x292x176	9	1150	8	200	12	730	110	8	4.1	3.5	1200	11
{1890 kN}	8	1030	8	200	12	650	110	8	4.1	3.5	1070	11

For guidance on the use of tables see Explanatory notes in Table G.1

BEAM: S275
END PLATES: S275
HOLLO-BOLTS: M20, 8.8

Table G.6 *Continued*

PARTIAL DEPTH END PLATES, HOLLO-BOLTS
200 x 12 or 180 x 10 mm End Plate

Beam size $\{V_{Rd,beam}\}$	Bolt rows n_1	Un-notched: Shear Resistance V_{Rd} kN	Un-notched: Critical Check	Fitting (End Plate): Width b_p mm	Fitting (End Plate): Thck t_p mm	Fitting (End Plate): Height h_p mm	Fitting (End Plate): Gauge p_3 mm	Web weld: Leg s mm	Min support thickness: S275 mm	Min support thickness: S355 mm	Tying: Resistance $N_{Rd,u}$ kN	Tying: Critical Check
762x267x197	8	1030	8	200	12	650	110	10	4.1	3.5	1150	11
{1950 kN}	7	897	8	200	12	570	110	10	4.1	3.5	1010	11
762x267x173	8	1030	8	200	12	650	110	10	4.1	3.5	1130	11
{1760 kN}	7	897	8	200	12	570	110	10	4.1	3.5	987	11
762x267x147	8	1030	8	200	12	650	110	8	4.1	3.5	1050	11
{1560 kN}	7	897	8	200	12	570	110	8	4.1	3.5	922	11
762x267x134	8	1030	8	200	12	650	110	8	4.1	3.5	1040	11
{1520 kN}	7	897	8	200	12	570	110	8	4.1	3.5	912	11
686x254x170	7	897	8	200	12	570	110	10	4.1	3.5	990	11
{1630 kN}	6	769	8	200	12	490	110	10	4.1	3.5	851	11
686x254x152	7	897	8	200	12	570	110	8	4.1	3.5	927	11
{1470 kN}	6	769	8	200	12	490	110	8	4.1	3.5	797	11
686x254x140	7	897	8	200	12	570	110	8	4.1	3.5	917	11
{1370 kN}	6	769	8	200	12	490	110	8	4.1	3.5	788	11
686x254x125	7	897	8	200	12	570	110	8	4.1	3.5	908	11
{1280 kN}	6	769	8	200	12	490	110	8	4.1	3.5	781	11
610x305x238 {1890 kN}	6	769	8	200	12	490	110	12	4.1	3.5	913	11
610x305x179 {1440 kN}	6	769	8	200	12	490	110	8	4.1	3.5	807	11
610x305x149 {1205 kN}	6	769	8	200	12	490	110	8	4.1	3.5	782	11

For guidance on the use of tables see Explanatory notes in Table G.1

BEAM: S275
END PLATES: S275
HOLLO-BOLTS: M20, 8.8

Table G.6 *Continued*

PARTIAL DEPTH END PLATES, HOLLO-BOLTS
200 x 12 or 180 x 10 mm End Plate

Beam size	Bolt rows	Un-notched		Fitting (End Plate)				Web weld	Min support thickness		Tying	
Beam size {$V_{Rd,beam}$}	n_1	Shear Resistance V_{Rd} kN	Critical Check	Width b_p mm	Thck t_p mm	Height h_p mm	Gauge p_3 mm	Leg s mm	S275 mm	S355 mm	Resistance $N_{Rd,u}$ kN	Critical Check
610x229x140 {1300 kN}	6	769	8	200	12	490	110	8	4.1	3.5	796	11
610x229x125 {1170 kN}	6	769	8	200	12	490	110	8	4.1	3.5	783	11
610x229x113 {1090 kN}	6	749	4	200	12	490	110	8	4.0	3.5	774	11
610x229x101 {1060 kN}	6	735	4	200	12	490	110	6	3.9	3.4	736	11
610x178x100+ {1110 kN}	6	762	4	200	12	490	110	8	4.0	3.5	776	11
610x178x92+ {1090 kN}	6	763	4	200	12	490	110	8	4.0	3.5	772	11
610x178x82+ {1000 kN}	6	700	4	200	12	490	110	6	3.7	3.2	731	11
533x312x272+ {1910 kN}	5	641	8	200	12	410	110	12	4.1	3.5	768	11
533x312x219+ {1630 kN}	5	641	8	200	12	410	110	12	4.1	3.5	761	11
533x312x182+ {1340 kN}	5	641	8	200	12	410	110	10	4.1	3.5	720	11
533x312x150+ {1120 kN}	5	641	8	200	12	410	110	8	4.1	3.5	662	11

For guidance on the use of tables see Explanatory notes in Table G.1

BEAM: S275
END PLATES: S275
HOLLO-BOLTS: M20, 8.8

Table G.6 *Continued*

PARTIAL DEPTH END PLATES, HOLLO-BOLTS
200 x 12 or 180 x 10 mm End Plate

Beam size $\{V_{Rd,beam}\}$	Bolt rows	Un-notched		Fitting (End Plate)				Web weld	Min support thickness		Tying	
	n_1	Shear Resistance V_{Rd} kN	Critical Check	Width b_p mm	Thck t_p mm	Height h_p mm	Gauge p_3 mm	Leg s mm	S275 mm	S355 mm	Resistance $N_{Rd,u}$ kN	Critical Check
533x210x138+ {1290 kN}	5	641	8	200	12	410	110	10	4.1	3.5	714	11
533x210x122 {1110 kN}	5	641	8	200	12	410	110	8	4.1	3.5	662	11
533x210x109 {1020 kN}	5	641	8	200	12	410	110	8	4.1	3.5	652	11
533x210x101 {952 kN}	5	610	4	200	12	410	110	8	3.9	3.4	645	11
533x210x92 {909 kN}	5	592	4	200	12	410	110	6	3.8	3.3	613	11
533x210x82 {865 kN}	5	562	4	200	12	410	110	6	3.6	3.1	609	11
533x165x85+ {902 kN}	5	581	4	200	12	410	110	6	3.7	3.2	614	11
533x165x74+ {871 kN}	5	568	4	200	12	410	110	6	3.6	3.1	609	11
533x165x66+ {793 kN}	5	521	4	200	12	410	110	6	3.3	2.9	603	11

For guidance on the use of tables see Explanatory notes in Table G.1

BEAM: S275
END PLATES: S275
HOLLO-BOLTS: M20, 8.8

Table G.6 *Continued*

PARTIAL DEPTH END PLATES, HOLLO-BOLTS
200 x 12 or 180 x 10 mm End Plate

Beam size {$V_{Rd,beam}$}	Bolt rows n_1	Un-notched Shear Resistance V_{Rd} kN	Un-notched Critical Check	Fitting (End Plate) Width b_p mm	Fitting (End Plate) Thck t_p mm	Fitting (End Plate) Height h_p mm	Fitting (End Plate) Gauge p_3 mm	Web weld Leg s mm	Min support thickness S275 mm	Min support thickness S355 mm	Tying Resistance $N_{Rd,u}$ kN	Tying Critical Check
457x191x161+ {1390 kN}	4	427	8	180	10	330	90	12	3.4	3.0	606	11
457x191x133+ {1160 kN}	4	427	8	180	10	330	90	10	3.4	3.0	595	11
457x191x106+ {947 kN}	4	427	8	180	10	330	90	8	3.4	3.0	520	11
457x191x98 {852 kN}	4	427	8	180	10	330	90	8	3.4	3.0	507	11
457x191x89 {789 kN}	4	427	8	180	10	330	90	6	3.4	3.0	467	11
457x191x82 {756 kN}	4	427	8	180	10	330	90	6	3.4	3.0	462	11
457x191x74 {693 kN}	4	424	4	180	10	330	90	6	3.4	2.9	454	11
457x191x67 {650 kN}	4	401	4	180	10	330	90	6	3.2	2.8	450	11
457x152x82 {798 kN}	4	427	8	180	10	330	90	6	3.4	3.0	467	11
457x152x74 {721 kN}	4	427	8	180	10	330	90	6	3.4	3.0	459	11
457x152x67 {697 kN}	4	424	4	180	10	330	90	6	3.4	2.9	454	11
457x152x60 {624 kN}	4	382	4	180	10	330	90	6	3.0	2.6	447	11
457x152x52 {578 kN}	4	358	4	180	10	330	90	6	2.8	2.5	443	11

For guidance on the use of tables see Explanatory notes in Table G.1

BEAM: S275
END PLATES: S275
HOLLO-BOLTS: M20, 8.8

Table G.6 *Continued*

PARTIAL DEPTH END PLATES, HOLLO-BOLTS
200 x 12 or 180 x 10 mm End Plate

	Bolt rows	Un-notched		Fitting (End Plate)				Web weld	Min support thickness		Tying	
Beam size $\{V_{Rd,beam}\}$	n_1	Shear Resistance V_{Rd} kN	Critical Check	Width b_p mm	Thck t_p mm	Height h_p mm	Gauge p_3 mm	Leg s mm	S275 mm	S355 mm	Resistance $N_{Rd,u}$ kN	Critical Check
406x178x85+ {742 kN}	4	427	8	180	10	330	90	8	3.4	3.0	502	11
406x178x74 {664 kN}	4	427	8	180	10	330	90	6	3.4	3.0	458	11
406x178x67 {612 kN}	4	415	4	180	10	330	90	6	3.3	2.9	452	11
406x178x60 {549 kN}	4	373	4	180	10	330	90	6	3.0	2.6	445	11
406x178x54 {529 kN}	4	363	4	180	10	330	90	6	2.9	2.5	444	11
406x140x53+ {549 kN}	4	373	4	180	10	330	90	6	3.0	2.6	445	11
406x140x46 {473 kN}	4	321	4	180	10	330	90	6	2.5	2.2	437	11
406x140x39 {438 kN}	4	302	4	180	10	330	90	6	2.4	2.1	434	11
356x171x67 {568 kN}	3	321	8	180	10	250	90	6	3.4	3.0	345	11
356x171x57 {501 kN}	3	289	4	180	10	250	90	6	3.1	2.7	338	11
356x171x51 {455 kN}	3	264	4	180	10	250	90	6	2.8	2.4	334	11
356x171x45 {425 kN}	3	250	4	180	10	250	90	6	2.6	2.3	332	11
356x127x39 {408 kN}	3	236	4	180	10	250	90	6	2.5	2.2	330	11
356x127x33 {366 kN}	3	214	4	180	10	250	90	6	2.3	2.0	327	11

For guidance on the use of tables see Explanatory notes in Table G.1

BEAM: S275
END PLATES: S275
HOLLO-BOLTS: M20, 8.8

Table G.6 *Continued*

PARTIAL DEPTH END PLATES, HOLLO-BOLTS
200 x 12 or 180 x 10 mm End Plate

Beam size {$V_{Rd,beam}$}	Bolt rows	Un-notched		Fitting (End Plate)				Web weld	Min support thickness		Tying	
	n_1	Shear Resistance V_{Rd} kN	Critical Check	Width b_p mm	Thck t_p mm	Height h_p mm	Gauge p_3 mm	Leg s mm	S275 mm	S355 mm	Resistance $N_{Rd,u}$ kN	Critical Check
305x165x54 {422 kN}	2	192	4	180	10	170	90	6	3.0	2.7	229	11
305x165x46 {357 kN}	2	163	4	180	10	170	90	6	2.6	2.2	225	11
305x165x40 {319 kN}	2	146	4	180	10	170	90	6	2.3	2.0	222	11
305x127x48 {474 kN}	2	214	8	180	10	170	90	6	3.4	3.0	234	11
305x127x42 {420 kN}	2	194	4	180	10	170	90	6	3.1	2.7	230	11
305x127x37 {372 kN}	2	172	4	180	10	170	90	6	2.7	2.4	226	11
305x102x33 {350 kN}	3	236	4	180	10	250	90	6	2.5	2.2	330	11
305x102x28 {315 kN}	3	214	4	180	10	250	90	6	2.3	2.0	327	11
305x102x25 {299 kN}	3	207	4	180	10	250	90	6	2.2	1.9	326	11
254x146x43 {321 kN}	2	175	4	180	10	170	90	6	2.8	2.4	227	11
254x146x37 {280 kN}	2	153	4	180	10	170	90	6	2.4	2.1	223	11
254x146x31 {260 kN}	2	146	4	180	10	170	90	6	2.3	2.0	222	11
254x102x28 {283 kN}	2	153	4	180	10	170	90	6	2.4	2.1	223	11
254x102x25 {265 kN}	2	146	4	180	10	170	90	6	2.3	2.0	222	11
254x102x22 {248 kN}	2	138	4	180	10	170	90	6	2.2	1.9	221	11

For guidance on the use of tables see Explanatory notes in Table G.1

BEAM: S355
END PLATES: S275
HOLLO-BOLTS: M20, 8.8

Table G.7

PARTIAL DEPTH END PLATES, HOLLO-BOLTS
200 x 12 or 180 x 10 mm End Plate

Beam size	Bolt rows	Un-notched		Fitting (End Plate)				Web weld	Min support thickness		Tying	
Beam size $\{V_{Rd,beam}\}$	n_1	Shear Resistance V_{Rd} kN	Critical Check	Width b_p mm	Thck t_p mm	Height h_p mm	Gauge p_3 mm	Leg s mm	S275 mm	S355 mm	Resistance $N_{Rd,u}$ kN	Critical Check
1016x305x487+	10	1280	8	200	12	810	110	22	4.1	3.5	1690	11
{6480 kN}	9	1150	8	200	12	730	110	22	4.1	3.5	1520	11
	8	1030	8	200	12	650	110	22	4.1	3.5	1350	11
1016x305x437+	10	1280	8	200	12	810	110	20	4.1	3.5	1630	11
{5810 kN}	9	1150	8	200	12	730	110	20	4.1	3.5	1470	11
	8	1030	8	200	12	650	110	20	4.1	3.5	1310	11
1016x305x393+	10	1280	8	200	12	810	110	18	4.1	3.5	1600	11
{5240 kN}	9	1150	8	200	12	730	110	18	4.1	3.5	1440	11
	8	1030	8	200	12	650	110	18	4.1	3.5	1280	11
1016x305x349+	10	1280	8	200	12	810	110	15	4.1	3.5	1560	11
{4700 kN}	9	1150	8	200	12	730	110	15	4.1	3.5	1410	11
	8	1030	8	200	12	650	110	15	4.1	3.5	1250	11
1016x305x314+	10	1280	8	200	12	810	110	15	4.1	3.5	1540	11
{4240 kN}	9	1150	8	200	12	730	110	15	4.1	3.5	1380	11
	8	1030	8	200	12	650	110	15	4.1	3.5	1230	11
1016x305x272+	10	1280	8	200	12	810	110	12	4.1	3.5	1500	11
{3680 kN}	9	1150	8	200	12	730	110	12	4.1	3.5	1350	11
	8	1030	8	200	12	650	110	12	4.1	3.5	1200	11
1016x305x249+	10	1280	8	200	12	810	110	12	4.1	3.5	1500	11
{3600 kN}	9	1150	8	200	12	730	110	12	4.1	3.5	1350	11
	8	1030	8	200	12	650	110	12	4.1	3.5	1200	11
1016x305x222+	10	1280	8	200	12	810	110	12	4.1	3.5	1500	11
{3430 kN}	9	1150	8	200	12	730	110	12	4.1	3.5	1350	11
	8	1030	8	200	12	650	110	12	4.1	3.5	1200	11

For guidance on the use of tables see Explanatory notes in Table G.1

BEAM: S355
END PLATES: S275
HOLLO-BOLTS: M20, 8.8

Table G.7 *Continued*

PARTIAL DEPTH END PLATES, HOLLO-BOLTS
200 x 12 or 180 x 10 mm End Plate

Beam size $\{V_{Rd,beam}\}$	Bolt rows n_1	Un-notched Shear Resistance V_{Rd} kN	Un-notched Critical Check	Fitting (End Plate) Width b_p mm	Fitting (End Plate) Thck t_p mm	Fitting (End Plate) Height h_p mm	Fitting (End Plate) Gauge p_3 mm	Web weld Leg s mm	Min support thickness S275 mm	Min support thickness S355 mm	Tying Resistance $N_{Rd,u}$ kN	Tying Critical Check
914x419x388	9	1150	8	200	12	730	110	15	4.1	3.5	1410	11
{4220 kN}	8	1030	8	200	12	650	110	15	4.1	3.5	1250	11
914x419x343	9	1150	8	200	12	730	110	15	4.1	3.5	1390	11
{3800 kN}	8	1030	8	200	12	650	110	15	4.1	3.5	1230	11
914x305x289	10	1280	8	200	12	810	110	15	4.1	3.5	1540	11
{3780 kN}	9	1150	8	200	12	730	110	15	4.1	3.5	1390	11
	8	1030	8	200	12	650	110	15	4.1	3.5	1230	11
914x305x253	10	1280	8	200	12	810	110	12	4.1	3.5	1510	11
{3350 kN}	9	1150	8	200	12	730	110	12	4.1	3.5	1360	11
	8	1030	8	200	12	650	110	12	4.1	3.5	1210	11
914x305x224	10	1280	8	200	12	810	110	12	4.1	3.5	1500	11
{3060 kN}	9	1150	8	200	12	730	110	12	4.1	3.5	1350	11
	8	1030	8	200	12	650	110	12	4.1	3.5	1200	11
914x305x201	10	1280	8	200	12	810	110	12	4.1	3.5	1490	11
{2870 kN}	9	1150	8	200	12	730	110	12	4.1	3.5	1340	11
	8	1030	8	200	12	650	110	12	4.1	3.5	1190	11
838x292x226	9	1150	8	200	12	730	110	12	4.1	3.5	1350	11
{2900 kN}	8	1030	8	200	12	650	110	12	4.1	3.5	1200	11
838x292x194	9	1150	8	200	12	730	110	10	4.1	3.5	1270	11
{2610 kN}	8	1030	8	200	12	650	110	10	4.1	3.5	1130	11
838x292x176	9	1150	8	200	12	730	110	10	4.1	3.5	1260	11
{2460 kN}	8	1030	8	200	12	650	110	10	4.1	3.5	1120	11

For guidance on the use of tables see Explanatory notes in Table G.1

BEAM: S355
END PLATES: S275
HOLLO-BOLTS: M20, 8.8

Table G.7 *Continued*

PARTIAL DEPTH END PLATES, HOLLO-BOLTS
200 x 12 or 180 x 10 mm End Plate

Beam size $\{V_{Rd,beam}\}$	Bolt rows n_1	Un-notched: Shear Resistance V_{Rd} kN	Un-notched: Critical Check	Fitting (End Plate): Width b_p mm	Fitting (End Plate): Thck t_p mm	Fitting (End Plate): Height h_p mm	Fitting (End Plate): Gauge p_3 mm	Web weld: Leg s mm	Min support thickness: S275 mm	Min support thickness: S355 mm	Tying: Resistance $N_{Rd,u}$ kN	Tying: Critical Check
762x267x197	8	1030	8	200	12	650	110	12	4.1	3.5	1200	11
{2530 kN}	7	897	8	200	12	570	110	12	4.1	3.5	1050	11
762x267x173	8	1030	8	200	12	650	110	10	4.1	3.5	1130	11
{2290 kN}	7	897	8	200	12	570	110	10	4.1	3.5	987	11
762x267x147	8	1030	8	200	12	650	110	10	4.1	3.5	1100	11
{2040 kN}	7	897	8	200	12	570	110	10	4.1	3.5	965	11
762x267x134	8	1030	8	200	12	650	110	10	4.1	3.5	1090	11
{1970 kN}	7	897	8	200	12	570	110	10	4.1	3.5	954	11
686x254x170	7	897	8	200	12	570	110	10	4.1	3.5	990	11
{2120 kN}	6	769	8	200	12	490	110	10	4.1	3.5	851	11
686x254x152	7	897	8	200	12	570	110	10	4.1	3.5	971	11
{1920 kN}	6	769	8	200	12	490	110	10	4.1	3.5	835	11
686x254x140	7	897	8	200	12	570	110	10	4.1	3.5	960	11
{1790 kN}	6	769	8	200	12	490	110	10	4.1	3.5	825	11
686x254x125	7	897	8	200	12	570	110	8	4.1	3.5	908	11
{1670 kN}	6	769	8	200	12	490	110	8	4.1	3.5	781	11
610x305x238	6	769	8	200	12	490	110	15	4.1	3.5	922	11
{2460 kN}												
610x305x179	6	769	8	200	12	490	110	10	4.1	3.5	846	11
{1880 kN}												
610x305x149	6	769	8	200	12	490	110	10	4.1	3.5	818	11
{1570 kN}												

For guidance on the use of tables see Explanatory notes in Table G.1

BEAM: S355
END PLATES: S275
HOLLO-BOLTS: M20, 8.8

Table G.7 *Continued*

PARTIAL DEPTH END PLATES, HOLLO-BOLTS
200 x 12 or 180 x 10 mm End Plate

Beam size $\{V_{Rd,beam}\}$	Bolt rows n_1	Un-notched Shear Resistance V_{Rd} kN	Un-notched Critical Check	Fitting (End Plate) Width b_p mm	Thck t_p mm	Height h_p mm	Gauge p_3 mm	Web weld Leg s mm	Min support thickness S275 mm	Min support thickness S355 mm	Tying Resistance $N_{Rd,u}$ kN	Tying Critical Check
610x229x140 {1690 kN}	6	769	8	200	12	490	110	10	4.1	3.5	834	11
610x229x125 {1520 kN}	6	769	8	200	12	490	110	10	4.1	3.5	819	11
610x229x113 {1420 kN}	6	769	8	200	12	490	110	8	4.1	3.5	774	11
610x229x101 {1370 kN}	6	769	8	200	12	490	110	8	4.1	3.5	768	11
610x178x100+ {1450 kN}	6	769	8	200	12	490	110	8	4.1	3.5	776	11
610x178x92+ {1410 kN}	6	769	8	200	12	490	110	8	4.1	3.5	772	11
610x178x82+ {1290 kN}	6	769	8	200	12	490	110	8	4.1	3.5	763	11
533x312x272+ {2480 kN}	5	641	8	200	12	410	110	15	4.1	3.5	782	11
533x312x219+ {2120 kN}	5	641	8	200	12	410	110	15	4.1	3.5	769	11
533x312x182+ {1740 kN}	5	641	8	200	12	410	110	12	4.1	3.5	747	11
533x312x150+ {1460 kN}	5	641	8	200	12	410	110	10	4.1	3.5	693	11

For guidance on the use of tables see Explanatory notes in Table G.1

BEAM: S355
END PLATES: S275
HOLLO-BOLTS: M20, 8.8

Table G.7 *Continued*

PARTIAL DEPTH END PLATES, HOLLO-BOLTS
200 x 12 or 180 x 10 mm End Plate

Beam size {$V_{Rd,beam}$}	Bolt rows n_1	Un-notched Shear Resistance V_{Rd} kN	Un-notched Critical Check	Fitting (End Plate) Width b_p mm	Thck t_p mm	Height h_p mm	Gauge p_3 mm	Web weld Leg s mm	Min support thickness S275 mm	Min support thickness S355 mm	Tying Resistance $N_{Rd,u}$ kN	Tying Critical Check
533x210x138+ {1680 kN}	5	641	8	200	12	410	110	10	4.1	3.5	714	11
533x210x122 {1450 kN}	5	641	8	200	12	410	110	10	4.1	3.5	693	11
533x210x109 {1330 kN}	5	641	8	200	12	410	110	8	4.1	3.5	652	11
533x210x101 {1240 kN}	5	641	8	200	12	410	110	8	4.1	3.5	645	11
533x210x92 {1170 kN}	5	641	8	200	12	410	110	8	4.1	3.5	639	11
533x210x82 {1120 kN}	5	641	8	200	12	410	110	8	4.1	3.5	635	11
533x165x85+ {1170 kN}	5	641	8	200	12	410	110	8	4.1	3.5	641	11
533x165x74+ {1120 kN}	5	641	8	200	12	410	110	8	4.1	3.5	636	11
533x165x66+ {1020 kN}	5	641	8	200	12	410	110	8	4.1	3.5	629	11

For guidance on the use of tables see Explanatory notes in Table G.1

BEAM: S355
END PLATES: S275
HOLLO-BOLTS: M20, 8.8

Table G.7 *Continued*

PARTIAL DEPTH END PLATES, HOLLO-BOLTS
200 x 12 or 180 x 10 mm End Plate

Beam size $\{V_{Rd,beam}\}$	Bolt rows n_1	Un-notched: Shear Resistance V_{Rd} kN	Un-notched: Critical Check	Fitting (End Plate): Width b_p mm	Fitting (End Plate): Thck t_p mm	Fitting (End Plate): Height h_p mm	Fitting (End Plate): Gauge p_3 mm	Web weld: Leg s mm	Min support thickness: S275 mm	Min support thickness: S355 mm	Tying: Resistance $N_{Rd,u}$ kN	Tying: Critical Check
457x191x161+ {1810 kN}	4	427	8	180	10	330	90	15	3.4	3.0	612	11
457x191x133+ {1510 kN}	4	427	8	180	10	330	90	12	3.4	3.0	601	11
457x191x106+ {1230 kN}	4	427	8	180	10	330	90	10	3.4	3.0	559	11
457x191x98 {1110 kN}	4	427	8	180	10	330	90	8	3.4	3.0	507	11
457x191x89 {1030 kN}	4	427	8	180	10	330	90	8	3.4	3.0	498	11
457x191x82 {976 kN}	4	427	8	180	10	330	90	8	3.4	3.0	492	11
457x191x74 {895 kN}	4	427	8	180	10	330	90	8	3.4	3.0	483	11
457x191x67 {839 kN}	4	427	8	180	10	330	90	6	3.4	3.0	450	11
457x152x82 {1040 kN}	4	427	8	180	10	330	90	8	3.4	3.0	498	11
457x152x74 {938 kN}	4	427	8	180	10	330	90	8	3.4	3.0	489	11
457x152x67 {899 kN}	4	427	8	180	10	330	90	8	3.4	3.0	483	11
457x152x60 {806 kN}	4	427	8	180	10	330	90	6	3.4	3.0	447	11
457x152x52 {747 kN}	4	427	8	180	10	330	90	6	3.4	3.0	443	11

For guidance on the use of tables see Explanatory notes in Table G.1

BEAM: S355
END PLATES: S275
HOLLO-BOLTS: M20, 8.8

Table G.7 *Continued*

PARTIAL DEPTH END PLATES, HOLLO-BOLTS
200 x 12 or 180 x 10 mm End Plate

Beam size	Bolt rows	Un-notched		Fitting (End Plate)				Web weld	Min support thickness		Tying	
Beam size {$V_{Rd,beam}$}	n_1	Shear Resistance V_{Rd} kN	Critical Check	Width b_p mm	Thck t_p mm	Height h_p mm	Gauge p_3 mm	Leg s mm	S275 mm	S355 mm	Resistance $N_{Rd,u}$ kN	Critical Check
406x178x85+ {966 kN}	4	427	8	180	10	330	90	8	3.4	3.0	502	11
406x178x74 {858 kN}	4	427	8	180	10	330	90	8	3.4	3.0	488	11
406x178x67 {790 kN}	4	427	8	180	10	330	90	6	3.4	3.0	452	11
406x178x60 {709 kN}	4	427	8	180	10	330	90	6	3.4	3.0	445	11
406x178x54 {683 kN}	4	427	8	180	10	330	90	6	3.4	3.0	444	11
406x140x53+ {709 kN}	4	427	8	180	10	330	90	6	3.4	3.0	445	11
406x140x46 {611 kN}	4	414	4	180	10	330	90	6	3.3	2.9	437	11
406x140x39 {566 kN}	4	390	4	180	10	330	90	6	3.1	2.7	434	11
356x171x67 {733 kN}	3	321	8	180	10	250	90	8	3.4	3.0	367	11
356x171x57 {646 kN}	3	321	8	180	10	250	90	6	3.4	3.0	338	11
356x171x51 {587 kN}	3	321	8	180	10	250	90	6	3.4	3.0	334	11
356x171x45 {549 kN}	3	321	8	180	10	250	90	6	3.4	3.0	332	11
356x127x39 {527 kN}	3	304	4	180	10	250	90	6	3.2	2.8	330	11
356x127x33 {472 kN}	3	277	4	180	10	250	90	6	2.9	2.5	327	11

For guidance on the use of tables see Explanatory notes in Table G.1

BEAM: S355
END PLATES: S275
Table G.7 *Continued* **HOLLO-BOLTS: M20, 8.8**

PARTIAL DEPTH END PLATES, HOLLO-BOLTS
200 x 12 or 180 x 10 mm End Plate

Beam size {$V_{\text{Rd,beam}}$}	Bolt rows	Un-notched		Fitting (End Plate)				Web weld	Min support thickness		Tying	
	n_1	Shear Resistance V_{Rd} kN	Critical Check	Width b_p mm	Thck t_p mm	Height h_p mm	Gauge p_3 mm	Leg s mm	S275 mm	S355 mm	Resistance $N_{\text{Rd,u}}$ kN	Critical Check
305x165x54 {545 kN}	2	214	8	180	10	170	90	6	3.4	3.0	229	11
305x165x46 {461 kN}	2	210	4	180	10	170	90	6	3.3	2.9	225	11
305x165x40 {411 kN}	2	188	4	180	10	170	90	6	3.0	2.6	222	11
305x127x48 {612 kN}	2	214	8	180	10	170	90	8	3.4	3.0	249	11
305x127x42 {542 kN}	2	214	8	180	10	170	90	6	3.4	3.0	230	11
305x127x37 {481 kN}	2	214	8	180	10	170	90	6	3.4	3.0	226	11
305x102x33 {452 kN}	3	304	4	180	10	250	90	6	3.2	2.8	330	11
305x102x28 {407 kN}	3	277	4	180	10	250	90	6	2.9	2.5	327	11
305x102x25 {386 kN}	3	267	4	180	10	250	90	6	2.8	2.5	326	11
254x146x43 {415 kN}	2	214	8	180	10	170	90	6	3.4	3.0	227	11
254x146x37 {361 kN}	2	198	4	180	10	170	90	6	3.1	2.7	223	11
254x146x31 {336 kN}	2	188	4	180	10	170	90	6	3.0	2.6	222	11
254x102x28 {365 kN}	2	198	4	180	10	170	90	6	3.1	2.7	223	11
254x102x25 {341 kN}	2	188	4	180	10	170	90	6	3.0	2.6	222	11
254x102x22 {320 kN}	2	179	4	180	10	170	90	6	2.8	2.5	221	11

For guidance on the use of tables see Explanatory notes in Table G.1

Table G.8

Explanatory notes – FULL DEPTH END PLATES
Use of Resistance Tables

The check numbers noted below refer to those listed in Table G.10 and described in Section 4.7 Design procedures.

The resistance tables are based on the standard details given in Table G.9.

1 SHEAR RESISTANCE OF THE BEAM

The value given in { } below each beam designation is the shear resistance of the beam, given by $\dfrac{A_v f_y}{\sqrt{3}\gamma_{M0}}$.

A + symbol adjacent to the beam designation indicates that the serial size is additional to those specified in BS 4-1.

2 SHEAR RESISTANCE OF THE CONNECTION

This is the critical value of the design checks for the 'supported beam side' of the connection, i.e. the minimum resistance from Checks 2, 4 and 8.

3 CRITICAL DESIGN CHECK

The check which gives the critical value of shear resistance. See Table G.10 for the description of the checks.

4 MINIMUM SUPPORT THICKNESS

This is the minimum thickness of supporting column or beam element that is needed to carry the given shear resistance of the connection. It is derived from Check 10 and e_t has been taken as 90 mm.

For a symmetrical two sided connection, the minimum support thickness would be twice the tabulated value.

If the applied shear force is less than the quoted resistance, the minimum support thickness reduces proportionally.

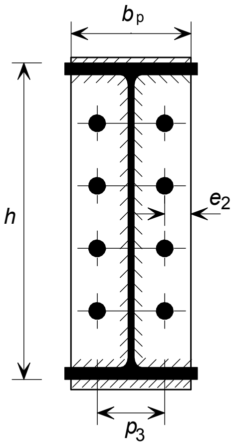
5 TYING RESISTANCE

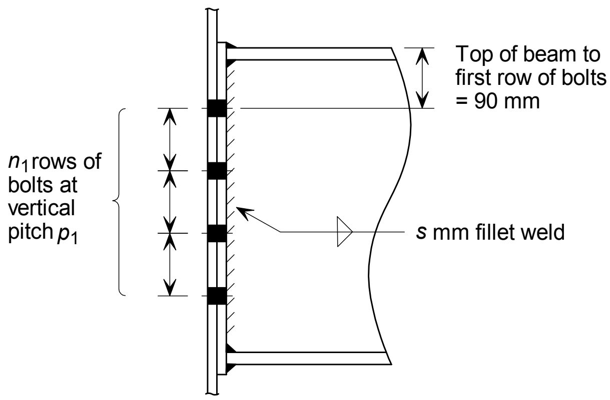
This is the critical value of the design checks for the 'supported beam' side of the connection, i.e. the minimum resistance from Checks 11, 12 and 13. The calculations assume that no washer is present and d_w is taken as the width across points of the bolt head.

Separate checks will have to be carried out on the supporting members (see Checks 14 and 15).

Table G.9

FULL DEPTH END PLATES
Standard details used in Resistance Tables





Ordinary and Flowdrill bolts:
- p_1 = 70 mm
- p_3 = 90 mm for beams ≤ 457 x 191 UB
- p_3 = 140 mm for beams > 457 x 191 UB
- e_2 = 30 mm

M20 8.8 bolts
or
M20 Flowdrill
or
M20 Hollo-Bolt

Hollo-Bolts:
- p_1 = 80 mm
- p_3 = 90 mm for beams ≤ 457 x 191 UB
- p_3 = 110 mm for beams > 457 x 191 UB
- e_2 = 45 mm

Table G.10

<table>
<tr><th colspan="4">FULL DEPTH END PLATES
Design Check List</th></tr>
<tr><th colspan="2"></th><th>Check Number</th><th>Description</th></tr>
<tr><td rowspan="4">SHEAR RESISTANCE</td><td rowspan="3">Supported beam side</td><td>2</td><td>Welds</td></tr>
<tr><td>4</td><td>Web in shear</td></tr>
<tr><td>8</td><td>Bolt group</td></tr>
<tr><td>Supporting member side</td><td>10</td><td>Shear and bearing</td></tr>
<tr><td colspan="2" rowspan="3">TYING RESISTANCE</td><td>11</td><td>Plate and bolts</td></tr>
<tr><td>12</td><td>Supported beam web</td></tr>
<tr><td>13</td><td>Welds</td></tr>
<tr><td colspan="4">Note: This table only lists the critical checks.
For a full list of design checks and further information, see Section 4.5.</td></tr>
</table>

BEAM: S275
END PLATES: S275
BOLTS: M20, 8.8

Table G.11

FULL DEPTH END PLATES, ORDINARY or FLOWDRILL BOLTS
200 x 12 or 150 x 10 mm End Plate

Beam size {$V_{Rd,beam}$}	Bolt rows n_1	Un-notched: Shear Resistance V_{Rd} kN	Un-notched: Critical check	Fitting (End Plate): Width b_p mm	Fitting (End Plate): Thck t_p mm	Fitting (End Plate): Gauge p_3 mm	Web weld: Leg s mm	Min support thickness: S275 mm	Min support thickness: S355 mm	Tying: Resistance $N_{Rd,u}$ kN	Tying: Critical check
1016x305x487+	11	1660	8	200	12	140	18	5.7	4.9	1550	11
{4930 kN}	10	1510	8	200	12	140	18	5.7	4.9	1430	11
	9	1350	8	200	12	140	18	5.7	4.9	1320	11
	8	1200	8	200	12	140	18	5.7	4.9	1210	11
1016x305x437+	11	1660	8	200	12	140	18	5.7	4.9	1480	11
{4420 kN}	10	1510	8	200	12	140	18	5.7	4.9	1370	11
	9	1350	8	200	12	140	18	5.7	4.9	1260	11
	8	1200	8	200	12	140	18	5.7	4.9	1160	11
1016x305x393+	11	1660	8	200	12	140	15	5.7	4.9	1350	11
{3990 kN}	10	1510	8	200	12	140	15	5.7	4.9	1250	11
	9	1350	8	200	12	140	15	5.7	4.9	1150	11
	8	1200	8	200	12	140	15	5.7	4.9	1050	11
1016x305x349+	11	1660	8	200	12	140	12	5.7	4.9	1230	11
{3610 kN}	10	1510	8	200	12	140	12	5.7	4.9	1140	11
	9	1350	8	200	12	140	12	5.7	4.9	1050	11
	8	1200	8	200	12	140	12	5.7	4.9	965	11
1016x305x314+	11	1660	8	200	12	140	12	5.7	4.9	1210	11
{3260 kN}	10	1510	8	200	12	140	12	5.7	4.9	1120	11
	9	1350	8	200	12	140	12	5.7	4.9	1030	11
	8	1200	8	200	12	140	12	5.7	4.9	944	11
1016x305x272+	11	1660	8	200	12	140	10	5.7	4.9	1150	11
{2830 kN}	10	1510	8	200	12	140	10	5.7	4.9	1060	11
	9	1350	8	200	12	140	10	5.7	4.9	980	11
	8	1200	8	200	12	140	10	5.7	4.9	897	11
1016x305x249+	11	1660	8	200	12	140	10	5.7	4.9	1140	11
{2770 kN}	10	1510	8	200	12	140	10	5.7	4.9	1060	11
	9	1350	8	200	12	140	10	5.7	4.9	976	11
	8	1200	8	200	12	140	10	5.7	4.9	893	11
1016x305x222+	11	1660	8	200	12	140	10	5.7	4.9	1130	11
{2640 kN}	10	1510	8	200	12	140	10	5.7	4.9	1050	11
	9	1350	8	200	12	140	10	5.7	4.9	969	11
	8	1200	8	200	12	140	10	5.7	4.9	887	11

For guidance on the use of tables see Explanatory notes in Table G.8

BEAM: S275
END PLATES: S275
BOLTS: M20, 8.8

Table G.11 *Continued*

FULL DEPTH END PLATES, ORDINARY or FLOWDRILL BOLTS
200 x 12 or 150 x 10 mm End Plate

Beam size $\{V_{Rd,beam}\}$	Bolt rows n_1	Un-notched: Shear Resistance V_{Rd} kN	Un-notched: Critical check	Fitting (End Plate): Width b_p mm	Fitting (End Plate): Thck t_p mm	Fitting (End Plate): Gauge p_3 mm	Web weld: Leg s mm	Min support thickness: S275 mm	Min support thickness: S355 mm	Tying: Resistance $N_{Rd,u}$ kN	Tying: Critical check
914x419x388	11	1660	8	200	12	140	15	5.7	4.9	1300	11
{3240 kN}	10	1510	8	200	12	140	15	5.7	4.9	1200	11
	9	1350	8	200	12	140	15	5.7	4.9	1100	11
	8	1200	8	200	12	140	15	5.7	4.9	1010	11
914x419x343	11	1660	8	200	12	140	12	5.7	4.9	1210	11
{2920 kN}	10	1510	8	200	12	140	12	5.7	4.9	1120	11
	9	1350	8	200	12	140	12	5.7	4.9	1030	11
	8	1200	8	200	12	140	12	5.7	4.9	943	11
914x305x289	11	1660	8	200	12	140	12	5.7	4.9	1210	11
{2900 kN}	10	1510	8	200	12	140	12	5.7	4.9	1120	11
	9	1350	8	200	12	140	12	5.7	4.9	1030	11
	8	1200	8	200	12	140	12	5.7	4.9	943	11
914x305x253	11	1660	8	200	12	140	10	5.7	4.9	1150	11
{2570 kN}	10	1510	8	200	12	140	10	5.7	4.9	1070	11
	9	1350	8	200	12	140	10	5.7	4.9	983	11
	8	1200	8	200	12	140	10	5.7	4.9	900	11
914x305x224	11	1660	8	200	12	140	10	5.7	4.9	1140	11
{2350 kN}	10	1510	8	200	12	140	10	5.7	4.9	1050	11
	9	1350	8	200	12	140	10	5.7	4.9	970	11
	8	1200	8	200	12	140	10	5.7	4.9	888	11
914x305x201	11	1660	8	200	12	140	10	5.7	4.9	1130	11
{2210 kN}	10	1510	8	200	12	140	10	5.7	4.9	1040	11
	9	1350	8	200	12	140	10	5.7	4.9	962	11
	8	1200	8	200	12	140	10	5.7	4.9	880	11
838x292x226	10	1510	8	200	12	140	10	5.7	4.9	1060	11
{2220 kN}	9	1350	8	200	12	140	10	5.7	4.9	973	11
	8	1200	8	200	12	140	10	5.7	4.9	891	11
838x292x194	10	1510	8	200	12	140	10	5.7	4.9	1040	11
{2000 kN}	9	1350	8	200	12	140	10	5.7	4.9	960	11
	8	1200	8	200	12	140	10	5.7	4.9	879	11
838x292x176	10	1510	8	200	12	140	8	5.7	4.9	1010	11
{1890 kN}	9	1350	8	200	12	140	8	5.7	4.9	930	11
	8	1200	8	200	12	140	8	5.7	4.9	852	11

For guidance on the use of tables see Explanatory notes in Table G.8

BEAM: S275
END PLATES: S275
BOLTS: M20, 8.8

Table G.11 *Continued*

FULL DEPTH END PLATES, ORDINARY or FLOWDRILL BOLTS
200 x 12 or 150 x 10 mm End Plate

	Bolt rows	Un-notched		Fitting (End Plate)			Web weld	Min support thickness		Tying	
Beam size {$V_{Rd,beam}$}	n_1	Shear Resistance V_{Rd} kN	Critical check	Width b_p mm	Thck t_p mm	Gauge p_3 mm	Leg s mm	S275 mm	S355 mm	Resistance $N_{Rd,u}$ kN	Critical check
762x267x197	9	1350	8	200	12	140	10	5.7	4.9	971	11
{1950 kN}	8	1200	8	200	12	140	10	5.7	4.9	887	11
	7	1050	8	200	12	140	10	5.7	4.9	805	11
762x267x173	9	1350	8	200	12	140	10	5.7	4.9	960	11
{1760 kN}	8	1200	8	200	12	140	10	5.7	4.9	876	11
	7	1050	8	200	12	140	10	5.7	4.9	795	11
762x267x147	9	1350	8	200	12	140	8	5.7	4.9	926	11
{1560 kN}	8	1200	8	200	12	140	8	5.7	4.9	845	11
	7	1050	8	200	12	140	8	5.7	4.9	767	11
762x267x134	9	1350	8	200	12	140	8	5.7	4.9	920	11
{1520 kN}	8	1200	8	200	12	140	8	5.7	4.9	839	11
	7	1050	8	200	12	140	8	5.7	4.9	762	11
686x254x170	8	1200	8	200	12	140	10	5.7	4.9	882	11
{1630 kN}	7	1050	8	200	12	140	10	5.7	4.9	797	11
	6	903	8	200	12	140	10	5.7	4.9	716	11
686x254x152	8	1200	8	200	12	140	8	5.7	4.9	852	11
{1470 kN}	7	1050	8	200	12	140	8	5.7	4.9	771	11
	6	903	8	200	12	140	8	5.7	4.9	693	11
686x254x140	8	1200	8	200	12	140	8	5.7	4.9	847	11
{1370 kN}	7	1050	8	200	12	140	8	5.7	4.9	766	11
	6	903	8	200	12	140	8	5.7	4.9	689	11
686x254x125	8	1200	8	200	12	140	8	5.7	4.9	843	11
{1280 kN}	7	1050	8	200	12	140	8	5.7	4.9	761	11
	6	903	8	200	12	140	8	5.7	4.9	685	11
610x305x238	7	1050	8	200	12	140	12	5.7	4.9	850	11
{1890 kN}	6	903	8	200	12	140	12	5.7	4.9	760	11
610x305x179	7	1050	8	200	12	140	8	5.7	4.9	781	11
{1440 kN}	6	903	8	200	12	140	8	5.7	4.9	699	11
610x305x149	7	1050	8	200	12	140	8	5.7	4.9	768	11
{1200 kN}	6	903	8	200	12	140	8	5.7	4.9	687	11

For guidance on the use of tables see Explanatory notes in Table G.8

BEAM: S275
END PLATES: S275
BOLTS: M20, 8.8

Table G.11 *Continued*

FULL DEPTH END PLATES, ORDINARY or FLOWDRILL BOLTS
200 x 12 or 150 x 10 mm End Plate

Beam size	Bolt rows	Un-notched		Fitting (End Plate)			Web weld	Min support thickness		Tying	
		Shear Resistance	Critical check	Width	Thck	Gauge	Leg	S275	S355	Resistance	Critical check
{$V_{Rd,beam}$}	n_1	V_{Rd} kN		b_p mm	t_p mm	p_3 mm	s mm	mm	mm	$N_{Rd,u}$ kN	
610x229x140	7	1050	8	200	12	140	8	5.7	4.9	775	11
{1300 kN}	6	903	8	200	12	140	8	5.7	4.9	693	11
610x229x125	7	1050	8	200	12	140	8	5.7	4.9	769	11
{1170 kN}	6	903	8	200	12	140	8	5.7	4.9	687	11
610x229x113	7	1050	8	200	12	140	8	5.7	4.9	764	11
{1090 kN}	6	903	8	200	12	140	8	5.7	4.9	683	11
610x229x101	7	1050	8	200	12	140	6	5.7	4.9	745	11
{1060 kN}	6	903	8	200	12	140	6	5.7	4.9	666	11
610x178x100+	7	1050	8	200	12	140	8	5.7	4.9	765	11
{1110 kN}	6	903	8	200	12	140	8	5.7	4.9	683	11
610x178x92+	7	1050	8	200	12	140	8	5.7	4.9	763	11
{1090 kN}	6	903	8	200	12	140	8	5.7	4.9	681	11
610x178x82+	7	1000	4	200	12	140	6	5.4	4.7	743	11
{1000 kN}	6	903	8	200	12	140	6	5.7	4.9	663	11
533x312x272+	6	903	8	200	12	140	12	5.7	4.9	783	11
{1910 kN}	5	753	8	200	12	140	12	5.7	4.9	692	11
533x312x219+	6	903	8	200	12	140	12	5.7	4.9	761	11
{1630 kN}	5	753	8	200	12	140	12	5.7	4.9	670	11
533x312x182+	6	903	8	200	12	140	10	5.7	4.9	724	11
{1340 kN}	5	753	8	200	12	140	10	5.7	4.9	638	11
533x312x150+	6	903	8	200	12	140	8	5.7	4.9	696	11
{1120 kN}	5	753	8	200	12	140	8	5.7	4.9	613	11

For guidance on the use of tables see Explanatory notes in Table G.8

BEAM: S275
END PLATES: S275
Table G.11 *Continued* **BOLTS: M20, 8.8**

FULL DEPTH END PLATES, ORDINARY or FLOWDRILL BOLTS
200 x 12 or 150 x 10 mm End Plate

Beam size {$V_{Rd,beam}$}	Bolt rows n_1	Un-notched: Shear Resistance V_{Rd} kN	Un-notched: Critical check	Fitting (End Plate): Width b_p mm	Fitting (End Plate): Thck t_p mm	Fitting (End Plate): Gauge p_3 mm	Web weld: Leg s mm	Min support thickness: S275 mm	Min support thickness: S355 mm	Tying: Resistance $N_{Rd,u}$ kN	Tying: Critical check
533x210x138+	6	903	8	200	12	140	10	5.7	4.9	722	11
{1290 kN}	5	753	8	200	12	140	10	5.7	4.9	636	11
533x210x122	6	903	8	200	12	140	8	5.7	4.9	696	11
{1110 kN}	5	753	8	200	12	140	8	5.7	4.9	614	11
533x210x109	6	903	8	200	12	140	8	5.7	4.9	691	11
{1020 kN}	5	753	8	200	12	140	8	5.7	4.9	609	11
533x210x101	6	903	8	200	12	140	8	5.7	4.9	687	11
{952 kN}	5	753	8	200	12	140	8	5.7	4.9	606	11
533x210x92	6	903	8	200	12	140	6	5.7	4.9	670	11
{909 kN}	5	753	8	200	12	140	6	5.7	4.9	591	11
533x210x82	6	865	4	200	12	140	6	5.4	4.7	668	11
{865 kN}	5	753	8	200	12	140	6	5.7	4.9	589	11
533x165x85+	6	902	4	150	10	90	6	5.7	4.9	710	11
{902 kN}	5	753	8	150	10	90	6	5.7	4.9	613	11
533x165x74+	6	871	4	150	10	90	6	5.5	4.8	705	11
{871 kN}	5	753	8	150	10	90	6	5.7	4.9	608	11
533x165x66+	6	793	4	150	10	90	6	5.0	4.3	698	11
{793 kN}	5	753	8	150	10	90	6	5.7	4.9	602	11

For guidance on the use of tables see Explanatory notes in Table G.8

BEAM: S275
END PLATES: S275
BOLTS: M20, 8.8

Table G.11 *Continued*

FULL DEPTH END PLATES, ORDINARY or FLOWDRILL BOLTS
200 x 12 or 150 x 10 mm End Plate

Beam size $\{V_{Rd,beam}\}$	Bolt rows n_1	Un-notched: Shear Resistance V_{Rd} kN	Un-notched: Critical check	Fitting (End Plate): Width b_p mm	Fitting (End Plate): Thck t_p mm	Fitting (End Plate): Gauge p_3 mm	Web weld: Leg s mm	Min support thickness: S275 mm	Min support thickness: S355 mm	Tying: Resistance $N_{Rd,u}$ kN	Tying: Critical check
457x191x161+	5	753	8	150	10	90	12	5.7	4.9	801	11
{1390 kN}	4	602	8	150	10	90	12	5.7	4.9	710	11
457x191x133+	5	753	8	150	10	90	10	5.7	4.9	724	11
{1160 kN}	4	602	8	150	10	90	10	5.7	4.9	606	11
457x191x106+	5	753	8	150	10	90	8	5.7	4.9	664	11
{947 kN}	4	602	8	150	10	90	8	5.7	4.9	557	11
457x191x98	5	753	8	150	10	90	8	5.7	4.9	653	11
{852 kN}	4	602	8	150	10	90	8	5.7	4.9	548	11
457x191x89	5	753	8	150	10	90	6	5.7	4.9	619	11
{789 kN}	4	602	8	150	10	90	6	5.7	4.9	522	11
457x191x82	5	753	8	150	10	90	6	5.7	4.9	615	11
{756 kN}	4	602	8	150	10	90	6	5.7	4.9	518	11
457x191x74	5	693	4	150	10	90	6	5.2	4.5	608	11
{693 kN}	4	602	8	150	10	90	6	5.7	4.9	512	11
457x191x67	5	650	4	150	10	90	6	4.9	4.3	605	11
{650 kN}	4	602	8	150	10	90	6	5.7	4.9	509	11
457x152x82	5	753	8	150	10	90	6	5.7	4.9	619	11
{798 kN}	4	602	8	150	10	90	6	5.7	4.9	522	11
457x152x74	5	721	4	150	10	90	6	5.4	4.7	612	11
{721 kN}	4	602	8	150	10	90	6	5.7	4.9	516	11
457x152x67	5	697	4	150	10	90	6	5.2	4.6	608	11
{697 kN}	4	602	8	150	10	90	6	5.7	4.9	512	11
457x152x60	5	624	4	150	10	90	6	4.7	4.1	602	11
{624 kN}	4	602	8	150	10	90	6	5.7	4.9	507	11
457x152x52	5	578	4	150	10	90	6	4.4	3.8	599	11
{578 kN}	4	578	4	150	10	90	6	5.4	4.7	504	11

For guidance on the use of tables see Explanatory notes in Table G.8

BEAM: S275
END PLATES: S275
BOLTS: M20, 8.8

Table G.11 *Continued*

FULL DEPTH END PLATES, ORDINARY or FLOWDRILL BOLTS
200 x 12 or 150 x 10 mm End Plate

Beam size {$V_{Rd,beam}$}	Bolt rows n_1	Un-notched: Shear Resistance V_{Rd} kN	Un-notched: Critical check	Fitting (End Plate): Width b_p mm	Fitting (End Plate): Thck t_p mm	Fitting (End Plate): Gauge p_3 mm	Web weld: Leg s mm	Min support thickness: S275 mm	Min support thickness: S355 mm	Tying: Resistance $N_{Rd,u}$ kN	Tying: Critical check
406x178x85+ {742 kN}	4	602	8	150	10	90	8	5.7	4.9	545	11
406x178x74 {664 kN}	4	602	8	150	10	90	6	5.7	4.9	515	11
406x178x67 {612 kN}	4	602	8	150	10	90	6	5.7	4.9	511	11
406x178x60 {549 kN}	4	549	4	150	10	90	6	5.2	4.5	506	11
406x178x54 {529 kN}	4	529	4	150	10	90	6	5.0	4.3	505	11
406x140x53+ {549 kN}	4	549	4	150	10	90	6	5.2	4.5	506	11
406x140x46 {473 kN}	4	473	4	150	10	90	6	4.4	3.9	501	11
406x140x39 {438 kN}	4	438	4	150	10	90	6	4.1	3.6	499	11
356x171x67 {568 kN}	3	452	8	150	10	90	6	5.7	4.9	422	11
356x171x57 {501 kN}	3	452	8	150	10	90	6	5.7	4.9	417	11
356x171x51 {455 kN}	3	452	8	150	10	90	6	5.7	4.9	414	11
356x171x45 {425 kN}	3	425	4	150	10	90	6	5.3	4.6	412	11
356x127x39 {408 kN}	3	408	4	150	10	90	6	5.1	4.5	411	11
356x127x33 {366 kN}	3	366	4	150	10	90	6	4.6	4.0	408	11

For guidance on the use of tables see Explanatory notes in Table G.8

BEAM: S275
END PLATES: S275
BOLTS: M20, 8.8

Table G.11 *Continued*

FULL DEPTH END PLATES, ORDINARY or FLOWDRILL BOLTS
200 x 12 or 150 x 10 mm End Plate

Beam size {$V_{Rd,beam}$}	Bolt rows	Un-notched		Fitting (End Plate)			Web weld	Min support thickness		Tying	
	n_1	Shear Resistance V_{Rd} kN	Critical check	Width b_p mm	Thck t_p mm	Gauge p_3 mm	Leg s mm	S275 mm	S355 mm	Resistance $N_{Rd,u}$ kN	Critical check
305x165x54 {422 kN}	3	422	4	150	10	90	6	5.3	4.6	423	11
305x165x46 {357 kN}	3	357	4	150	10	90	6	4.5	3.9	418	11
305x165x40 {319 kN}	3	319	4	150	10	90	6	4.0	3.5	416	11
305x127x48 {474 kN}	3	452	8	150	10	90	6	5.7	4.9	427	11
305x127x42 {420 kN}	3	420	4	150	10	90	6	5.3	4.6	423	11
305x127x37 {372 kN}	3	372	4	150	10	90	6	4.7	4.1	420	11
305x102x33 {350 kN}	3	350	4	150	10	90	6	4.4	3.8	416	11
305x102x28 {315 kN}	3	315	4	150	10	90	6	4.0	3.4	414	11
305x102x25 {299 kN}	3	299	4	150	10	90	6	3.8	3.3	413	11
254x146x43 {321 kN}	2	301	8	150	10	90	6	5.7	4.9	327	11
254x146x37 {280 kN}	2	280	4	150	10	90	6	5.3	4.6	325	11
254x146x31 {260 kN}	2	260	4	150	10	90	6	4.9	4.3	324	11
254x102x28 {283 kN}	2	283	4	150	10	90	6	5.3	4.6	324	11
254x102x25 {265 kN}	2	265	4	150	10	90	6	5.0	4.3	323	11
254x102x22 {248 kN}	2	248	4	150	10	90	6	4.7	4.1	322	11

For guidance on the use of tables see Explanatory notes in Table G.8

BEAM: S355
END PLATES: S275
BOLTS: M20, 8.8

Table G.12

FULL DEPTH END PLATES, ORDINARY or FLOWDRILL BOLTS
200 x 12 or 150 x 10 mm End Plate

Beam size	Bolt rows	Un-notched		Fitting (End Plate)			Web weld	Min support thickness		Tying	
Beam size {$V_{Rd,beam}$}	n_1	Shear Resistance V_{Rd} kN	Critical check	Width b_p mm	Thck t_p mm	Gauge p_3 mm	Leg s mm	S275 mm	S355 mm	Resistance $N_{Rd,u}$ kN	Critical check
1016x305x487+	11	1660	8	200	12	140	22	5.7	4.9	1660	11
{6480 kN}	10	1510	8	200	12	140	22	5.7	4.9	1540	11
	9	1350	8	200	12	140	22	5.7	4.9	1410	11
	8	1200	8	200	12	140	22	5.7	4.9	1290	11
1016x305x437+	11	1660	8	200	12	140	20	5.7	4.9	1540	11
{5810 kN}	10	1510	8	200	12	140	20	5.7	4.9	1430	11
	9	1350	8	200	12	140	20	5.7	4.9	1320	11
	8	1200	8	200	12	140	20	5.7	4.9	1200	11
1016x305x393+	11	1660	8	200	12	140	18	5.7	4.9	1420	11
{5240 kN}	10	1510	8	200	12	140	18	5.7	4.9	1320	11
	9	1350	8	200	12	140	18	5.7	4.9	1210	11
	8	1200	8	200	12	140	18	5.7	4.9	1110	11
1016x305x349+	11	1660	8	200	12	140	15	5.7	4.9	1300	11
{4700 kN}	10	1510	8	200	12	140	15	5.7	4.9	1200	11
	9	1350	8	200	12	140	15	5.7	4.9	1110	11
	8	1200	8	200	12	140	15	5.7	4.9	1010	11
1016x305x314+	11	1660	8	200	12	140	15	5.7	4.9	1260	11
{4240 kN}	10	1510	8	200	12	140	15	5.7	4.9	1170	11
	9	1350	8	200	12	140	15	5.7	4.9	1080	11
	8	1200	8	200	12	140	15	5.7	4.9	986	11
1016x305x272+	11	1660	8	200	12	140	12	5.7	4.9	1180	11
{3680 kN}	10	1510	8	200	12	140	12	5.7	4.9	1090	11
	9	1350	8	200	12	140	12	5.7	4.9	1010	11
	8	1200	8	200	12	140	12	5.7	4.9	921	11
1016x305x249+	11	1660	8	200	12	140	12	5.7	4.9	1170	11
{3600 kN}	10	1510	8	200	12	140	12	5.7	4.9	1090	11
	9	1350	8	200	12	140	12	5.7	4.9	1000	11
	8	1200	8	200	12	140	12	5.7	4.9	916	11
1016x305x222+	11	1660	8	200	12	140	12	5.7	4.9	1170	11
{3430 kN}	10	1510	8	200	12	140	12	5.7	4.9	1080	11
	9	1350	8	200	12	140	12	5.7	4.9	995	11
	8	1200	8	200	12	140	12	5.7	4.9	909	11

For guidance on the use of tables see Explanatory notes in Table G.8

BEAM: S355
END PLATES: S275
BOLTS: M20, 8.8

Table G.12 *Continued*

FULL DEPTH END PLATES, ORDINARY or FLOWDRILL BOLTS
200 x 12 or 150 x 10 mm End Plate

	Bolt rows	Un-notched		Fitting (End Plate)			Web weld	Min support thickness		Tying	
Beam size {$V_{Rd,beam}$}	n_1	Shear Resistance V_{Rd} kN	Critical check	Width b_p mm	Thck t_p mm	Gauge p_3 mm	Leg s mm	S275 mm	S355 mm	Resistance $N_{Rd,u}$ kN	Critical check
914x419x388	11	1660	8	200	12	140	15	5.7	4.9	1300	11
{4220 kN}	10	1510	8	200	12	140	15	5.7	4.9	1200	11
	9	1350	8	200	12	140	15	5.7	4.9	1100	11
	8	1200	8	200	12	140	15	5.7	4.9	1010	11
914x419x343	11	1660	8	200	12	140	15	5.7	4.9	1270	11
{3800 kN}	10	1510	8	200	12	140	15	5.7	4.9	1170	11
	9	1350	8	200	12	140	15	5.7	4.9	1080	11
	8	1200	8	200	12	140	15	5.7	4.9	984	11
914x305x289	11	1660	8	200	12	140	15	5.7	4.9	1270	11
{3780 kN}	10	1510	8	200	12	140	15	5.7	4.9	1170	11
	9	1350	8	200	12	140	15	5.7	4.9	1080	11
	8	1200	8	200	12	140	15	5.7	4.9	985	11
914x305x253	11	1660	8	200	12	140	12	5.7	4.9	1180	11
{3350 kN}	10	1510	8	200	12	140	12	5.7	4.9	1100	11
	9	1350	8	200	12	140	12	5.7	4.9	1010	11
	8	1200	8	200	12	140	12	5.7	4.9	924	11
914x305x224	11	1660	8	200	12	140	12	5.7	4.9	1170	11
{3060 kN}	10	1510	8	200	12	140	12	5.7	4.9	1080	11
	9	1350	8	200	12	140	12	5.7	4.9	996	11
	8	1200	8	200	12	140	12	5.7	4.9	911	11
914x305x201	11	1660	8	200	12	140	12	5.7	4.9	1160	11
{2870 kN}	10	1510	8	200	12	140	12	5.7	4.9	1070	11
	9	1350	8	200	12	140	12	5.7	4.9	987	11
	8	1200	8	200	12	140	12	5.7	4.9	903	11
838x292x226	10	1510	8	200	12	140	12	5.7	4.9	1090	11
{2900 kN}	9	1350	8	200	12	140	12	5.7	4.9	999	11
	8	1200	8	200	12	140	12	5.7	4.9	914	11
838x292x194	10	1510	8	200	12	140	10	5.7	4.9	1040	11
{2610 kN}	9	1350	8	200	12	140	10	5.7	4.9	960	11
	8	1200	8	200	12	140	10	5.7	4.9	879	11
838x292x176	10	1510	8	200	12	140	10	5.7	4.9	1040	11
{2460 kN}	9	1350	8	200	12	140	10	5.7	4.9	953	11
	8	1200	8	200	12	140	10	5.7	4.9	873	11

For guidance on the use of tables see Explanatory notes in Table G.8

BEAM: S355
END PLATES: S275
BOLTS: M20, 8.8

Table G.12 *Continued*

FULL DEPTH END PLATES, ORDINARY or FLOWDRILL BOLTS
200 x 12 or 150 x 10 mm End Plate

	Bolt rows	Un-notched		Fitting (End Plate)			Web weld	Min support thickness		Tying	
Beam size {$V_{Rd,beam}$}	n_1	Shear Resistance V_{Rd} kN	Critical check	Width b_p mm	Thck t_p mm	Gauge p_3 mm	Leg s mm	S275 mm	S355 mm	Resistance $N_{Rd,u}$ kN	Critical check
762x267x197	9	1350	8	200	12	140	12	5.7	4.9	997	11
{2530 kN}	8	1200	8	200	12	140	12	5.7	4.9	910	11
	7	1050	8	200	12	140	12	5.7	4.9	825	11
762x267x173	9	1350	8	200	12	140	10	5.7	4.9	960	11
{2290 kN}	8	1200	8	200	12	140	10	5.7	4.9	876	11
	7	1050	8	200	12	140	10	5.7	4.9	795	11
762x267x147	9	1350	8	200	12	140	10	5.7	4.9	949	11
{2040 kN}	8	1200	8	200	12	140	10	5.7	4.9	865	11
	7	1050	8	200	12	140	10	5.7	4.9	785	11
762x267x134	9	1350	8	200	12	140	10	5.7	4.9	943	11
{1970 kN}	8	1200	8	200	12	140	10	5.7	4.9	859	11
	7	1050	8	200	12	140	10	5.7	4.9	780	11
686x254x170	8	1200	8	200	12	140	10	5.7	4.9	882	11
{2120 kN}	7	1050	8	200	12	140	10	5.7	4.9	797	11
	6	903	8	200	12	140	10	5.7	4.9	716	11
686x254x152	8	1200	8	200	12	140	10	5.7	4.9	873	11
{1920 kN}	7	1050	8	200	12	140	10	5.7	4.9	789	11
	6	903	8	200	12	140	10	5.7	4.9	709	11
686x254x140	8	1200	8	200	12	140	10	5.7	4.9	868	11
{1790 kN}	7	1050	8	200	12	140	10	5.7	4.9	783	11
	6	903	8	200	12	140	10	5.7	4.9	704	11
686x254x125	8	1200	8	200	12	140	8	5.7	4.9	843	11
{1670 kN}	7	1050	8	200	12	140	8	5.7	4.9	761	11
	6	903	8	200	12	140	8	5.7	4.9	685	11
610x305x238	7	1050	8	200	12	140	15	5.7	4.9	886	11
{2460 kN}	6	903	8	200	12	140	15	5.7	4.9	791	11
610x305x179	7	1050	8	200	12	140	10	5.7	4.9	799	11
{1880 kN}	6	903	8	200	12	140	10	5.7	4.9	714	11
610x305x149	7	1050	8	200	12	140	10	5.7	4.9	786	11
{1570 kN}	6	903	8	200	12	140	10	5.7	4.9	702	11

For guidance on the use of tables see Explanatory notes in Table G.8

BEAM: S355
END PLATES: S275
BOLTS: M20, 8.8

Table G.12 *Continued*

FULL DEPTH END PLATES, ORDINARY or FLOWDRILL BOLTS
200 x 12 or 150 x 10 mm End Plate

Beam size {$V_{Rd,beam}$}	Bolt rows n_1	Un-notched: Shear Resistance V_{Rd} kN	Un-notched: Critical check	Fitting (End Plate): Width b_p mm	Fitting (End Plate): Thck t_p mm	Fitting (End Plate): Gauge p_3 mm	Web weld: Leg s mm	Min support thickness: S275 mm	Min support thickness: S355 mm	Tying: Resistance $N_{Rd,u}$ kN	Tying: Critical check
610x229x140	7	1050	8	200	12	140	10	5.7	4.9	794	11
{1690 kN}	6	903	8	200	12	140	10	5.7	4.9	709	11
610x229x125	7	1050	8	200	12	140	10	5.7	4.9	787	11
{1520 kN}	6	903	8	200	12	140	10	5.7	4.9	702	11
610x229x113	7	1050	8	200	12	140	8	5.7	4.9	764	11
{1420 kN}	6	903	8	200	12	140	8	5.7	4.9	683	11
610x229x101	7	1050	8	200	12	140	8	5.7	4.9	761	11
{1370 kN}	6	903	8	200	12	140	8	5.7	4.9	679	11
610x178x100+	7	1050	8	200	12	140	8	5.7	4.9	765	11
{1450 kN}	6	903	8	200	12	140	8	5.7	4.9	683	11
610x178x92+	7	1050	8	200	12	140	8	5.7	4.9	763	11
{1410 kN}	6	903	8	200	12	140	8	5.7	4.9	681	11
610x178x82+	7	1050	8	200	12	140	8	5.7	4.9	759	11
{1290 kN}	6	903	8	200	12	140	8	5.7	4.9	676	11
533x312x272+	6	903	8	200	12	140	15	5.7	4.9	818	11
{2480 kN}	5	753	8	200	12	140	15	5.7	4.9	720	11
533x312x219+	6	903	8	200	12	140	15	5.7	4.9	792	11
{2120 kN}	5	753	8	200	12	140	15	5.7	4.9	696	11
533x312x182+	6	903	8	200	12	140	12	5.7	4.9	742	11
{1740 kN}	5	753	8	200	12	140	12	5.7	4.9	653	11
533x312x150+	6	903	8	200	12	140	10	5.7	4.9	711	11
{1460 kN}	5	753	8	200	12	140	10	5.7	4.9	626	11

For guidance on the use of tables see Explanatory notes in Table G.8

BEAM: S355
END PLATES: S275
BOLTS: M20, 8.8

Table G.12 *Continued*

FULL DEPTH END PLATES, ORDINARY or FLOWDRILL BOLTS
200 x 12 or 150 x 10 mm End Plate

Beam size $\{V_{\mathrm{Rd,beam}}\}$	Bolt rows n_1	Un-notched: Shear Resistance V_{Rd} kN	Un-notched: Critical check	Fitting (End Plate): Width b_p mm	Fitting (End Plate): Thck t_p mm	Fitting (End Plate): Gauge p_3 mm	Web weld: Leg s mm	Min support thickness: S275 mm	Min support thickness: S355 mm	Tying: Resistance $N_{\mathrm{Rd,u}}$ kN	Tying: Critical check
533x210x138+	6	903	8	200	12	140	10	5.7	4.9	722	11
{1680 kN}	5	753	8	200	12	140	10	5.7	4.9	636	11
533x210x122	6	903	8	200	12	140	10	5.7	4.9	712	11
{1450 kN}	5	753	8	200	12	140	10	5.7	4.9	627	11
533x210x109	6	903	8	200	12	140	8	5.7	4.9	691	11
{1330 kN}	5	753	8	200	12	140	8	5.7	4.9	609	11
533x210x101	6	903	8	200	12	140	8	5.7	4.9	687	11
{1240 kN}	5	753	8	200	12	140	8	5.7	4.9	606	11
533x210x92	6	903	8	200	12	140	8	5.7	4.9	684	11
{1170 kN}	5	753	8	200	12	140	8	5.7	4.9	602	11
533x210x82	6	903	8	200	12	140	8	5.7	4.9	682	11
{1120 kN}	5	753	8	200	12	140	8	5.7	4.9	600	11
533x165x85+	6	903	8	150	10	90	8	5.7	4.9	741	11
{1170 kN}	5	753	8	150	10	90	8	5.7	4.9	639	11
533x165x74+	6	903	8	150	10	90	8	5.7	4.9	735	11
{1120 kN}	5	753	8	150	10	90	8	5.7	4.9	633	11
533x165x66+	6	903	8	150	10	90	8	5.7	4.9	727	11
{1020 kN}	5	753	8	150	10	90	8	5.7	4.9	626	11

For guidance on the use of tables see Explanatory notes in Table G.8

BEAM: S355
END PLATES: S275
BOLTS: M20, 8.8

Table G.12 *Continued*

FULL DEPTH END PLATES, ORDINARY or FLOWDRILL BOLTS
200 x 12 or 150 x 10 mm End Plate

Beam size	Bolt rows	Un-notched		Fitting (End Plate)			Web weld	Min support thickness		Tying	
{$V_{Rd,beam}$}	n_1	Shear Resistance V_{Rd} kN	Critical check	Width b_p mm	Thck t_p mm	Gauge p_3 mm	Leg s mm	S275 mm	S355 mm	Resistance $N_{Rd,u}$ kN	Critical check
457x191x161+	5	753	8	150	10	90	15	5.7	4.9	880	11
{1810 kN}	4	602	8	150	10	90	15	5.7	4.9	766	11
457x191x133+	5	753	8	150	10	90	12	5.7	4.9	764	11
{1510 kN}	4	602	8	150	10	90	12	5.7	4.9	637	11
457x191x106+	5	753	8	150	10	90	10	5.7	4.9	695	11
{1230 kN}	4	602	8	150	10	90	10	5.7	4.9	582	11
457x191x98	5	753	8	150	10	90	8	5.7	4.9	653	11
{1110 kN}	4	602	8	150	10	90	8	5.7	4.9	548	11
457x191x89	5	753	8	150	10	90	8	5.7	4.9	645	11
{1030 kN}	4	602	8	150	10	90	8	5.7	4.9	542	11
457x191x82	5	753	8	150	10	90	8	5.7	4.9	640	11
{976 kN}	4	602	8	150	10	90	8	5.7	4.9	538	11
457x191x74	5	753	8	150	10	90	8	5.7	4.9	633	11
{895 kN}	4	602	8	150	10	90	8	5.7	4.9	532	11
457x191x67	5	753	8	150	10	90	6	5.7	4.9	605	11
{839 kN}	4	602	8	150	10	90	6	5.7	4.9	509	11
457x152x82	5	753	8	150	10	90	8	5.7	4.9	645	11
{1040 kN}	4	602	8	150	10	90	8	5.7	4.9	542	11
457x152x74	5	753	8	150	10	90	8	5.7	4.9	638	11
{938 kN}	4	602	8	150	10	90	8	5.7	4.9	536	11
457x152x67	5	753	8	150	10	90	8	5.7	4.9	633	11
{899 kN}	4	602	8	150	10	90	8	5.7	4.9	532	11
457x152x60	5	753	8	150	10	90	6	5.7	4.9	602	11
{806 kN}	4	602	8	150	10	90	6	5.7	4.9	507	11
457x152x52	5	747	4	150	10	90	6	5.6	4.9	599	11
{747 kN}	4	602	8	150	10	90	6	5.7	4.9	504	11

For guidance on the use of tables see Explanatory notes in Table G.8

Full depth end plates

BEAM: S355
END PLATES: S275
BOLTS: M20, 8.8

Table G.12 *Continued*

FULL DEPTH END PLATES, ORDINARY or FLOWDRILL BOLTS
200 x 12 or 150 x 10 mm End Plate

Beam size {$V_{Rd,beam}$}	Bolt rows	Un-notched		Fitting (End Plate)			Web weld	Min support thickness		Tying	
	n_1	Shear Resistance V_{Rd} kN	Critical check	Width b_p mm	Thck t_p mm	Gauge p_3 mm	Leg s mm	S275 mm	S355 mm	Resistance $N_{Rd,u}$ kN	Critical check
406x178x85+ {966 kN}	4	602	8	150	10	90	8	5.7	4.9	545	11
406x178x74 {858 kN}	4	602	8	150	10	90	8	5.7	4.9	535	11
406x178x67 {790 kN}	4	602	8	150	10	90	6	5.7	4.9	511	11
406x178x60 {709 kN}	4	602	8	150	10	90	6	5.7	4.9	506	11
406x178x54 {683 kN}	4	602	8	150	10	90	6	5.7	4.9	505	11
406x140x53+ {709 kN}	4	602	8	150	10	90	6	5.7	4.9	506	11
406x140x46 {611 kN}	4	602	8	150	10	90	6	5.7	4.9	501	11
406x140x39 {566 kN}	4	566	4	150	10	90	6	5.3	4.6	499	11
356x171x67 {733 kN}	3	452	8	150	10	90	8	5.7	4.9	436	11
356x171x57 {646 kN}	3	452	8	150	10	90	6	5.7	4.9	417	11
356x171x51 {587 kN}	3	452	8	150	10	90	6	5.7	4.9	414	11
356x171x45 {549 kN}	3	452	8	150	10	90	6	5.7	4.9	412	11
356x127x39 {527 kN}	3	452	8	150	10	90	6	5.7	4.9	411	11
356x127x33 {472 kN}	3	452	8	150	10	90	6	5.7	4.9	408	11

For guidance on the use of tables see Explanatory notes in Table G.8

BEAM: S355
END PLATES: S275
BOLTS: M20, 8.8

Table G.12 *Continued*

FULL DEPTH END PLATES, ORDINARY or FLOWDRILL BOLTS 200 x 12 or 150 x 10 mm End Plate											
	Bolt rows	Un-notched		Fitting (End Plate)			Web weld	Min support thickness		Tying	
Beam size {$V_{Rd,beam}$}	n_1	Shear Resistance V_{Rd} kN	Critical check	Width b_p mm	Thck t_p mm	Gauge p_3 mm	Leg s mm	S275 mm	S355 mm	Resistance $N_{Rd,u}$ kN	Critical check
305x165x54 {545 kN}	3	452	8	150	10	90	6	5.7	4.9	423	11
305x165x46 {461 kN}	3	452	8	150	10	90	6	5.7	4.9	418	11
305x165x40 {411 kN}	3	411	4	150	10	90	6	5.2	4.5	416	11
305x127x48 {612 kN}	3	452	8	150	10	90	8	5.7	4.9	442	11
305x127x42 {542 kN} 0	3	452	8	150	10	90	6	5.7	4.9	423	11
305x127x37 {481 kN}	3	452	8	150	10	90	6	5.7	4.9	420	11
305x102x33 {452 kN}	3	452	8	150	10	90	6	5.7	4.9	416	11
305x102x28 {407 kN}	3	407	4	150	10	90	6	5.1	4.4	414	11
305x102x25 {386 kN}	3	386	4	150	10	90	6	4.8	4.2	413	11
254x146x43 {415 kN}	2	301	8	150	10	90	6	5.7	4.9	327	11
254x146x37 {361 kN}	2	301	8	150	10	90	6	5.7	4.9	325	11
254x146x31 {336 kN}	2	301	8	150	10	90	6	5.7	4.9	324	11
254x102x28 {365 kN}	2	301	8	150	10	90	6	5.7	4.9	324	11
254x102x25 {341 kN}	2	301	8	150	10	90	6	5.7	4.9	323	11
254x102x22 {320 kN}	2	301	8	150	10	90	6	5.7	4.9	322	11

For guidance on the use of tables see Explanatory notes in Table G.8

BEAM: S275
END PLATES: S275
HOLLO-BOLTS: M20, 8.8

Table G.13

FULL DEPTH END PLATES, HOLLO-BOLTS
200 x 12 or 180 x 10 mm End Plate

Beam size {$V_{Rd,beam}$}	Bolt rows n_1	Un-notched: Shear Resistance V_{Rd} kN	Un-notched: Critical check	Fitting (End Plate): Width b_p mm	Fitting (End Plate): Thck t_p mm	Fitting (End Plate): Gauge p_3 mm	Web weld: Leg s mm	Min support thickness: S275 mm	Min support thickness: S355 mm	Tying: Resistance $N_{Rd,u}$ kN	Tying: Critical check
1016x305x487+	11	1680	8	200	12	110	18	4.9	4.2	1850	11
{4930 kN}	10	1530	8	200	12	110	18	4.9	4.2	1690	11
	9	1380	8	200	12	110	18	4.9	4.2	1520	11
	8	1230	8	200	12	110	18	4.9	4.2	1360	11
1016x305x437+	11	1680	8	200	12	110	18	4.9	4.2	1810	11
{4420 kN}	10	1530	8	200	12	110	18	4.9	4.2	1650	11
	9	1380	8	200	12	110	18	4.9	4.2	1490	11
	8	1230	8	200	12	110	18	4.9	4.2	1330	11
1016x305x393+	11	1680	8	200	12	110	15	4.9	4.2	1780	11
{3990 kN}	10	1530	8	200	12	110	15	4.9	4.2	1620	11
	9	1380	8	200	12	110	15	4.9	4.2	1470	11
	8	1230	8	200	12	110	15	4.9	4.2	1310	11
1016x305x349+	11	1680	8	200	12	110	12	4.9	4.2	1750	11
{3610 kN}	10	1530	8	200	12	110	12	4.9	4.2	1600	11
	9	1380	8	200	12	110	12	4.9	4.2	1450	11
	8	1230	8	200	12	110	12	4.9	4.2	1290	11
1016x305x314+	11	1680	8	200	12	110	12	4.9	4.2	1750	11
{3260 kN}	10	1530	8	200	12	110	12	4.9	4.2	1590	11
	9	1380	8	200	12	110	12	4.9	4.2	1440	11
	8	1230	8	200	12	110	12	4.9	4.2	1290	11
1016x305x272+	11	1680	8	200	12	110	10	4.9	4.2	1690	11
{2830 kN}	10	1530	8	200	12	110	10	4.9	4.2	1540	11
	9	1380	8	200	12	110	10	4.9	4.2	1400	11
	8	1230	8	200	12	110	10	4.9	4.2	1250	11
1016x305x249+	11	1680	8	200	12	110	10	4.9	4.2	1690	11
{2770 kN}	10	1530	8	200	12	110	10	4.9	4.2	1540	11
	9	1380	8	200	12	110	10	4.9	4.2	1400	11
	8	1230	8	200	12	110	10	4.9	4.2	1250	11
1016x305x222+	11	1680	8	200	12	110	10	4.9	4.2	1690	11
{2640 kN}	10	1530	8	200	12	110	10	4.9	4.2	1530	11
	9	1380	8	200	12	110	10	4.9	4.2	1390	11
	8	1230	8	200	12	110	10	4.9	4.2	1250	11

For guidance on the use of tables see Explanatory notes in Table G.8

BEAM: S275
END PLATES: S275
HOLLO-BOLTS: M20, 8.8

Table G.13 *Continued*

FULL DEPTH END PLATES, HOLLO-BOLTS
200 x 12 or 180 x 10 mm End Plate

Beam size	Bolt rows	Un-notched		Fitting (End Plate)			Web weld	Min support thickness		Tying	
{$V_{Rd,beam}$}	n_1	Shear Resistance V_{Rd} kN	Critical check	Width b_p mm	Thck t_p mm	Gauge p_3 mm	Leg s mm	S275 mm	S355 mm	Resistance $N_{Rd,u}$ kN	Critical check
914x419x388	10	1530	8	200	12	110	15	4.9	4.2	1610	11
{3240 kN}	9	1380	8	200	12	110	15	4.9	4.2	1450	11
	8	1230	8	200	12	110	15	4.9	4.2	1300	11
914x419x343	10	1530	8	200	12	110	12	4.9	4.2	1590	11
{2920 kN}	9	1380	8	200	12	110	12	4.9	4.2	1430	11
	8	1230	8	200	12	110	12	4.9	4.2	1290	11
914x305x289	10	1530	8	200	12	110	12	4.9	4.2	1590	11
{2900 kN}	9	1380	8	200	12	110	12	4.9	4.2	1430	11
	8	1230	8	200	12	110	12	4.9	4.2	1290	11
914x305x253	10	1530	8	200	12	110	10	4.9	4.2	1550	11
{2570 kN}	9	1380	8	200	12	110	10	4.9	4.2	1390	11
	8	1230	8	200	12	110	10	4.9	4.2	1260	11
914x305x224	10	1530	8	200	12	110	10	4.9	4.2	1530	11
{2350 kN}	9	1380	8	200	12	110	10	4.9	4.2	1380	11
	8	1230	8	200	12	110	10	4.9	4.2	1250	11
914x305x201	10	1530	8	200	12	110	10	4.9	4.2	1530	11
{2210 kN}	9	1380	8	200	12	110	10	4.9	4.2	1380	11
	8	1230	8	200	12	110	10	4.9	4.2	1240	11
838x292x226	9	1380	8	200	12	110	10	4.9	4.2	1390	11
{2220 kN}	8	1230	8	200	12	110	10	4.9	4.2	1240	11
838x292x194	9	1380	8	200	12	110	10	4.9	4.2	1380	11
{2000 kN}	8	1230	8	200	12	110	10	4.9	4.2	1230	11
838x292x176	9	1380	8	200	12	110	8	4.9	4.2	1350	11
{1890 kN}	8	1230	8	200	12	110	8	4.9	4.2	1200	11

For guidance on the use of tables see Explanatory notes in Table G.8

BEAM: S275
END PLATES: S275
Table G.13 *Continued* **HOLLO-BOLTS: M20, 8.8**

FULL DEPTH END PLATES, HOLLO-BOLTS
200 x 12 or 180 x 10 mm End Plate

Beam size {$V_{Rd,beam}$}	Bolt rows n_1	Un-notched: Shear Resistance V_{Rd} kN	Un-notched: Critical check	Fitting (End Plate): Width b_p mm	Fitting (End Plate): Thck t_p mm	Fitting (End Plate): Gauge p_3 mm	Web weld: Leg s mm	Min support thickness: S275 mm	Min support thickness: S355 mm	Tying: Resistance $N_{Rd,u}$ kN	Tying: Critical check
762x267x197	8	1230	8	200	12	110	10	4.9	4.2	1230	11
{1950 kN}	7	1070	8	200	12	110	10	4.9	4.2	1080	11
762x267x173	8	1230	8	200	12	110	10	4.9	4.2	1230	11
{1760 kN}	7	1070	8	200	12	110	10	4.9	4.2	1080	11
762x267x147	8	1230	8	200	12	110	8	4.9	4.2	1200	11
{1560 kN}	7	1070	8	200	12	110	8	4.9	4.2	1050	11
762x267x134	8	1230	8	200	12	110	8	4.9	4.2	1190	11
{1520 kN}	7	1070	8	200	12	110	8	4.9	4.2	1050	11
686x254x170	7	1070	8	200	12	110	10	4.9	4.2	1080	11
{1630 kN}	6	919	8	200	12	110	10	4.9	4.2	931	11
686x254x152	7	1070	8	200	12	110	8	4.9	4.2	1050	11
{1470 kN}	6	919	8	200	12	110	8	4.9	4.2	910	11
686x254x140	7	1070	8	200	12	110	8	4.9	4.2	1050	11
{1370 kN}	6	919	8	200	12	110	8	4.9	4.2	906	11
686x254x125	7	1070	8	200	12	110	8	4.9	4.2	1050	11
{1280 kN}	6	919	8	200	12	110	8	4.9	4.2	902	11
610x305x238 {1890 kN}	6	919	8	200	12	110	12	4.9	4.2	968	11
610x305x179 {1440 kN}	6	919	8	200	12	110	8	4.9	4.2	915	11
610x305x149 {1200 kN}	6	919	8	200	12	110	8	4.9	4.2	905	11

For guidance on the use of tables see Explanatory notes in Table G.8

BEAM: S275
END PLATES: S275
HOLLO-BOLTS: M20, 8.8

Table G.13 *Continued*

FULL DEPTH END PLATES, HOLLO-BOLTS
200 x 12 or 180 x 10 mm End Plate

Beam size $\{V_{Rd,beam}\}$	Bolt rows n_1	Un-notched: Shear Resistance V_{Rd} kN	Un-notched: Critical check	Fitting (End Plate): Width b_p mm	Fitting (End Plate): Thck t_p mm	Fitting (End Plate): Gauge p_3 mm	Web weld: Leg s mm	Min support thickness: S275 mm	Min support thickness: S355 mm	Tying: Resistance $N_{Rd,u}$ kN	Tying: Critical check
610x229x140 {1300 kN}	6	919	8	200	12	110	8	4.9	4.2	911	11
610x229x125 {1170 kN}	6	919	8	200	12	110	8	4.9	4.2	905	11
610x229x113 {1090 kN}	6	919	8	200	12	110	8	4.9	4.2	901	11
610x229x101 {1060 kN}	6	919	8	200	12	110	6	4.9	4.2	885	11
610x178x100+ {1110 kN}	6	919	8	200	12	110	8	4.9	4.2	902	11
610x178x92+ {1090 kN}	6	919	8	200	12	110	8	4.9	4.2	900	11
610x178x82+ {1000 kN}	6	919	8	200	12	110	6	4.9	4.2	882	11
533x312x272+ {1910 kN}	6	919	8	200	12	110	12	4.9	4.2	979	11
	5	766	8	200	12	110	12	4.9	4.2	820	11
533x312x219+ {1630 kN}	5	766	8	200	12	110	12	4.9	4.2	813	11
533x312x182+ {1340 kN}	5	766	8	200	12	110	10	4.9	4.2	787	11
533x312x150+ {1120 kN}	5	766	8	200	12	110	8	4.9	4.2	765	11

For guidance on the use of tables see Explanatory notes in Table G.8

BEAM: S275
END PLATES: S275
HOLLO-BOLTS: M20, 8.8

Table G.13 *Continued*

FULL DEPTH END PLATES, HOLLO-BOLTS
200 x 12 or 180 x 10 mm End Plate

Beam size {$V_{Rd,beam}$}	Bolt rows n_1	Un-notched: Shear Resistance V_{Rd} kN	Un-notched: Critical check	Fitting (End Plate): Width b_p mm	Fitting (End Plate): Thck t_p mm	Fitting (End Plate): Gauge p_3 mm	Web weld: Leg s mm	Min support thickness: S275 mm	Min support thickness: S355 mm	Tying: Resistance $N_{Rd,u}$ kN	Tying: Critical check
533x210x138+ {1290 kN}	5	766	8	200	12	110	10	4.9	4.2	785	11
533x210x122 {1110 kN}	5	766	8	200	12	110	8	4.9	4.2	765	11
533x210x109 {1020 kN}	5	766	8	200	12	110	8	4.9	4.2	761	11
533x210x101 {952 kN}	5	766	8	200	12	110	8	4.9	4.2	758	11
533x210x92 {909 kN}	5	766	8	200	12	110	6	4.9	4.2	744	11
533x210x82 {865 kN}	5	766	8	200	12	110	6	4.9	4.2	742	11
533x165x85+ {902 kN}	5	766	8	200	12	110	6	4.9	4.2	745	11
533x165x74+ {871 kN}	5	766	8	200	12	110	6	4.9	4.2	743	11
533x165x66+ {793 kN}	5	766	8	200	12	110	6	4.9	4.2	740	11

For guidance on the use of tables see Explanatory notes in Table G.8

BEAM: S275
END PLATES: S275
HOLLO-BOLTS: M20, 8.8

Table G.13 *Continued*

FULL DEPTH END PLATES, HOLLO-BOLTS
200 x 12 or 180 x 10 mm End Plate

Beam size $\{V_{Rd,beam}\}$	Bolt rows n_1	Un-notched: Shear Resistance V_{Rd} kN	Un-notched: Critical check	Fitting (End Plate): Width b_p mm	Fitting (End Plate): Thck t_p mm	Fitting (End Plate): Gauge p_3 mm	Web weld: Leg s mm	Min support thickness: S275 mm	Min support thickness: S355 mm	Tying: Resistance $N_{Rd,u}$ kN	Tying: Critical check
457x191x161+ {1390 kN}	5	638	8	180	10	90	12	4.0	3.5	792	11
	4	510	8	180	10	90	12	4.0	3.5	638	11
457x191x133+ {1160 kN}	4	510	8	180	10	90	10	4.0	3.5	629	11
457x191x106+ {947 kN}	4	510	8	180	10	90	8	4.0	3.5	622	11
457x191x98 {852 kN}	4	510	8	180	10	90	8	4.0	3.5	620	11
457x191x89 {789 kN}	4	510	8	180	10	90	6	4.0	3.5	589	11
457x191x82 {756 kN}	4	510	8	180	10	90	6	4.0	3.5	584	11
457x191x74 {693 kN}	4	510	8	180	10	90	6	4.0	3.5	577	11
457x191x67 {650 kN}	4	510	8	180	10	90	6	4.0	3.5	574	11
457x152x82 {798 kN}	4	510	8	180	10	90	6	4.0	3.5	589	11
457x152x74 {721 kN}	4	510	8	180	10	90	6	4.0	3.5	582	11
457x152x67 {697 kN}	4	510	8	180	10	90	6	4.0	3.5	577	11
457x152x60 {624 kN}	4	510	8	180	10	90	6	4.0	3.5	571	11
457x152x52 {578 kN}	4	510	8	180	10	90	6	4.0	3.5	568	11

For guidance on the use of tables see Explanatory notes in Table G.8

BEAM: S275
END PLATES: S275
HOLLO-BOLTS: M20, 8.8

Table G.13 *Continued*

FULL DEPTH END PLATES, HOLLO-BOLTS
200 x 12 or 180 x 10 mm End Plate

Beam size $\{V_{Rd,beam}\}$	Bolt rows n_1	Un-notched: Shear Resistance V_{Rd} kN	Un-notched: Critical check	Fitting (End Plate): Width b_p mm	Fitting (End Plate): Thck t_p mm	Fitting (End Plate): Gauge p_3 mm	Web weld: Leg s mm	Min support thickness: S275 mm	Min support thickness: S355 mm	Tying: Resistance $N_{Rd,u}$ kN	Tying: Critical check
406x178x85+ {742 kN}	4	510	8	180	10	90	8	4.0	3.5	621	11
406x178x74 {664 kN}	4	510	8	180	10	90	6	4.0	3.5	587	11
406x178x67 {612 kN}	4	510	8	180	10	90	6	4.0	3.5	583	11
406x178x60 {549 kN}	4	510	8	180	10	90	6	4.0	3.5	577	11
406x178x54 {529 kN}	4	510	8	180	10	90	6	4.0	3.5	577	11
406x140x53+ {549 kN}	4	510	8	180	10	90	6	4.0	3.5	577	11
406x140x46 {473 kN}	4	473	4	180	10	90	6	3.7	3.3	571	11
406x140x39 {438 kN}	4	438	4	180	10	90	6	3.5	3.0	570	11
356x171x67 {568 kN}	3	383	8	180	10	90	6	4.0	3.5	469	11
356x171x57 {501 kN}	3	383	8	180	10	90	6	4.0	3.5	468	11
356x171x51 {455 kN}	3	383	8	180	10	90	6	4.0	3.5	466	11
356x171x45 {425 kN}	3	383	8	180	10	90	6	4.0	3.5	465	11
356x127x39 {408 kN}	3	383	8	180	10	90	6	4.0	3.5	464	11
356x127x33 {366 kN}	3	366	4	180	10	90	6	3.9	3.4	461	11

For guidance on the use of tables see Explanatory notes in Table G.8

BEAM: S275
END PLATES: S275
HOLLO-BOLTS: M20, 8.8

Table G.13 *Continued*

FULL DEPTH END PLATES, HOLLO-BOLTS
200 x 12 or 180 x 10 mm End Plate

Beam size $\{V_{Rd,beam}\}$	Bolt rows n_1	Un-notched: Shear Resistance V_{Rd} kN	Un-notched: Critical check	Fitting (End Plate): Width b_p mm	Fitting (End Plate): Thck t_p mm	Fitting (End Plate): Gauge p_3 mm	Web weld: Leg s mm	Min support thickness: S275 mm	Min support thickness: S355 mm	Tying: Resistance $N_{Rd,u}$ kN	Tying: Critical check
305x165x54 {422 kN}	3	383	8	180	10	90	6	4.0	3.5	470	11
305x165x46 {357 kN}	3	357	4	180	10	90	6	3.8	3.3	468	11
305x165x40 {319 kN}	3	319	4	180	10	90	6	3.4	2.9	466	11
305x127x48 {474 kN}	3	383	8	180	10	90	6	4.0	3.5	472	11
305x127x42 {420 kN}	3	383	8	180	10	90	6	4.0	3.5	471	11
305x127x37 {372 kN}	3	372	4	180	10	90	6	3.9	3.4	469	11
305x102x33 {350 kN}	3	350	4	180	10	90	6	3.7	3.2	467	11
305x102x28 {315 kN}	3	315	4	180	10	90	6	3.3	2.9	466	11
305x102x25 {299 kN}	3	299	4	180	10	90	6	3.2	2.8	465	11
254x146x43 {321 kN}	2	255	8	180	10	90	6	4.0	3.5	321	11
254x146x37 {280 kN}	2	255	8	180	10	90	6	4.0	3.5	320	11
254x146x31 {260 kN}	2	255	8	180	10	90	6	4.0	3.5	319	11
254x102x28 {283 kN}	2	255	8	180	10	90	6	4.0	3.5	319	11
254x102x25 {265 kN}	2	255	8	180	10	90	6	4.0	3.5	319	11
254x102x22 {248 kN}	2	248	4	180	10	90	6	3.9	3.4	319	11

For guidance on the use of tables see Explanatory notes in Table G.8

BEAM: S355
END PLATES: S275
Table G.14
HOLLO-BOLTS: M20, 8.8

FULL DEPTH END PLATES, HOLLO-BOLTS
200 x 12 or 180 x 10 mm End Plate

Beam size {$V_{Rd,beam}$}	Bolt rows n_1	Un-notched: Shear Resistance V_{Rd} kN	Un-notched: Critical check	Fitting (End Plate): Width b_p mm	Fitting (End Plate): Thck t_p mm	Fitting (End Plate): Gauge p_3 mm	Web weld: Leg s mm	Min support thickness: S275 mm	Min support thickness: S355 mm	Tying: Resistance $N_{Rd,u}$ kN	Tying: Critical check
1016x305x487+	11	1680	8	200	12	110	22	4.9	4.2	1910	11
{6480 kN}	10	1530	8	200	12	110	22	4.9	4.2	1740	11
	9	1380	8	200	12	110	22	4.9	4.2	1570	11
	8	1230	8	200	12	110	22	4.9	4.2	1400	11
1016x305x437+	11	1680	8	200	12	110	20	4.9	4.2	1850	11
{5810 kN}	10	1530	8	200	12	110	20	4.9	4.2	1690	11
	9	1380	8	200	12	110	20	4.9	4.2	1520	11
	8	1230	8	200	12	110	20	4.9	4.2	1360	11
1016x305x393+	11	1680	8	200	12	110	18	4.9	4.2	1820	11
{5240 kN}	10	1530	8	200	12	110	18	4.9	4.2	1660	11
	9	1380	8	200	12	110	18	4.9	4.2	1500	11
	8	1230	8	200	12	110	18	4.9	4.2	1340	11
1016x305x349+	11	1680	8	200	12	110	15	4.9	4.2	1780	11
{4700 kN}	10	1530	8	200	12	110	15	4.9	4.2	1620	11
	9	1380	8	200	12	110	15	4.9	4.2	1470	11
	8	1230	8	200	12	110	15	4.9	4.2	1310	11
1016x305x314+	11	1680	8	200	12	110	15	4.9	4.2	1760	11
{4240 kN}	10	1530	8	200	12	110	15	4.9	4.2	1600	11
	9	1380	8	200	12	110	15	4.9	4.2	1450	11
	8	1230	8	200	12	110	15	4.9	4.2	1300	11
1016x305x272+	11	1680	8	200	12	110	12	4.9	4.2	1720	11
{3680 kN}	10	1530	8	200	12	110	12	4.9	4.2	1570	11
	9	1380	8	200	12	110	12	4.9	4.2	1420	11
	8	1230	8	200	12	110	12	4.9	4.2	1270	11
1016x305x249+	11	1680	8	200	12	110	12	4.9	4.2	1720	11
{3600 kN}	10	1530	8	200	12	110	12	4.9	4.2	1570	11
	9	1380	8	200	12	110	12	4.9	4.2	1420	11
	8	1230	8	200	12	110	12	4.9	4.2	1270	11
1016x305x222+	11	1680	8	200	12	110	12	4.9	4.2	1720	11
{3430 kN}	10	1530	8	200	12	110	12	4.9	4.2	1560	11
	9	1380	8	200	12	110	12	4.9	4.2	1420	11
	8	1230	8	200	12	110	12	4.9	4.2	1270	11

For guidance on the use of tables see Explanatory notes in Table G.8

BEAM: S355
END PLATES: S275
HOLLO-BOLTS: M20, 8.8

Table G.14 *Continued*

FULL DEPTH END PLATES, HOLLO-BOLTS
200 x 12 or 180 x 10 mm End Plate

Beam size $\{V_{Rd,beam}\}$	Bolt rows n_1	Un-notched: Shear Resistance V_{Rd} kN	Un-notched: Critical check	Fitting (End Plate): Width b_p mm	Fitting (End Plate): Thck t_p mm	Fitting (End Plate): Gauge p_3 mm	Web weld: Leg s mm	Min support thickness: S275 mm	Min support thickness: S355 mm	Tying: Resistance $N_{Rd,u}$ kN	Tying: Critical check
914x419x388	10	1530	8	200	12	110	15	4.9	4.2	1620	11
{4220 kN}	9	1380	8	200	12	110	15	4.9	4.2	1460	11
	8	1230	8	200	12	110	15	4.9	4.2	1310	11
914x419x343	10	1530	8	200	12	110	15	4.9	4.2	1600	11
{3800 kN}	9	1380	8	200	12	110	15	4.9	4.2	1440	11
	8	1230	8	200	12	110	15	4.9	4.2	1300	11
914x305x289	10	1530	8	200	12	110	15	4.9	4.2	1600	11
{3780 kN}	9	1380	8	200	12	110	15	4.9	4.2	1450	11
	8	1230	8	200	12	110	15	4.9	4.2	1300	11
914x305x253	10	1530	8	200	12	110	12	4.9	4.2	1570	11
{3350 kN}	9	1380	8	200	12	110	12	4.9	4.2	1420	11
	8	1230	8	200	12	110	12	4.9	4.2	1280	11
914x305x224	10	1530	8	200	12	110	12	4.9	4.2	1560	11
{3060 kN}	9	1380	8	200	12	110	12	4.9	4.2	1410	11
	8	1230	8	200	12	110	12	4.9	4.2	1270	11
914x305x201	10	1530	8	200	12	110	12	4.9	4.2	1550	11
{2870 kN}	9	1380	8	200	12	110	12	4.9	4.2	1400	11
	8	1230	8	200	12	110	12	4.9	4.2	1260	11
838x292x226	9	1380	8	200	12	110	12	4.9	4.2	1410	11
{2900 kN}	8	1230	8	200	12	110	12	4.9	4.2	1260	11
838x292x194	9	1380	8	200	12	110	10	4.9	4.2	1380	11
{2610 kN}	8	1230	8	200	12	110	10	4.9	4.2	1230	11
838x292x176	9	1380	8	200	12	110	10	4.9	4.2	1370	11
{2460 kN}	8	1230	8	200	12	110	10	4.9	4.2	1220	11

For guidance on the use of tables see Explanatory notes in Table G.8

BEAM: S355
END PLATES: S275
HOLLO-BOLTS: M20, 8.8

Table G.14 *Continued*

FULL DEPTH END PLATES, HOLLO-BOLTS
200 x 12 or 180 x 10 mm End Plate

Beam size $\{V_{Rd,beam}\}$	Bolt rows n_1	Un-notched: Shear Resistance V_{Rd} kN	Un-notched: Critical check	Fitting (End Plate): Width b_p mm	Fitting (End Plate): Thck t_p mm	Fitting (End Plate): Gauge p_3 mm	Web weld: Leg s mm	Min support thickness: S275 mm	Min support thickness: S355 mm	Tying: Resistance $N_{Rd,u}$ kN	Tying: Critical check
762x267x197	8	1230	8	200	12	110	12	4.9	4.2	1260	11
{2530 kN}	7	1070	8	200	12	110	12	4.9	4.2	1100	11
762x267x173	8	1230	8	200	12	110	10	4.9	4.2	1230	11
{2290 kN}	7	1070	8	200	12	110	10	4.9	4.2	1080	11
762x267x147	8	1230	8	200	12	110	10	4.9	4.2	1220	11
{2040 kN}	7	1070	8	200	12	110	10	4.9	4.2	1070	11
762x267x134	8	1230	8	200	12	110	10	4.9	4.2	1210	11
{1970 kN}	7	1070	8	200	12	110	10	4.9	4.2	1060	11
686x254x170	7	1070	8	200	12	110	10	4.9	4.2	1080	11
{2120 kN}	6	919	8	200	12	110	10	4.9	4.2	931	11
686x254x152	7	1070	8	200	12	110	10	4.9	4.2	1070	11
{1920 kN}	6	919	8	200	12	110	10	4.9	4.2	924	11
686x254x140	7	1070	8	200	12	110	10	4.9	4.2	1070	11
{1790 kN}	6	919	8	200	12	110	10	4.9	4.2	920	11
686x254x125	7	1070	8	200	12	110	8	4.9	4.2	1050	11
{1670 kN}	6	919	8	200	12	110	8	4.9	4.2	902	11
610x305x238	6	919	8	200	12	110	15	4.9	4.2	979	11
{2460 kN}											
610x305x179	6	919	8	200	12	110	10	4.9	4.2	930	11
{1880 kN}											
610x305x149	6	919	8	200	12	110	10	4.9	4.2	919	11
{1570 kN}											

For guidance on the use of tables see Explanatory notes in Table G.8

BEAM: S355
END PLATES: S275
HOLLO-BOLTS: M20, 8.8

Table G.14 *Continued*

FULL DEPTH END PLATES, HOLLO-BOLTS
200 x 12 or 180 x 10 mm End Plate

	Bolt rows	Un-notched		Fitting (End Plate)			Web weld	Min support thickness		Tying	
Beam size $\{V_{Rd,beam}\}$	n_1	Shear Resistance V_{Rd} kN	Critical check	Width b_p mm	Thck t_p mm	Gauge p_3 mm	Leg s mm	S275 mm	S355 mm	Resistance $N_{Rd,u}$ kN	Critical check
610x229x140 {1690 kN}	6	919	8	200	12	110	10	4.9	4.2	925	11
610x229x125 {1520 kN}	6	919	8	200	12	110	10	4.9	4.2	919	11
610x229x113 {1420 kN}	6	919	8	200	12	110	8	4.9	4.2	901	11
610x229x101 {1370 kN}	6	919	8	200	12	110	8	4.9	4.2	899	11
610x178x100+ {1450 kN}	6	919	8	200	12	110	8	4.9	4.2	902	11
610x178x92+ {1410 kN}	6	919	8	200	12	110	8	4.9	4.2	900	11
610x178x82+ {1290 kN}	6	919	8	200	12	110	8	4.9	4.2	896	11
533x312x272+	6	919	8	200	12	110	15	4.9	4.2	990	11
{2480 kN}	5	766	8	200	12	110	15	4.9	4.2	829	11
533x312x219+ {2120 kN}	5	766	8	200	12	110	15	4.9	4.2	822	11
533x312x182+ {1740 kN}	5	766	8	200	12	110	12	4.9	4.2	800	11
533x312x150+ {1460 kN}	5	766	8	200	12	110	10	4.9	4.2	777	11

For guidance on the use of tables see Explanatory notes in Table G.8

BEAM: S355
END PLATES: S275
HOLLO-BOLTS: M20, 8.8

Table G.14 *Continued*

FULL DEPTH END PLATES, HOLLO-BOLTS
200 x 12 or 180 x 10 mm End Plate

Beam size $\{V_{Rd,beam}\}$	Bolt rows n_1	Un-notched: Shear Resistance V_{Rd} kN	Un-notched: Critical check	Fitting (End Plate): Width b_p mm	Fitting (End Plate): Thck t_p mm	Fitting (End Plate): Gauge p_3 mm	Web weld: Leg s mm	Min support thickness: S275 mm	Min support thickness: S355 mm	Tying: Resistance $N_{Rd,u}$ kN	Tying: Critical check
533x210x138+ {1680 kN}	5	766	8	200	12	110	10	4.9	4.2	785	11
533x210x122 {1450 kN}	5	766	8	200	12	110	10	4.9	4.2	777	11
533x210x109 {1330 kN}	5	766	8	200	12	110	8	4.9	4.2	761	11
533x210x101 {1240 kN}	5	766	8	200	12	110	8	4.9	4.2	758	11
533x210x92 {1170 kN}	5	766	8	200	12	110	8	4.9	4.2	755	11
533x210x82 {1120 kN}	5	766	8	200	12	110	8	4.9	4.2	753	11
533x165x85+ {1170 kN}	5	766	8	200	12	110	8	4.9	4.2	756	11
533x165x74+ {1120 kN}	5	766	8	200	12	110	8	4.9	4.2	754	11
533x165x66+ {1020 kN}	5	766	8	200	12	110	8	4.9	4.2	751	11

For guidance on the use of tables see Explanatory notes in Table G.8

BEAM: S355
END PLATES: S275
HOLLO-BOLTS: M20, 8.8

Table G.14 *Continued*

FULL DEPTH END PLATES, HOLLO-BOLTS
200 x 12 or 180 x 10 mm End Plate

Beam size $\{V_{Rd,beam}\}$	Bolt rows n_1	Un-notched Shear Resistance V_{Rd} kN	Un-notched Critical check	Fitting (End Plate) Width b_p mm	Fitting (End Plate) Thck t_p mm	Fitting (End Plate) Gauge p_3 mm	Web weld Leg s mm	Min support thickness S275 mm	Min support thickness S355 mm	Tying Resistance $N_{Rd,u}$ kN	Tying Critical check
457x191x161+	5	638	8	180	10	90	15	4.0	3.5	802	11
{1810 kN}	4	510	8	180	10	90	15	4.0	3.5	646	11
457x191x133+ {1510 kN}	4	510	8	180	10	90	12	4.0	3.5	633	11
457x191x106+ {1230 kN}	4	510	8	180	10	90	10	4.0	3.5	625	11
457x191x98 {1110 kN}	4	510	8	180	10	90	8	4.0	3.5	620	11
457x191x89 {1030 kN}	4	510	8	180	10	90	8	4.0	3.5	614	11
457x191x82 {976 kN}	4	510	8	180	10	90	8	4.0	3.5	609	11
457x191x74 {895 kN}	4	510	8	180	10	90	8	4.0	3.5	601	11
457x191x67 {839 kN}	4	510	8	180	10	90	6	4.0	3.5	574	11
457x152x82 {1040 kN}	4	510	8	180	10	90	8	4.0	3.5	614	11
457x152x74 {938 kN}	4	510	8	180	10	90	8	4.0	3.5	606	11
457x152x67 {899 kN}	4	510	8	180	10	90	8	4.0	3.5	601	11
457x152x60 {806 kN}	4	510	8	180	10	90	6	4.0	3.5	571	11
457x152x52 {747 kN}	4	510	8	180	10	90	6	4.0	3.5	568	11

For guidance on the use of tables see Explanatory notes in Table G.8

Full depth end plates

BEAM: S355
END PLATES: S275
HOLLO-BOLTS: M20, 8.8

Table G.14 *Continued*

FULL DEPTH END PLATES, HOLLO-BOLTS
200 x 12 or 180 x 10 mm End Plate

Beam size $\{V_{Rd,beam}\}$	Bolt rows n_1	Un-notched		Fitting (End Plate)			Web weld	Min support thickness		Tying	
		Shear Resistance V_{Rd} kN	Critical check	Width b_p mm	Thck t_p mm	Gauge p_3 mm	Leg s mm	S275 mm	S355 mm	Resistance $N_{Rd,u}$ kN	Critical check
406x178x85+ {966 kN}	4	510	8	180	10	90	8	4.0	3.5	621	11
406x178x74 {858 kN}	4	510	8	180	10	90	8	4.0	3.5	611	11
406x178x67 {790 kN}	4	510	8	180	10	90	6	4.0	3.5	583	11
406x178x60 {709 kN}	4	510	8	180	10	90	6	4.0	3.5	577	11
406x178x54 {683 kN}	4	510	8	180	10	90	6	4.0	3.5	577	11
406x140x53+ {709 kN}	4	510	8	180	10	90	6	4.0	3.5	577	11
406x140x46 {611 kN}	4	510	8	180	10	90	6	4.0	3.5	571	11
406x140x39 {566 kN}	4	510	8	180	10	90	6	4.0	3.5	570	11
356x171x67 {733 kN}	3	383	8	180	10	90	8	4.0	3.5	471	11
356x171x57 {646 kN}	3	383	8	180	10	90	6	4.0	3.5	468	11
356x171x51 {587 kN}	3	383	8	180	10	90	6	4.0	3.5	466	11
356x171x45 {549 kN}	3	383	8	180	10	90	6	4.0	3.5	465	11
356x127x39 {527 kN}	3	383	8	180	10	90	6	4.0	3.5	464	11
356x127x33 {472 kN}	3	383	8	180	10	90	6	4.0	3.5	461	11

For guidance on the use of tables see Explanatory notes in Table G.8

BEAM: S355
END PLATES: S275
HOLLO-BOLTS: M20, 8.8

Table G.14 *Continued*

FULL DEPTH END PLATES, HOLLO-BOLTS
200 x 12 or 180 x 10 mm End Plate

Beam size {$V_{Rd,beam}$}	Bolt rows n_1	Un-notched: Shear Resistance V_{Rd} kN	Un-notched: Critical check	Fitting (End Plate): Width b_p mm	Fitting (End Plate): Thck t_p mm	Fitting (End Plate): Gauge p_3 mm	Web weld: Leg s mm	Min support thickness: S275 mm	Min support thickness: S355 mm	Tying: Resistance $N_{Rd,u}$ kN	Tying: Critical check
305x165x54 {545 kN}	3	383	8	180	10	90	6	4.0	3.5	470	11
305x165x46 {461 kN}	3	383	8	180	10	90	6	4.0	3.5	468	11
305x165x40 {411 kN}	3	383	8	180	10	90	6	4.0	3.5	466	11
305x127x48 {612 kN}	3	383	8	180	10	90	8	4.0	3.5	474	11
305x127x42 {542 kN}	3	383	8	180	10	90	6	4.0	3.5	471	11
305x127x37 {481 kN}	3	383	8	180	10	90	6	4.0	3.5	469	11
305x102x33 {452 kN}	3	383	8	180	10	90	6	4.0	3.5	467	11
305x102x28 {407 kN}	3	383	8	180	10	90	6	4.0	3.5	466	11
305x102x25 {386 kN}	3	383	8	180	10	90	6	4.0	3.5	465	11
254x146x43 {415 kN}	2	255	8	180	10	90	6	4.0	3.5	321	11
254x146x37 {361 kN}	2	255	8	180	10	90	6	4.0	3.5	320	11
254x146x31 {336 kN}	2	255	8	180	10	90	6	4.0	3.5	319	11
254x102x28 {365 kN}	2	255	8	180	10	90	6	4.0	3.5	319	11
254x102x25 {341 kN}	2	255	8	180	10	90	6	4.0	3.5	319	11
254x102x22 {320 kN}	2	255	8	180	10	90	6	4.0	3.5	319	11

For guidance on the use of tables see Explanatory notes in Table G.8

Table G.15

Explanatory notes – FIN PLATES
Use of Resistance Tables

The check numbers noted below refer to those listed in Table G.17 and described in Section 5.5 Design procedures.

The resistance tables are based on the standard details given in Table G.16.

The value given in { } below each beam designation is the shear resistance of the beam, given by $\frac{A_v f_y}{\sqrt{3}\gamma_{M0}}$.

A + symbol adjacent to the beam designation indicates that the serial size is additional to those specified in BS 4-1.

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. $(n_1 - 1)p_1$ = 490 mm, which does not exceed the limit of 530 mm).

1 SHEAR RESISTANCE OF THE CONNECTION

This is the critical value of the design checks for the 'supported beam side' of the connection, i.e. the minimum resistance from Checks 2, 3, 4, and 8.

2 CRITICAL DESIGN CHECK

The check which gives the critical value of shear resistance. See Table G.17 for the description of the checks.

3 MINIMUM SUPPORT THICKNESS

This is the minimum thickness of supporting column or beam element that is needed to carry the given shear resistance (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.

If the applied shear force is less than the quoted resistance, the minimum support thickness reduces proportionally.

4 NOTCH LIMIT

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 resistance of the connection. The notch is measured from the end of the beam. The calculated resistances allow for the size of the gap between the beam and the support.

It is assumed that the beam is fully restrained against lateral torsional buckling, and the limiting notch 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 is the same as the length of the fin plate.

5 TYING RESISTANCE

This is the critical value of the design checks for the 'supported beam' side of the connection, i.e. the minimum resistance from Checks 11 and 12.

Separate checks will have to be carried out on the supporting member (see Checks 14, 15 and 16).

Table G.16

FIN PLATES
Standard details used in Resistance Tables

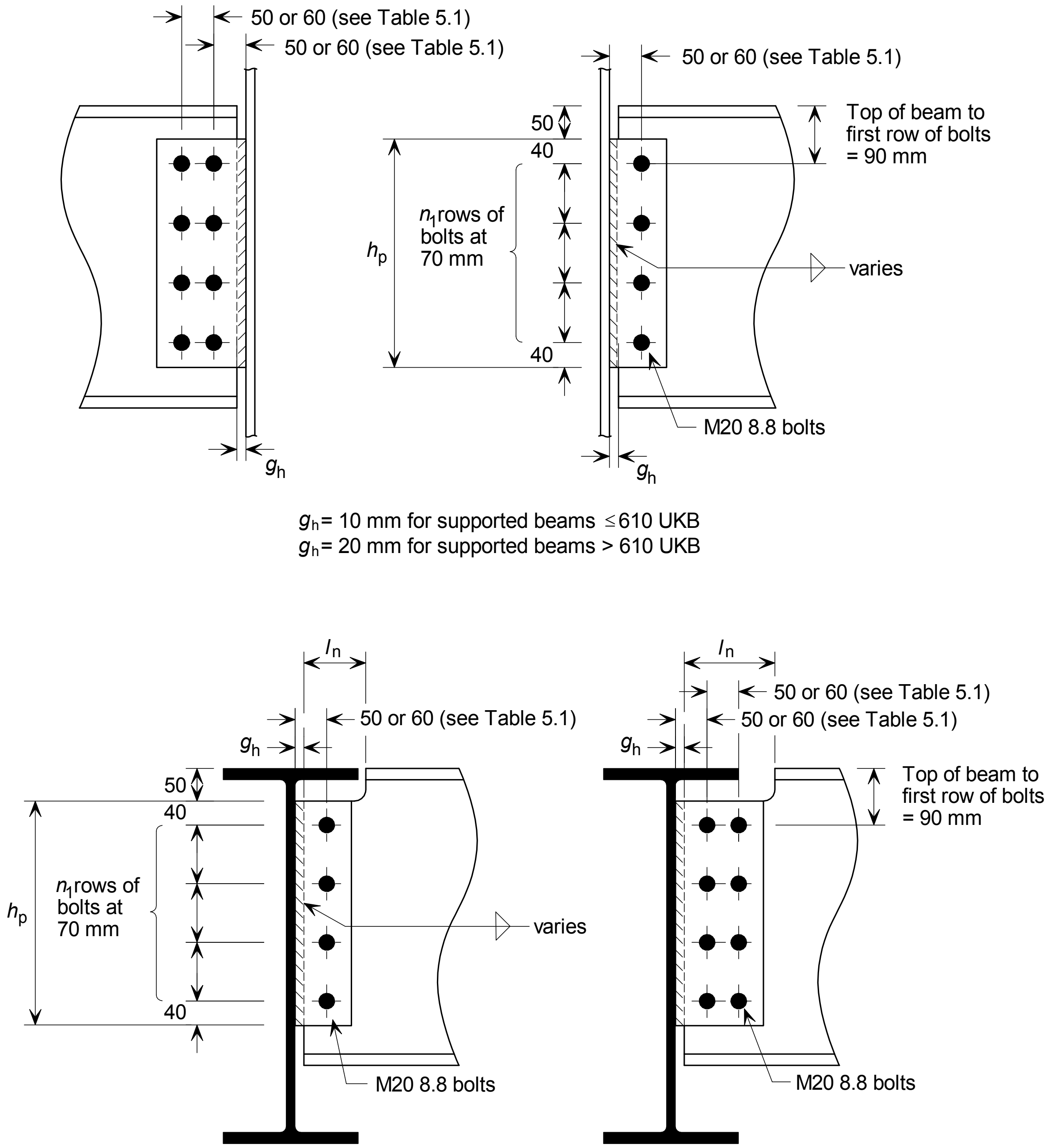


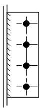
Table G.17

FIN PLATES Design Check List			
		Check Number	**Description**
SHEAR RESISTANCE	**Supported beam side**	2	Bolt group
		3	Shear resistance of fin plate
		4	Web in shear
	Supporting member side	8	Welds
TYING RESISTANCE		11	Plate and bolts
		12	Supported beam web

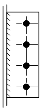
Note: This table only lists the critical checks.
For a full list of design checks and further information, see Section 5.5.

BEAM: S275
FIN PLATES: S275
BOLTS: M20, 8.8

Table G.18



FIN PLATES, ORDINARY BOLTS
Single line of bolts
120x10 or 100x10 mm Fin Plate

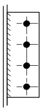


Beam size	Bolt rows	Un-notched		Single Notch		Double Notch		Fitting Fin Plate			Fillet weld	Min support thickness		Tying	
$\{V_{Rd,beam}\}$	n_1	Shear Resist V_{Rd} kN	Crit chck	Shear Resist V_{Rd} kN	Notch limit l_n mm	Shear Resist V_{Rd} kN	Notch limit l_n mm	Width b_p mm	Thck t_p mm	Length h_p mm	Leg s mm	S275 mm	S355 mm	Resist $N_{Rd,u}$ kN	Crit chck
1016x305x487+ {4930 kN}	8	653	2	653	2470	653	1340	120	10	570	8	4.0	3.1	855	11
1016x305x437+ {4420 kN}	8	653	2	653	2220	653	1230	120	10	570	8	4.0	3.1	855	11
1016x305x393+ {3990 kN}	8	653	2	653	1980	653	1090	120	10	570	8	4.0	3.1	855	11
1016x305x349+ {3610 kN}	8	653	2	653	1800	653	1010	120	10	570	8	4.0	3.1	855	11
1016x305x314+ {3260 kN}	8	653	2	653	1610	653	891	120	10	570	8	4.0	3.1	855	11
1016x305x272+ {2830 kN}	8	653	2	653	1370	653	748	120	10	570	8	4.0	3.1	855	11
1016x305x249+ {2770 kN}	8	653	2	653	1330	653	767	120	10	570	8	4.0	3.1	855	11
1016x305x222+ {2640 kN}	8	653	2	653	1230	653	726	120	10	570	8	4.0	3.1	855	11
914x419x388 {3240 kN}	8	653	2	653	1580	653	818	120	10	570	8	4.0	3.1	855	11
914x419x343 {2920 kN}	8	653	2	653	1440	653	761	120	10	570	8	4.0	3.1	855	11
914x305x289 {2900 kN}	8	653	2	653	1420	653	795	120	10	570	8	4.0	3.1	855	11
914x305x253 {2570 kN}	8	653	2	653	1290	653	763	120	10	570	8	4.0	3.1	855	11
914x305x224 {2350 kN}	8	653	2	653	1160	653	686	120	10	570	8	4.0	3.1	855	11
914x305x201 {2210 kN}	8	653	2	653	1060	653	638	120	10	570	8	4.0	3.1	855	11

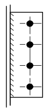
For guidance on the use of tables see Explanatory notes in Table G.15

BEAM: S275
FIN PLATES: S275
BOLTS: M20, 8.8

Table G.18 *Continued*



FIN PLATES, ORDINARY BOLTS
Single line of bolts
120x10 or 100x10 mm Fin Plate



	Bolt rows	Un-notched		Single Notch		Double Notch		Fitting Fin Plate			Fillet weld	Min support thickness		Tying	
Beam size {$V_{Rd,beam}$}	n_1	Shear Resist V_{Rd} kN	Crit chck	Shear Resist V_{Rd} kN	Notch limit l_n mm	Shear Resist V_{Rd} kN	Notch limit l_n mm	Width b_p mm	Thck t_p mm	Length h_p mm	Leg s mm	S275 mm	S355 mm	Resist $N_{Rd,u}$ kN	Crit chck
838x292x226	8	653	2	653	1030	653	594	120	10	570	8	4.0	3.1	855	11
{2220 kN}															
838x292x194	8	653	2	653	905	653	525	120	10	570	8	4.0	3.1	855	11
{2000 kN}															
838x292x176	8	653	2	653	839	653	491	120	10	570	8	4.0	3.1	855	11
{1890 kN}															
762x267x197	8	653	2	653	803	653	453	120	10	570	8	4.0	3.1	855	11
{1950 kN}	7	554	2	554	949	554	538	120	10	500	8	3.9	3.0	748	11
762x267x173	8	653	2	653	715	653	404	120	10	570	8	4.0	3.1	855	11
{1760 kN}	7	554	2	554	845	554	480	120	10	500	8	3.9	3.0	748	11
762x267x147	8	653	2	653	617	653	342	120	10	570	8	4.0	3.1	855	11
{1560 kN}	7	554	2	554	729	554	416	120	10	500	8	3.9	3.0	748	11
762x267x134	8	653	2	653	589	653	316	120	10	570	8	4.0	3.1	855	11
{1520 kN}	7	554	2	554	696	554	399	120	10	500	8	3.9	3.0	748	11
686x254x170	8	653	2	653	599	653	314	120	10	570	8	4.0	3.1	855	11
{1630 kN}	7	554	2	554	708	554	386	120	10	500	8	3.9	3.0	748	11
	6	455	2	455	865	455	475	120	10	430	8	3.7	2.9	641	11
686x254x152	8	653	2	653	535	653	259	120	10	570	8	4.0	3.1	855	11
{1470 kN}	7	554	2	554	633	554	343	120	10	500	8	3.9	3.0	748	11
	6	455	2	455	773	455	422	120	10	430	8	3.7	2.9	641	11
686x254x140	8	653	2	653	493	653	219	120	10	570	8	4.0	3.1	855	11
{1370 kN}	7	554	2	554	585	554	312	120	10	500	8	3.9	3.0	748	11
	6	455	2	455	714	455	390	120	10	430	8	3.7	2.9	641	11
686x254x125	8	653	2	653	444	653	176	120	10	570	8	4.0	3.1	855	11
{1280 kN}	7	554	2	554	535	554	279	120	10	500	8	3.9	3.0	748	11
	6	455	2	455	653	455	359	120	10	430	8	3.7	2.9	641	11

For guidance on the use of tables see Explanatory notes in Table G.15

BEAM: S275
FIN PLATES: S275
BOLTS: M20, 8.8

Table G.18 *Continued*

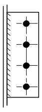
FIN PLATES, ORDINARY BOLTS
Single line of bolts
120x10 or 100x10 mm Fin Plate

Beam size {$V_{Rd,beam}$}	Bolt rows n_1	Un-notched: Shear Resist V_{Rd} kN	Un-notched: Crit chck	Single Notch: Shear Resist V_{Rd} kN	Single Notch: Notch limit l_n mm	Double Notch: Shear Resist V_{Rd} kN	Double Notch: Notch limit l_n mm	Fitting Fin Plate: Width b_p mm	Fitting Fin Plate: Thck t_p mm	Fitting Fin Plate: Length h_p mm	Fillet weld: Leg s mm	Min support thickness: S275 mm	Min support thickness: S355 mm	Tying: Resist $N_{Rd,u}$ kN	Tying: Crit chck
610x305x238	7	576	3	576	744	576	395	100	10	500	8	4.0	3.1	748	11
{1890 kN}	6	481	2	481	892	481	475	100	10	430	8	3.9	3.0	641	11
610x305x179	7	576	3	576	543	576	270	100	10	500	8	4.0	3.1	748	11
{1440 kN}	6	481	2	481	652	481	340	100	10	430	8	3.9	3.0	641	11
610x305x149	7	576	3	576	440	576	174	100	10	500	8	4.0	3.1	748	11
{1200 kN}	6	481	2	481	532	481	267	100	10	430	8	3.9	3.0	641	11
610x229x140	7	576	3	576	479	576	233	100	10	500	8	4.0	3.1	748	11
{1300 kN}	6	481	2	481	575	481	311	100	10	430	8	3.9	3.0	641	11
610x229x125	7	576	3	576	423	576	178	100	10	500	8	4.0	3.1	748	11
{1170 kN}	6	481	2	481	512	481	270	100	10	430	8	3.9	3.0	641	11
610x229x113	7	576	3	576	376	576	132	100	10	500	8	4.0	3.1	748	11
{1090 kN}	6	481	2	481	467	481	237	100	10	430	8	3.9	3.0	641	11
610x229x101	7	576	3	576	351	563	110	100	10	500	8	4.0	3.1	748	11
{1060 kN}	6	481	2	481	445	481	215	100	10	430	8	3.9	3.0	641	11
610x178x100+	7	576	3	576	368	576	142	100	10	500	8	4.0	3.1	748	11
{1110 kN}	6	481	2	481	454	481	244	100	10	430	8	3.9	3.0	641	11
610x178x92+	7	576	3	576	355	576	118	100	10	500	8	4.0	3.1	748	11
{1090 kN}	6	481	2	481	441	481	232	100	10	430	8	3.9	3.0	641	11
610x178x82+	7	557	4	557	315	N/A	110	100	10	500	8	3.9	3.0	748	11
{1000 kN}	6	481	4	481	392	481	187	100	10	430	8	3.9	3.0	641	11
533x312x272+	6	481	2	481	775	481	361	100	10	430	8	3.9	3.0	641	11
{1910 kN}	5	383	2	383	978	383	456	100	10	360	8	3.7	2.9	535	11
533x312x219+	6	481	2	481	679	481	346	100	10	430	8	3.9	3.0	641	11
{1630 kN}	5	383	2	383	856	383	438	100	10	360	8	3.7	2.9	535	11
533x312x182+	6	481	2	481	546	481	272	100	10	430	8	3.9	3.0	641	11
{1340 kN}	5	383	2	383	690	383	347	100	10	360	8	3.7	2.9	535	11
533x312x150+	6	481	2	481	443	481	195	100	10	430	8	3.9	3.0	641	11
{1120 kN}	5	383	2	383	560	383	278	100	10	360	8	3.7	2.9	535	11

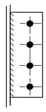
For guidance on the use of tables see Explanatory notes in Table G.15

BEAM: S275
FIN PLATES: S275
BOLTS: M20, 8.8

Table G.18 *Continued*



FIN PLATES, ORDINARY BOLTS
Single line of bolts
120x10 or 100x10 mm Fin Plate



Beam size {$V_{Rd,beam}$}	Bolt rows n_1	Un-notched Shear Resist V_{Rd} kN	Un-notched Crit chck	Single Notch Shear Resist V_{Rd} kN	Single Notch Notch limit l_n mm	Double Notch Shear Resist V_{Rd} kN	Double Notch Notch limit l_n mm	Fitting Fin Plate Width b_p mm	Fitting Fin Plate Thck t_p mm	Fitting Fin Plate Length h_p mm	Fillet weld Leg s mm	Min support thickness S275 mm	Min support thickness S355 mm	Tying Resist $N_{Rd,u}$ kN	Tying Crit chck
533x210x138+	6	481	2	481	494	481	258	100	10	430	8	3.9	3.0	641	11
{1290 kN}	5	383	2	383	624	383	332	100	10	360	8	3.7	2.9	535	11
533x210x122	6	481	2	481	423	481	197	100	10	430	8	3.9	3.0	641	11
{1110 kN}	5	383	2	383	534	383	280	100	10	360	8	3.7	2.9	535	11
533x210x109	6	481	2	481	375	481	151	100	10	430	8	3.9	3.0	641	11
{1020 kN}	5	383	2	383	477	383	246	100	10	360	8	3.7	2.9	535	11
533x210x101	6	481	2	481	338	481	114	100	10	430	8	3.9	3.0	641	11
{952 kN}	5	383	2	383	439	383	220	100	10	360	8	3.7	2.9	535	11
533x210x92	6	481	2	481	317	452	110	100	10	430	8	3.9	3.0	641	11
{909 kN}	5	383	2	383	418	383	202	100	10	360	8	3.7	2.9	535	11
533x210x82	6	462	4	462	299	N/A	110	100	10	430	8	3.8	2.9	641	11
{865 kN}	5	383	2	383	383	383	176	100	10	360	8	3.7	2.9	535	11
533x165x85+	6	480	4	480	301	480	110	100	10	430	8	3.9	3.0	641	11
{902 kN}	5	383	2	383	398	383	201	100	10	360	8	3.7	2.9	535	11
533x165x74+	6	467	4	467	289	N/A	110	100	10	430	8	3.8	2.9	641	11
{871 kN}	5	383	2	383	374	383	181	100	10	360	8	3.7	2.9	535	11
533x165x66+	6	428	4	428	279	N/A	110	100	10	430	8	3.5	2.7	603	12
{793 kN}	5	360	2	360	354	360	165	100	10	360	8	3.5	2.7	503	12

For guidance on the use of tables see Explanatory notes in Table G.15

BEAM: S275
FIN PLATES: S275
BOLTS: M20, 8.8

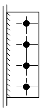
Table G.18 *Continued*

FIN PLATES, ORDINARY BOLTS
Single line of bolts
120x10 or 100x10 mm Fin Plate

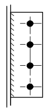
Beam size	Bolt rows	Un-notched		Single Notch		Double Notch		Fitting Fin Plate			Fillet weld	Min support thickness		Tying	
Beam size {$V_{\mathrm{Rd,beam}}$}	n_1	Shear Resist V_{Rd} kN	Crit chck	Shear Resist V_{Rd} kN	Notch limit l_n mm	Shear Resist V_{Rd} kN	Notch limit l_n mm	Width b_p mm	Thck t_p mm	Length h_p mm	Leg s mm	S275 mm	S355 mm	Resist $N_{\mathrm{Rd,u}}$ kN	Crit chck
457x191x161+	5	383	2	383	600	383	309	100	10	360	8	3.7	2.9	535	11
{1390 kN}	4	286	2	286	807	286	418	100	10	290	8	3.4	2.7	428	11
457x191x133+	5	383	2	383	485	383	246	100	10	360	8	3.7	2.9	535	11
{1160 kN}	4	286	2	286	654	286	333	100	10	290	8	3.4	2.7	428	11
457x191x106+	5	383	2	383	379	383	174	100	10	360	8	3.7	2.9	535	11
{947 kN}	4	286	2	286	511	286	255	100	10	290	8	3.4	2.7	428	11
457x191x98	5	383	2	383	342	383	137	100	10	360	8	3.7	2.9	535	11
{852 kN}	4	286	2	286	461	286	228	100	10	290	8	3.4	2.7	428	11
457x191x89	5	383	2	383	304	382	110	100	10	360	8	3.7	2.9	535	11
{789 kN}	4	286	2	286	417	286	203	100	10	290	8	3.4	2.7	428	11
457x191x82	5	383	2	383	289	350	110	100	10	360	8	3.7	2.9	535	11
{756 kN}	4	286	2	286	400	286	192	100	10	290	8	3.4	2.7	428	11
457x191x74	5	364	2	364	267	N/A	110	100	10	360	8	3.5	2.7	508	12
{693 kN}	4	272	2	272	377	272	176	100	10	290	8	3.3	2.5	407	12
457x191x67	5	344	2	344	258	N/A	110	100	10	360	8	3.3	2.6	480	
{650 kN}	4	257	2	257	368	257	170	100	10	290	8	3.1	2.4	384	12
457x152x82	5	383	2	383	300	396	104	100	10	360	8	3.7	2.9	535	11
{798 kN}	4	286	2	286	410	286	206	100	10	290	8	3.4	2.7	428	11
457x152x74	5	376	4	376	265	339	110	100	10	360	8	3.7	2.8	535	11
{721 kN}	4	286	2	286	368	286	177	100	10	290	8	3.4	2.7	428	11
457x152x67	5	364	2	364	260	N/A	110	100	10	360	8	3.5	2.7	508	12
{697 kN}	4	272	2	272	368	272	177	100	10	290	8	3.3	2.5	407	12
457x152x60	5	328	2	328	255	N/A	110	100	10	360	8	3.2	2.5	457	12
{624 kN}	4	245	2	245	362	245	172	100	10	290	8	3.0	2.3	366	12
457x152x52	5	307	2	307	242	N/A	110	100	10	360	8	3.0	2.3	429	12
{578 kN}	4	229	2	229	347	229	165	100	10	290	8	2.8	2.1	343	12

For guidance on the use of tables see Explanatory notes in Table G.15

BEAM: S275
FIN PLATES: S275
Table G.18 *Continued* **BOLTS: M20, 8.8**



FIN PLATES, ORDINARY BOLTS
Single line of bolts
120x10 or 100x10 mm Fin Plate



Beam size $\{V_{Rd,beam}\}$	Bolt rows n_1	Un-notched Shear Resist V_{Rd} kN	Un-notched Crit chck	Single Notch Shear Resist V_{Rd} kN	Single Notch Notch limit l_n mm	Double Notch Shear Resist V_{Rd} kN	Double Notch Notch limit l_n mm	Fitting Fin Plate Width b_p mm	Fitting Fin Plate Thck t_p mm	Fitting Fin Plate Length h_p mm	Fillet weld Leg s mm	Min support thickness S275 mm	Min support thickness S355 mm	Tying Resist $N_{Rd,u}$ kN	Tying Crit chck
406x178x85+ {742 kN}	4	286	2	286	340	286	150	100	10	290	8	3.4	2.7	428	11
406x178x74 {664 kN}	4	286	2	286	301	311	110	100	10	290	8	3.4	2.7	428	11
406x178x67 {612 kN}	4	266	2	266	294	272	103	100	10	290	8	3.2	2.5	398	12
406x178x60 {549 kN}	4	238	2	238	290	234	110	100	10	290	8	2.9	2.2	357	12
406x178x54 {529 kN}	4	232	2	232	279	220	110	100	10	290	8	2.8	2.2	348	12
406x140x53+ {549 kN}	4	238	2	238	280	234	110	100	10	290	8	2.9	2.2	357	12
406x140x46 {473 kN}	4	205	2	205	276	195	110	100	10	290	8	2.5	1.9	307	12
406x140x39 {438 kN}	4	193	2	193	261	177	110	100	10	290	8	2.3	1.8	289	12
356x171x67 {568 kN}	3	185	2	185	338	185	145	100	10	220	8	2.9	2.3	308	12
356x171x57 {501 kN}	3	165	2	165	326	165	137	100	10	220	8	2.6	2.0	274	12
356x171x51 {455 kN}	3	151	2	151	319	151	133	100	10	220	8	2.4	1.9	251	12
356x171x45 {425 kN}	3	142	2	142	308	142	128	100	10	220	8	2.3	1.8	237	12
356x127x39 {408 kN}	3	134	2	134	303	134	131	100	10	220	8	2.1	1.7	224	12
356x127x33 {366 kN}	3	122	2	122	290	122	124	100	10	220	8	1.9	1.5	203	12

For guidance on the use of tables see Explanatory notes in Table G.15

BEAM: S275
FIN PLATES: S275
BOLTS: M20, 8.8

Table G.18 *Continued*

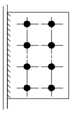
FIN PLATES, ORDINARY BOLTS
Single line of bolts
120x10 or 100x10 mm Fin Plate

Beam size {$V_{Rd,beam}$}	Bolt rows n_1	Un-notched Shear Resist V_{Rd} kN	Un-notched Crit chck	Single Notch Shear Resist V_{Rd} kN	Single Notch Notch limit l_n mm	Double Notch Shear Resist V_{Rd} kN	Double Notch Notch limit l_n mm	Fitting Fin Plate Width b_p mm	Fitting Fin Plate Thck t_p mm	Fitting Fin Plate Length h_p mm	Fillet weld Leg s mm	Min support thickness S275 mm	Min support thickness S355 mm	Tying Resist $N_{Rd,u}$ kN	Tying Crit chck
305x165x54 {422 kN}	3	161	2	161	235	N/A	110	100	10	220	8	2.6	2.0	268	12
305x165x46 {357 kN}	3	136	2	136	230	N/A	110	100	10	220	8	2.2	1.7	227	12
305x165x40 {319 kN}	3	122	2	122	224	N/A	110	100	10	220	8	1.9	1.5	203	12
305x127x48 {474 kN}	3	183	2	183	222	N/A	110	100	10	220	8	2.9	2.3	305	12
305x127x42 {420 kN}	3	163	2	163	216	N/A	110	100	10	220	8	2.6	2.0	271	12
305x127x37 {372 kN}	3	144	2	144	212	N/A	110	100	10	220	8	2.3	1.8	241	12
305x102x33 {350 kN}	3	134	2	134	222	N/A	110	100	10	220	8	2.1	1.7	224	12
305x102x28 {315 kN}	3	122	2	122	212	N/A	110	100	10	220	8	1.9	1.5	203	12
305x102x25 {299 kN}	3	118	2	118	201	N/A	110	100	10	220	8	1.9	1.5	197	12
254x146x43 {321 kN}	2	82	2	82	275	76	110	100	10	150	8	1.9	1.5	163	12
254x146x37 {280 kN}	2	72	2	72	267	63	110	100	10	150	8	1.7	1.3	142	12
254x146x31 {260 kN}	2	68	2	68	251	56	110	100	10	150	8	1.6	1.2	136	12
254x102x28 {283 kN}	2	72	2	72	262	67	110	100	10	150	8	1.7	1.3	142	12
254x102x25 {265 kN}	2	68	2	68	250	61	110	100	10	150	8	1.6	1.2	136	12
254x102x22 {248 kN}	2	65	2	65	238	56	110	100	10	150	8	1.5	1.2	129	12

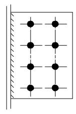
For guidance on the use of tables see Explanatory notes in Table G.15

BEAM: S275
FIN PLATES: S275
BOLTS: M20, 8.8

Table G.19



FIN PLATES, ORDINARY BOLTS
Double line of bolts
180x10 or 160x10 mm Fin Plate



Beam size {$V_{Rd,beam}$}	Bolt rows	Un-notched		Single Notch		Double Notch		Fitting Fin Plate			Fillet weld	Min support thickness		Tying	
	n_1	Shear Resist V_{Rd} kN	Crit chck	Shear Resist V_{Rd} kN	Notch limit l_n mm	Shear Resist V_{Rd} kN	Notch limit l_n mm	Width b_p mm	Thck t_p mm	Length h_p mm	Leg s mm	S275 mm	S355 mm	Resist $N_{Rd,u}$ kN	Crit chck
1016x305x487+ {4930 kN}	8	713	3	713	2260	713	1230	180	10	570	8	4.4	3.4	1320	11
1016x305x437+ {4420 kN}	8	713	3	713	2040	713	1130	180	10	570	8	4.4	3.4	1320	11
1016x305x393+ {3990 kN}	8	713	3	713	1810	713	997	180	10	570	8	4.4	3.4	1320	11
1016x305x349+ {3610 kN}	8	713	3	713	1650	713	921	180	10	570	8	4.4	3.4	1320	11
1016x305x314+ {3260 kN}	8	713	3	713	1470	713	815	180	10	570	8	4.4	3.4	1320	11
1016x305x272+ {2830 kN}	8	713	3	713	1250	713	685	180	10	570	8	4.4	3.4	1320	11
1016x305x249+ {2770 kN}	8	713	3	713	1220	713	702	180	10	570	8	4.4	3.4	1320	11
1016x305x222+ {2640 kN}	8	713	3	713	1130	713	664	180	10	570	8	4.4	3.4	1320	11
914x419x388 {3240 kN}	8	713	3	713	1450	713	748	180	10	570	8	4.4	3.4	1320	11
914x419x343 {2920 kN}	8	713	3	713	1320	713	696	180	10	570	8	4.4	3.4	1320	11
914x305x289 {2900 kN}	8	713	3	713	1300	713	728	180	10	570	8	4.4	3.4	1320	11
914x305x253 {2570 kN}	8	713	3	713	1180	713	698	180	10	570	8	4.4	3.4	1320	11
914x305x224 {2350 kN}	8	713	3	713	1060	713	627	180	10	570	8	4.4	3.4	1320	11
914x305x201 {2210 kN}	8	713	3	713	973	713	583	180	10	570	8	4.4	3.4	1320	11

For guidance on the use of tables see Explanatory notes in Table G.15

BEAM: S275
FIN PLATES: S275
BOLTS: M20, 8.8

Table G.19 *Continued*

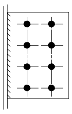
FIN PLATES, ORDINARY BOLTS
Double line of bolts
180x10 or 160x10 mm Fin Plate

Beam size $\{V_{Rd,beam}\}$	Bolt rows n_1	Un-notched Shear Resist V_{Rd} kN	Un-notched Crit chck	Single Notch Shear Resist V_{Rd} kN	Single Notch Notch limit l_n mm	Double Notch Shear Resist V_{Rd} kN	Double Notch Notch limit l_n mm	Fitting Fin Plate Width b_p mm	Fitting Fin Plate Thck t_p mm	Fitting Fin Plate Length h_p mm	Fillet weld Leg s mm	Min support thickness S275 mm	Min support thickness S355 mm	Tying Resist $N_{Rd,u}$ kN	Tying Crit chck
838x292x226 {2220 kN}	8	713	3	713	945	713	543	180	10	570	8	4.4	3.4	1320	11
838x292x194 {2000 kN}	8	713	3	713	829	713	480	180	10	570	8	4.4	3.4	1320	11
838x292x176 {1890 kN}	8	713	3	713	768	713	449	180	10	570	8	4.4	3.4	1320	11
762x267x197	8	713	3	713	735	713	414	180	10	570	8	4.4	3.4	1320	11
{1950 kN}	7	625	3	625	840	625	475	180	10	500	8	4.4	3.4	1160	11
762x267x173	8	713	3	713	654	713	365	180	10	570	8	4.4	3.4	1320	11
{1760 kN}	7	625	3	625	747	625	423	180	10	500	8	4.4	3.4	1160	11
762x267x147	8	713	3	713	565	713	297	180	10	570	8	4.4	3.4	1320	11
{1560 kN}	7	625	3	625	645	625	364	180	10	500	8	4.4	3.4	1160	11
762x267x134	8	713	3	713	539	713	266	180	10	570	8	4.4	3.4	1320	11
{1520 kN}	7	625	3	625	616	625	346	180	10	500	8	4.4	3.4	1160	11
686x254x170	8	713	3	713	548	713	272	180	10	570	8	4.4	3.4	1320	11
{1630 kN}	7	625	3	625	627	625	339	180	10	500	8	4.4	3.4	1160	11
	6	538	3	538	730	538	399	180	10	430	8	4.4	3.4	1000	11
686x254x152	8	713	3	713	487	713	212	180	10	570	8	4.4	3.4	1320	11
{1470 kN}	7	625	3	625	560	625	292	180	10	500	8	4.4	3.4	1160	11
	6	538	3	538	652	538	354	180	10	430	8	4.4	3.4	1000	11
686x254x140	8	713	3	713	442	713	168	180	10	570	8	4.4	3.4	1320	11
{1370 kN}	7	625	3	625	517	625	258	180	10	500	8	4.4	3.4	1160	11
	6	538	3	538	603	538	325	180	10	430	8	4.4	3.4	1000	11
686x254x125	8	713	3	713	392	713	121	180	10	570	8	4.4	3.4	1320	11
{1280 kN}	7	625	3	625	470	625	221	180	10	500	8	4.4	3.4	1160	11
	6	538	3	538	551	538	293	180	10	430	8	4.4	3.4	1000	11

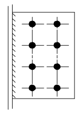
For guidance on the use of tables see Explanatory notes in Table G.15

BEAM: S275
FIN PLATES: S275
BOLTS: M20, 8.8

Table G.19 *Continued*



FIN PLATES, ORDINARY BOLTS
Double line of bolts
180x10 or 160x10 mm Fin Plate

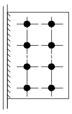


Beam size	Bolt rows	Un-notched		Single Notch		Double Notch		Fitting Fin Plate			Fillet weld	Min support thickness		Tying	
$\{V_{Rd,beam}\}$	n_1	Shear Resist V_{Rd} kN	Crit chck	Shear Resist V_{Rd} kN	Notch limit l_n mm	Shear Resist V_{Rd} kN	Notch limit l_n mm	Width b_p mm	Thck t_p mm	Length h_p mm	Leg s mm	S275 mm	S355 mm	Resist $N_{Rd,u}$ kN	Crit chck
610x305x238	7	625	3	625	685	625	363	160	10	500	8	4.4	3.4	1160	11
{1890 kN}	6	538	3	538	798	538	424	160	10	430	8	4.4	3.4	1000	11
610x305x179	7	625	3	625	500	625	233	160	10	500	8	4.4	3.4	1160	11
{1440 kN}	6	538	3	538	583	538	302	160	10	430	8	4.4	3.4	1000	11
610x305x149	7	625	3	625	395	625	129	160	10	500	8	4.4	3.4	1160	11
{1200 kN}	6	538	3	538	475	538	223	160	10	430	8	4.4	3.4	1000	11
610x229x140	7	625	3	625	438	625	193	160	10	500	8	4.4	3.4	1160	11
{1300 kN}	6	538	3	538	514	538	271	160	10	430	8	4.4	3.4	1000	11
610x229x125	7	625	3	625	378	625	134	160	10	500	8	4.4	3.4	1160	11
{1170 kN}	6	538	3	538	458	538	226	160	10	430	8	4.4	3.4	1000	11
610x229x113	7	625	3	625	329	599	110	160	10	500	8	4.4	3.4	1160	11
{1090 kN}	6	538	3	538	413	538	190	160	10	430	8	4.4	3.4	1000	11
610x229x101	7	625	3	625	302	562	110	160	10	500	8	4.4	3.4	1160	11
{1060 kN}	6	538	3	538	390	538	163	160	10	430	8	4.4	3.4	1000	11
610x178x100+	7	625	3	625	323	610	110	160	10	500	8	4.4	3.4	1160	11
{1110 kN}	6	538	3	538	403	538	198	160	10	430	8	4.4	3.4	1000	11
610x178x92+	7	625	3	625	310	584	110	160	10	500	8	4.4	3.4	1160	11
{1090 kN}	6	538	3	538	391	538	182	160	10	430	8	4.4	3.4	1000	11
610x178x82+	7	625	3	625	248	N/A	110	160	10	500	8	4.4	3.4	1160	12
{1000 kN}	6	538	3	538	338	538	131	160	10	430	8	4.4	3.4	1000	12
533x312x272+	6	538	3	538	693	538	322	160	10	430	8	4.4	3.4	1000	11
{1910 kN}	5	450	3	450	830	450	387	160	10	360	8	4.4	3.4	839	11
533x312x219+	6	538	3	538	607	538	309	160	10	430	8	4.4	3.4	1000	11
{1630 kN}	5	450	3	450	727	450	371	160	10	360	8	4.4	3.4	839	11
533x312x182+	6	538	3	538	488	538	234	160	10	430	8	4.4	3.4	1000	11
{1340 kN}	5	450	3	450	585	450	294	160	10	360	8	4.4	3.4	839	11
533x312x150+	6	538	3	538	393	538	149	160	10	430	8	4.4	3.4	1000	11
{1120 kN}	5	450	3	450	475	450	228	160	10	360	8	4.4	3.4	839	11

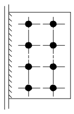
For guidance on the use of tables see Explanatory notes in Table G.15

BEAM: S275
FIN PLATES: S275
BOLTS: M20, 8.8

Table G.19 *Continued*



FIN PLATES, ORDINARY BOLTS
Double line of bolts
180x10 or 160x10 mm Fin Plate

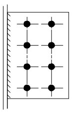


Beam size	Bolt rows	Un-notched		Single Notch		Double Notch		Fitting Fin Plate			Fillet weld	Min support thickness		Tying	
$\{V_{Rd,beam}\}$	n_1	Shear Resist V_{Rd} kN	Crit chck	Shear Resist V_{Rd} kN	Notch limit l_n mm	Shear Resist V_{Rd} kN	Notch limit l_n mm	Width b_p mm	Thck t_p mm	Length h_p mm	Leg s mm	S275 mm	S355 mm	Resist $N_{Rd,u}$ kN	Crit chck
533x210x138+	6	538	3	538	442	538	219	160	10	430	8	4.4	3.4	1000	11
{1290 kN}	5	450	3	450	530	450	281	160	10	360	8	4.4	3.4	839	11
533x210x122	6	538	3	538	375	538	151	160	10	430	8	4.4	3.4	1000	11
{1110 kN}	5	450	3	450	453	450	230	160	10	360	8	4.4	3.4	839	11
533x210x109	6	538	3	538	323	538	100	160	10	430	8	4.4	3.4	1000	11
{1020 kN}	5	450	3	450	404	450	192	160	10	360	8	4.4	3.4	839	11
533x210x101	6	538	3	538	283	486	110	160	10	430	8	4.4	3.4	1000	11
{952 kN}	5	450	3	450	369	450	162	160	10	360	8	4.4	3.4	839	11
533x210x92	6	538	3	538	260	452	110	160	10	430	8	4.4	3.4	1000	11
{909 kN}	5	450	3	450	349	450	138	160	10	360	8	4.4	3.4	839	11
533x210x82	6	530	4	530	227	N/A	110	160	10	430	8	4.3	3.3	960	12
{865 kN}	5	450	3	450	312	485	109	160	10	360	8	4.4	3.4	805	12
533x165x85+	6	538	3	538	243	538	110	160	10	430	8	4.4	3.4	1000	11
{902 kN}	5	450	3	450	330	450	140	160	10	360	8	4.4	3.4	839	11
533x165x74+	6	535	4	535	220	N/A	110	160	10	430	8	4.4	3.4	970	12
{871 kN}	5	450	3	450	307	450	115	160	10	360	8	4.4	3.4	813	12
533x165x66+	6	491	4	491	210	N/A	110	160	10	430	8	4.0	3.1	890	12
{793 kN}	5	423	4	423	285	411	110	160	10	360	8	4.1	3.2	746	12

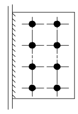
For guidance on the use of tables see Explanatory notes in Table G.15

BEAM: S275
FIN PLATES: S275
BOLTS: M20, 8.8

Table G.19 *Continued*



FIN PLATES, ORDINARY BOLTS
Double line of bolts
180x10 or 160x10 mm Fin Plate

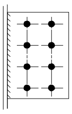


Beam size	Bolt rows	Un-notched		Single Notch		Double Notch		Fitting Fin Plate			Fillet weld	Min support thickness		Tying	
$\{V_{\mathrm{Rd,beam}}\}$	n_1	Shear Resist V_{Rd} kN	Crit chck	Shear Resist V_{Rd} kN	Notch limit l_n mm	Shear Resist V_{Rd} kN	Notch limit l_n mm	Width b_p mm	Thck t_p mm	Length h_p mm	Leg s mm	S275 mm	S355 mm	Resist $N_{\mathrm{Rd,u}}$ kN	Crit chck
457x191x161+	5	450	3	450	509	450	261	160	10	360	8	4.4	3.4	839	11
{1390 kN}	4	363	3	363	634	363	327	160	10	290	8	4.4	3.4	678	11
457x191x133+	5	450	3	450	411	450	201	160	10	360	8	4.4	3.4	839	11
{1160 kN}	4	363	3	363	513	363	260	160	10	290	8	4.4	3.4	678	11
457x191x106+	5	450	3	450	318	450	118	160	10	360	8	4.4	3.4	839	11
{947 kN}	4	363	3	363	401	363	196	160	10	290	8	4.4	3.4	678	11
457x191x98	5	450	3	450	277	412	110	160	10	360	8	4.4	3.4	839	11
{852 kN}	4	363	3	363	361	363	165	160	10	290	8	4.4	3.4	678	11
457x191x89	5	450	3	450	234	373	110	160	10	360	8	4.4	3.4	839	11
{789 kN}	4	363	3	363	325	363	135	160	10	290	8	4.4	3.4	678	11
457x191x82	5	450	3	450	218	350	110	160	10	360	8	4.4	3.4	830	12
{756 kN}	4	363	3	363	310	363	120	160	10	290	8	4.4	3.4	671	12
457x191x74	5	428	4	428	192	N/A	110	160	10	360	8	4.2	3.2	755	12
{693 kN}	4	360	4	360	272	336	110	160	10	290	8	4.3	3.4	610	12
457x191x67	5	404	4	404	184	N/A	110	160	10	360	8	3.9	3.0	713	
{650 kN}	4	340	4	340	264	313	110	160	10	290	8	4.1	3.2	576	12
457x152x82	5	450	3	450	232	377	110	160	10	360	8	4.4	3.4	839	11
{798 kN}	4	363	3	363	320	363	138	160	10	290	8	4.4	3.4	678	11
457x152x74	5	444	4	444	190	339	110	160	10	360	8	4.3	3.3	805	12
{721 kN}	4	363	3	363	281	371	103	160	10	290	8	4.4	3.4	651	12
457x152x67	5	428	4	428	188	N/A	110	160	10	360	8	4.2	3.2	755	12
{697 kN}	4	360	4	360	265	337	110	160	10	290	8	4.3	3.4	610	12
457x152x60	5	385	4	385	183	N/A	110	160	10	360	8	3.7	2.9	679	12
{624 kN}	4	324	4	324	260	299	110	160	10	290	8	3.9	3.0	549	12
457x152x52	5	362	4	362	171	N/A	110	160	10	360	8	3.5	2.7	637	12
{578 kN}	4	304	4	304	248	274	110	160	10	290	8	3.7	2.8	515	12

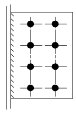
For guidance on the use of tables see Explanatory notes in Table G.15

BEAM: S275
FIN PLATES: S275
BOLTS: M20, 8.8

Table G.19 *Continued*



FIN PLATES, ORDINARY BOLTS
Double line of bolts
180x10 or 160x10 mm Fin Plate

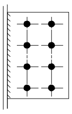


Beam size {$V_{Rd,beam}$}	Bolt rows n_1	Un-notched Shear Resist V_{Rd} kN	Un-notched Crit chck	Single Notch Shear Resist V_{Rd} kN	Single Notch Notch limit l_n mm	Double Notch Shear Resist V_{Rd} kN	Double Notch Notch limit l_n mm	Fitting Fin Plate Width b_p mm	Fitting Fin Plate Thck t_p mm	Fitting Fin Plate Length h_p mm	Fillet weld Leg s mm	Min support thickness S275 mm	Min support thickness S355 mm	Tying Resist $N_{Rd,u}$ kN	Tying Crit chck
406x178x85+ {742 kN}	4	363	3	363	260	331	110	160	10	290	8	4.4	3.4	678	11
406x178x74 {664 kN}	4	363	3	363	217	286	110	160	10	290	8	4.4	3.4	644	12
406x178x67 {612 kN}	4	352	4	352	192	260	110	160	10	290	8	4.2	3.3	596	12
406x178x60 {549 kN}	4	316	4	316	189	230	110	160	10	290	8	3.8	3.0	535	12
406x178x54 {529 kN}	4	308	4	308	179	0	110	160	10	290	8	3.7	2.9	522	12
406x140x53+ {549 kN}	4	316	4	316	182	230	110	160	10	290	8	3.8	3.0	535	12
406x140x46 {473 kN}	4	272	4	272	179	0	110	160	10	290	8	3.3	2.5	461	12
406x140x39 {438 kN}	4	256	4	256	165	0	110	160	10	290	8	3.1	2.4	434	12
356x171x67 {568 kN}	3	236	2	236	262	219	110	160	10	220	8	3.8	2.9	470	12
356x171x57 {501 kN}	3	210	2	210	253	189	110	160	10	220	8	3.3	2.6	418	12
356x171x51 {455 kN}	3	192	2	192	248	169	110	160	10	220	8	3.1	2.4	382	12
356x171x45 {425 kN}	3	182	2	182	239	156	110	160	10	220	8	2.9	2.2	362	12
356x127x39 {408 kN}	3	172	2	172	235	149	110	160	10	220	8	2.7	2.1	341	12
356x127x33 {366 kN}	3	156	2	156	224	131	110	160	10	220	8	2.5	1.9	310	12

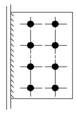
For guidance on the use of tables see Explanatory notes in Table G.15

BEAM: S275
FIN PLATES: S275
BOLTS: M20, 8.8

Table G.19 *Continued*



FIN PLATES, ORDINARY BOLTS
Double line of bolts
180x10 or 160x10 mm Fin Plate

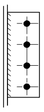


Beam size	Bolt rows	Un-notched		Single Notch		Double Notch		Fitting Fin Plate			Fillet weld	Min support thickness		Tying	
{$V_{Rd,beam}$}	n_1	Shear Resist V_{Rd} kN	Crit chck	Shear Resist V_{Rd} kN	Notch limit l_n mm	Shear Resist V_{Rd} kN	Notch limit l_n mm	Width b_p mm	Thck t_p mm	Length h_p mm	Leg s mm	S275 mm	S355 mm	Resist $N_{Rd,u}$ kN	Crit chck
305x165x54 {422 kN}	3	205	2	205	174	N/A	110	160	10	220	8	3.3	2.5	408	12
305x165x46 {357 kN}	3	174	2	174	170	N/A	110	160	10	220	8	2.8	2.1	346	12
305x165x40 {319 kN}	3	156	2	156	165	N/A	110	160	10	220	8	2.5	1.9	310	12
305x127x48 {474 kN}	3	234	2	234	164	N/A	110	160	10	220	8	3.7	2.9	465	12
305x127x42 {420 kN}	3	208	2	208	158	N/A	110	160	10	220	8	3.3	2.6	413	12
305x127x37 {372 kN}	3	185	2	185	155	N/A	110	160	10	220	8	2.9	2.3	367	12
305x102x33 {350 kN}	3	172	2	172	164	N/A	110	160	10	220	8	2.7	2.1	341	12
305x102x28 {315 kN}	3	156	2	156	156	N/A	110	160	10	220	8	2.5	1.9	310	12
305x102x25 {299 kN}	3	151	2	151	146	N/A	110	160	10	220	8	2.4	1.9	300	12
254x146x43 {321 kN}	2	101	2	101	222	70	110	160	10	150	8	2.4	1.8	256	12
254x146x37 {280 kN}	2	88	2	88	215	58	110	160	10	150	8	2.1	1.6	224	12
254x146x31 {260 kN}	2	84	2	84	203	52	110	160	10	150	8	2.0	1.5	213	12
254x102x28 {283 kN}	2	88	2	88	211	62	110	160	10	150	8	2.1	1.6	224	12
254x102x25 {265 kN}	2	84	2	84	202	57	110	160	10	150	8	2.0	1.5	213	12
254x102x22 {248 kN}	2	80	2	80	192	51	110	160	10	150	8	1.9	1.4	203	12

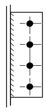
For guidance on the use of tables see Explanatory notes in Table G.15

Beam: S355
FIN PLATES: S275
BOLTS: M20, 8.8

Table G.20



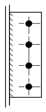
FIN PLATES, ORDINARY BOLTS
Single line of bolts
120x10 or 100x10 mm Fin Plate



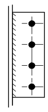
Beam size $\{V_{\mathrm{Rd,beam}}\}$	Bolt rows n_1	Un-notched Shear Resist V_{Rd} kN	Un-notched Crit chck	Single Notch Shear Resist V_{Rd} kN	Single Notch Notch limit l_n mm	Double Notch Shear Resist V_{Rd} kN	Double Notch Notch limit l_n mm	Fitting Fin Plate Width b_p mm	Fitting Fin Plate Thck t_p mm	Fitting Fin Plate Length h_p mm	Fillet weld Leg s mm	Min support thickness S275 mm	Min support thickness S355 mm	Tying Resist $N_{\mathrm{Rd,u}}$ kN	Tying Crit chck
1016x305x487+ {6480 kN}	8	653	2	653	3240	653	1770	120	10	570	8	4.0	3.1	855	11
1016x305x437+ {5810 kN}	8	653	2	653	2920	653	1630	120	10	570	8	4.0	3.1	855	11
1016x305x393+ {5240 kN}	8	653	2	653	2600	653	1440	120	10	570	8	4.0	3.1	855	11
1016x305x349+ {4700 kN}	8	653	2	653	2350	653	1320	120	10	570	8	4.0	3.1	855	11
1016x305x314+ {4240 kN}	8	653	2	653	2100	653	1170	120	10	570	8	4.0	3.1	855	11
1016x305x272+ {3680 kN}	8	653	2	653	1780	653	980	120	10	570	8	4.0	3.1	855	11
1016x305x249+ {3600 kN}	8	653	2	653	1740	653	1000	120	10	570	8	4.0	3.1	855	11
1016x305x222+ {3430 kN}	8	653	2	653	1610	653	951	120	10	570	8	4.0	3.1	855	11
914x419x388 {4220 kN}	8	653	2	653	2060	653	1070	120	10	570	8	4.0	3.1	855	11
914x419x343 {3800 kN}	8	653	2	653	1870	653	997	120	10	570	8	4.0	3.1	855	11
914x305x289 {3780 kN}	8	653	2	653	1850	653	1040	120	10	570	8	4.0	3.1	855	11
914x305x253 {3350 kN}	8	653	2	653	1680	653	1000	120	10	570	8	4.0	3.1	855	11
914x305x224 {3060 kN}	8	653	2	653	1510	653	899	120	10	570	8	4.0	3.1	855	11
914x305x201 {2870 kN}	8	653	2	653	1390	653	837	120	10	570	8	4.0	3.1	855	11

For guidance on the use of tables see Explanatory notes in Table G.15

BEAM: S355
FIN PLATES: S275
Table G.20 *Continued* **BOLTS: M20, 8.8**



FIN PLATES, ORDINARY BOLTS
Single line of bolts
120x10 or 100x10 mm Fin Plate



Beam size $\{V_{Rd,beam}\}$	Bolt rows n_1	Un-notched Shear Resist V_{Rd} kN	Un-notched Crit chck	Single Notch Shear Resist V_{Rd} kN	Single Notch Notch limit l_n mm	Double Notch Shear Resist V_{Rd} kN	Double Notch Notch limit l_n mm	Fitting Fin Plate Width b_p mm	Fitting Fin Plate Thck t_p mm	Fitting Fin Plate Length h_p mm	Fillet weld Leg s mm	Min support thickness S275 mm	Min support thickness S355 mm	Tying Resist $N_{Rd,u}$ kN	Tying Crit chck
838x292x226 {2900 kN}	8	653	2	653	1350	653	779	120	10	570	8	4.0	3.1	855	11
838x292x194 {2610 kN}	8	653	2	653	1180	653	690	120	10	570	8	4.0	3.1	855	11
838x292x176 {2460 kN}	8	653	2	653	1090	653	645	120	10	570	8	4.0	3.1	855	11
762x267x197	8	653	2	653	1050	653	596	120	10	570	8	4.0	3.1	855	11
{2530 kN}	7	554	2	554	1240	554	706	120	10	500	8	3.9	3.0	748	11
762x267x173	8	653	2	653	933	653	532	120	10	570	8	4.0	3.1	855	11
{2290 kN}	7	554	2	554	1100	554	631	120	10	500	8	3.9	3.0	748	11
762x267x147	8	653	2	653	806	653	462	120	10	570	8	4.0	3.1	855	11
{2040 kN}	7	554	2	554	953	554	548	120	10	500	8	3.9	3.0	748	11
762x267x134	8	653	2	653	763	653	436	120	10	570	8	4.0	3.1	855	11
{1970 kN}	7	554	2	554	902	554	522	120	10	500	8	3.9	3.0	748	11
686x254x170	8	653	2	653	783	653	428	120	10	570	8	4.0	3.1	855	11
{2120 kN}	7	554	2	554	925	554	509	120	10	500	8	3.9	3.0	748	11
	6	455	2	455	1130	455	624	120	10	430	8	3.7	2.9	641	11
686x254x152	8	653	2	653	699	653	373	120	10	570	8	4.0	3.1	855	11
{1920 kN}	7	554	2	554	827	554	453	120	10	500	8	3.9	3.0	748	11
	6	455	2	455	1010	455	556	120	10	430	8	3.7	2.9	641	11
686x254x140	8	653	2	653	646	653	333	120	10	570	8	4.0	3.1	855	11
{1790 kN}	7	554	2	554	764	554	418	120	10	500	8	3.9	3.0	748	11
	6	455	2	455	933	455	514	120	10	430	8	3.7	2.9	641	11
686x254x125	8	653	2	653	591	653	290	120	10	570	8	4.0	3.1	855	11
{1670 kN}	7	554	2	554	699	554	385	120	10	500	8	3.9	3.0	748	11
	6	455	2	455	853	455	474	120	10	430	8	3.7	2.9	641	11

For guidance on the use of tables see Explanatory notes in Table G.15

BEAM: S355
FIN PLATES: S275
BOLTS: M20, 8.8

Table G.20 *Continued*

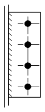
FIN PLATES, ORDINARY BOLTS
Single line of bolts
120x10 or 100x10 mm Fin Plate

	Bolt rows	Un-notched		Single Notch		Double Notch		Fitting Fin Plate			Fillet weld	Min support thickness		Tying	
Beam size $\{V_{\mathrm{Rd,beam}}\}$	n_1	Shear Resist V_{Rd} kN	Crit chck	Shear Resist V_{Rd} kN	Notch limit l_n mm	Shear Resist V_{Rd} kN	Notch limit l_n mm	Width b_p mm	Thck t_p mm	Length h_p mm	Leg s mm	S275 mm	S355 mm	Resist $N_{\mathrm{Rd,u}}$ kN	Crit chck
610x305x238	7	576	3	576	972	576	517	100	10	500	8	4.0	3.1	748	11
{2460 kN}	6	481	2	481	1160	481	621	100	10	430	8	3.9	3.0	641	11
610x305x179	7	576	3	576	710	576	370	100	10	500	8	4.0	3.1	748	11
{1880 kN}	6	481	2	481	851	481	446	100	10	430	8	3.9	3.0	641	11
610x305x149	7	576	3	576	580	576	274	100	10	500	8	4.0	3.1	748	11
{1570 kN}	6	481	2	481	696	481	360	100	10	430	8	3.9	3.0	641	11
610x229x140	7	576	3	576	627	576	333	100	10	500	8	4.0	3.1	748	11
{1690 kN}	6	481	2	481	752	481	409	100	10	430	8	3.9	3.0	641	11
610x229x125	7	576	3	576	558	576	278	100	10	500	8	4.0	3.1	748	11
{1520 kN}	6	481	2	481	670	481	363	100	10	430	8	3.9	3.0	641	11
610x229x113	7	576	3	576	509	576	232	100	10	500	8	4.0	3.1	748	11
{1420 kN}	6	481	2	481	611	481	329	100	10	430	8	3.9	3.0	641	11
610x229x101	7	576	3	576	478	576	197	100	10	500	8	4.0	3.1	748	11
{1370 kN}	6	481	2	481	577	481	308	100	10	430	8	3.9	3.0	641	11
610x178x100+	7	576	3	576	495	576	242	100	10	500	8	4.0	3.1	748	11
{1450 kN}	6	481	2	481	594	481	336	100	10	430	8	3.9	3.0	641	11
610x178x92+	7	576	3	576	476	576	220	100	10	500	8	4.0	3.1	748	11
{1410 kN}	6	481	2	481	573	481	324	100	10	430	8	3.9	3.0	641	11
610x178x82+	7	576	3	576	418	N/A	158	100	10	500	8	4.0	3.1	748	11
{1290 kN}	6	481	2	481	511	481	279	100	10	430	8	3.9	3.0	641	11
533x312x272+	6	481	2	481	1010	481	473	100	10	430	8	3.9	3.0	641	11
{2480 kN}	5	383	2	383	1280	383	597	100	10	360	8	3.7	2.9	535	11
533x312x219+	6	481	2	481	887	481	454	100	10	430	8	3.9	3.0	641	11
{2120 kN}	5	383	2	383	1120	383	574	100	10	360	8	3.7	2.9	535	11
533x312x182+	6	481	2	481	714	481	360	100	10	430	8	3.9	3.0	641	11
{1740 kN}	5	383	2	383	901	383	455	100	10	360	8	3.7	2.9	535	11
533x312x150+	6	481	2	481	580	481	281	100	10	430	8	3.9	3.0	641	11
{1460 kN}	5	383	2	383	732	383	364	100	10	360	8	3.7	2.9	535	11

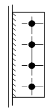
For guidance on the use of tables see Explanatory notes in Table G.15

BEAM: S355
FIN PLATES: S275
BOLTS: M20, 8.8

Table G.20 *Continued*



FIN PLATES, ORDINARY BOLTS
Single line of bolts
120x10 or 100x10 mm Fin Plate

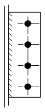


Beam size $\{V_{Rd,beam}\}$	Bolt rows n_1	Un-notched Shear Resist V_{Rd} kN	Un-notched Crit chck	Single Notch Shear Resist V_{Rd} kN	Single Notch Notch limit l_n mm	Double Notch Shear Resist V_{Rd} kN	Double Notch Notch limit l_n mm	Fitting Fin Plate Width b_p mm	Fitting Fin Plate Thck t_p mm	Fitting Fin Plate Length h_p mm	Fillet weld Leg s mm	Min support thickness S275 mm	Min support thickness S355 mm	Tying Resist $N_{Rd,u}$ kN	Tying Crit chck
533x210x138+	6	481	2	481	647	481	344	100	10	430	8	3.9	3.0	641	11
{1680 kN}	5	383	2	383	816	383	435	100	10	360	8	3.7	2.9	535	11
533x210x122	6	481	2	481	553	481	283	100	10	430	8	3.9	3.0	641	11
{1450 kN}	5	383	2	383	698	383	367	100	10	360	8	3.7	2.9	535	11
533x210x109	6	481	2	481	494	481	237	100	10	430	8	3.9	3.0	641	11
{1330 kN}	5	383	2	383	624	383	327	100	10	360	8	3.7	2.9	535	11
533x210x101	6	481	2	481	455	481	200	100	10	430	8	3.9	3.0	641	11
{1240 kN}	5	383	2	383	574	383	299	100	10	360	8	3.7	2.9	535	11
533x210x92	6	481	2	481	428	481	165	100	10	430	8	3.9	3.0	641	11
{1170 kN}	5	383	2	383	542	383	280	100	10	360	8	3.7	2.9	535	11
533x210x82	6	481	2	481	388	N/A	127	100	10	430	8	3.9	3.0	641	11
{1120 kN}	5	383	2	383	498	383	254	100	10	360	8	3.7	2.9	535	11
533x165x85+	6	481	2	481	412	481	172	100	10	430	8	3.9	3.0	641	11
{1170 kN}	5	383	2	383	521	383	280	100	10	360	8	3.7	2.9	535	11
533x165x74+	6	481	2	481	380	N/A	135	100	10	430	8	3.9	3.0	641	11
{1120 kN}	5	383	2	383	486	383	259	100	10	360	8	3.7	2.9	535	11
533x165x66+	6	481	2	481	325	N/A	110	100	10	430	8	3.9	3.0	641	11
{1020 kN}	5	383	2	383	432	383	219	100	10	360	8	3.7	2.9	535	11

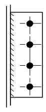
For guidance on the use of tables see Explanatory notes in Table G.15

BEAM: S355
FIN PLATES: S275
BOLTS: M20, 8.8

Table G.20 *Continued*



FIN PLATES, ORDINARY BOLTS
Single line of bolts
120x10 or 100x10 mm Fin Plate



Beam size $\{V_{Rd,beam}\}$	Bolt rows n_1	Un-notched Shear Resist V_{Rd} kN	Un-notched Crit chck	Single Notch Shear Resist V_{Rd} kN	Single Notch Notch limit l_n mm	Double Notch Shear Resist V_{Rd} kN	Double Notch Notch limit l_n mm	Fitting Fin Plate Width b_p mm	Fitting Fin Plate Thck t_p mm	Fitting Fin Plate Length h_p mm	Fillet weld Leg s mm	Min support thickness S275 mm	Min support thickness S355 mm	Tying Resist $N_{Rd,u}$ kN	Tying Crit chck
457x191x161+	5	383	2	383	784	383	405	100	10	360	8	3.7	2.9	535	11
{1810 kN}	4	286	2	286	1050	286	547	100	10	290	8	3.4	2.7	428	11
457x191x133+	5	383	2	383	635	383	323	100	10	360	8	3.7	2.9	535	11
{1510 kN}	4	286	2	286	854	286	436	100	10	290	8	3.4	2.7	428	11
457x191x106+	5	383	2	383	496	383	245	100	10	360	8	3.7	2.9	535	11
{1230 kN}	4	286	2	286	668	286	336	100	10	290	8	3.4	2.7	428	11
457x191x98	5	383	2	383	448	383	208	100	10	360	8	3.7	2.9	535	11
{1110 kN}	4	286	2	286	604	286	299	100	10	290	8	3.4	2.7	428	11
457x191x89	5	383	2	383	405	383	171	100	10	360	8	3.7	2.9	535	11
{1030 kN}	4	286	2	286	546	286	269	100	10	290	8	3.4	2.7	428	11
457x191x82	5	383	2	383	385	383	145	100	10	360	8	3.7	2.9	535	11
{976 kN}	4	286	2	286	519	286	256	100	10	290	8	3.4	2.7	428	11
457x191x74	5	383	2	383	341	N/A	110	100	10	360	8	3.7	2.9	535	11
{895 kN}	4	286	2	286	466	286	225	100	10	290	8	3.4	2.7	428	11
457x191x67	5	383	2	383	306	N/A	110	100	10	360	8	3.7	2.9	535	
{839 kN}	4	286	2	286	428	286	202	100	10	290	8	3.4	2.7	428	11
457x152x82	5	383	2	383	399	383	175	100	10	360	8	3.7	2.9	535	11
{1040 kN}	4	286	2	286	537	286	273	100	10	290	8	3.4	2.7	428	11
457x152x74	5	383	2	383	357	383	132	100	10	360	8	3.7	2.9	535	11
{938 kN}	4	286	2	286	482	286	243	100	10	290	8	3.4	2.7	428	11
457x152x67	5	383	2	383	332	N/A	110	100	10	360	8	3.7	2.9	535	11
{899 kN}	4	286	2	286	454	286	227	100	10	290	8	3.4	2.7	428	11
457x152x60	5	376	2	376	289	N/A	110	100	10	360	8	3.6	2.8	524	12
{806 kN}	4	280	2	280	409	280	195	100	10	290	8	3.4	2.6	420	12
457x152x52	5	352	2	352	274	N/A	110	100	10	360	8	3.4	2.7	492	12
{747 kN}	4	263	2	263	392	263	187	100	10	290	8	3.2	2.5	394	12

For guidance on the use of tables see Explanatory notes in Table G.15

BEAM: S355
FIN PLATES: S275
BOLTS: M20, 8.8

Table G.20 *Continued*

FIN PLATES, ORDINARY BOLTS
Single line of bolts
120x10 or 100x10 mm Fin Plate

Beam size {$V_{Rd,beam}$}	Bolt rows	Un-notched		Single Notch		Double Notch		Fitting Fin Plate			Fillet weld	Min support thickness		Tying	
	n_1	Shear Resist V_{Rd} kN	Crit chck	Shear Resist V_{Rd} kN	Notch limit l_n mm	Shear Resist V_{Rd} kN	Notch limit l_n mm	Width b_p mm	Thck t_p mm	Length h_p mm	Leg s mm	S275 mm	S355 mm	Resist $N_{Rd,u}$ kN	Crit chck
406x178x85+ {966 kN}	4	286	2	286	446	286	209	100	10	290	8	3.4	2.7	428	11
406x178x74 {858 kN}	4	286	2	286	392	286	169	100	10	290	8	3.4	2.7	428	11
406x178x67 {790 kN}	4	286	2	286	355	286	139	100	10	290	8	3.4	2.7	428	11
406x178x60 {709 kN}	4	273	2	273	328	273	111	100	10	290	8	3.3	2.6	409	12
406x178x54 {683 kN}	4	266	2	266	316	276	104	100	10	290	8	3.2	2.5	399	12
406x140x53+ {709 kN}	4	273	2	273	317	273	112	100	10	290	8	3.3	2.6	409	12
406x140x46 {611 kN}	4	235	2	235	313	246	105	100	10	290	8	2.8	2.2	352	12
406x140x39 {566 kN}	4	221	2	221	295	213	110	100	10	290	8	2.7	2.1	331	12
356x171x67 {733 kN}	3	193	2	193	422	193	184	100	10	220	8	3.1	2.4	321	11
356x171x57 {646 kN}	3	189	2	189	368	189	156	100	10	220	8	3.0	2.3	315	12
356x171x51 {587 kN}	3	173	2	173	361	173	151	100	10	220	8	2.7	2.1	287	12
356x171x45 {549 kN}	3	163	2	163	348	163	145	100	10	220	8	2.6	2.0	272	12
356x127x39 {527 kN}	3	154	2	154	343	154	148	100	10	220	8	2.4	1.9	256	12
356x127x33 {472 kN}	3	140	2	140	328	140	141	100	10	220	8	2.2	1.7	233	12

For guidance on the use of tables see Explanatory notes in Table G.15

BEAM: S355
FIN PLATES: S275
BOLTS: M20, 8.8

Table G.20 *Continued*

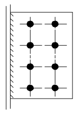
FIN PLATES, ORDINARY BOLTS
Single line of bolts
120x10 or 100x10 mm Fin Plate

Beam size {$V_{Rd,beam}$}	Bolt rows	Un-notched		Single Notch		Double Notch		Fitting Fin Plate			Fillet weld	Min support thickness		Tying	
	n_1	Shear Resist V_{Rd} kN	Crit chck	Shear Resist V_{Rd} kN	Notch limit l_n mm	Shear Resist V_{Rd} kN	Notch limit l_n mm	Width b_p mm	Thck t_p mm	Length h_p mm	Leg s mm	S275 mm	S355 mm	Resist $N_{Rd,u}$ kN	Crit chck
305x165x54 {545 kN}	3	184	2	184	266	N/A	110	100	10	220	8	2.9	2.3	307	12
305x165x46 {461 kN}	3	156	2	156	260	N/A	110	100	10	220	8	2.5	1.9	260	12
305x165x40 {411 kN}	3	140	2	140	254	N/A	110	100	10	220	8	2.2	1.7	233	12
305x127x48 {612 kN}	3	193	2	193	275	N/A	110	100	10	220	8	3.1	2.4	321	11
305x127x42 {542 kN}	3	187	2	187	244	N/A	110	100	10	220	8	3.0	2.3	311	12
305x127x37 {481 kN}	3	166	2	166	240	N/A	110	100	10	220	8	2.6	2.0	276	12
305x102x33 {452 kN}	3	154	2	154	251	N/A	110	100	10	220	8	2.4	1.9	256	12
305x102x28 {407 kN}	3	140	2	140	241	N/A	110	100	10	220	8	2.2	1.7	233	12
305x102x25 {386 kN}	3	135	2	135	228	N/A	110	100	10	220	8	2.2	1.7	225	12
254x146x43 {415 kN}	2	94	2	94	311	98	105	100	10	150	8	2.2	1.7	186	12
254x146x37 {361 kN}	2	82	2	82	302	82	110	100	10	150	8	1.9	1.5	163	12
254x146x31 {336 kN}	2	78	2	78	284	72	110	100	10	150	8	1.8	1.4	155	12
254x102x28 {365 kN}	2	82	2	82	296	87	106	100	10	150	8	1.9	1.5	163	12
254x102x25 {341 kN}	2	78	2	78	283	79	101	100	10	150	8	1.8	1.4	155	12
254x102x22 {320 kN}	2	74	2	74	269	72	110	100	10	150	8	1.7	1.3	148	12

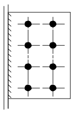
For guidance on the use of tables see Explanatory notes in Table G.15

BEAM: S355
FIN PLATES: S275
BOLTS: M20, 8.8

Table G.21



FIN PLATES, ORDINARY BOLTS
Double line of bolts
180x10 or 160x10 mm Fin Plate



Beam size	Bolt rows	Un-notched		Single Notch		Double Notch		Fitting Fin Plate			Fillet weld	Min support thickness		Tying	
$\{V_{Rd,beam}\}$	n_1	Shear Resist V_{Rd} kN	Crit chck	Shear Resist V_{Rd} kN	Notch limit l_n mm	Shear Resist V_{Rd} kN	Notch limit l_n mm	Width b_p mm	Thck t_p mm	Length h_p mm	Leg s mm	S275 mm	S355 mm	Resist $N_{Rd,u}$ kN	Crit chck
1016x305x487+ {6480 kN}	8	713	3	713	2970	713	1620	180	10	570	8	4.4	3.4	1320	11
1016x305x437+ {5810 kN}	8	713	3	713	2680	713	1490	180	10	570	8	4.4	3.4	1320	11
1016x305x393+ {5240 kN}	8	713	3	713	2380	713	1320	180	10	570	8	4.4	3.4	1320	11
1016x305x349+ {4700 kN}	8	713	3	713	2160	713	1200	180	10	570	8	4.4	3.4	1320	11
1016x305x314+ {4240 kN}	8	713	3	713	1920	713	1070	180	10	570	8	4.4	3.4	1320	11
1016x305x272+ {3680 kN}	8	713	3	713	1630	713	897	180	10	570	8	4.4	3.4	1320	11
1016x305x249+ {3600 kN}	8	713	3	713	1590	713	920	180	10	570	8	4.4	3.4	1320	11
1016x305x222+ {3430 kN}	8	713	3	713	1470	713	870	180	10	570	8	4.4	3.4	1320	11
914x419x388 {4220 kN}	8	713	3	713	1890	713	980	180	10	570	8	4.4	3.4	1320	11
914x419x343 {3800 kN}	8	713	3	713	1720	713	912	180	10	570	8	4.4	3.4	1320	11
914x305x289 {3780 kN}	8	713	3	713	1700	713	954	180	10	570	8	4.4	3.4	1320	11
914x305x253 {3350 kN}	8	713	3	713	1540	713	915	180	10	570	8	4.4	3.4	1320	11
914x305x224 {3060 kN}	8	713	3	713	1380	713	823	180	10	570	8	4.4	3.4	1320	11
914x305x201 {2870 kN}	8	713	3	713	1270	713	766	180	10	570	8	4.4	3.4	1320	11

For guidance on the use of tables see Explanatory notes in Table G.15

BEAM: S355
FIN PLATES: S275
BOLTS: M20, 8.8

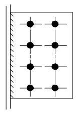
Table G.21 *Continued*

FIN PLATES, ORDINARY BOLTS
Double line of bolts
180x10 or 160x10 mm Fin Plate

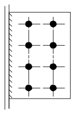
Beam size	Bolt rows	Un-notched		Single Notch		Double Notch		Fitting Fin Plate			Fillet weld	Min support thickness		Tying	
Beam size {$V_{Rd,beam}$}	n_1	Shear Resist V_{Rd} kN	Crit chck	Shear Resist V_{Rd} kN	Notch limit l_n mm	Shear Resist V_{Rd} kN	Notch limit l_n mm	Width b_p mm	Thck t_p mm	Length h_p mm	Leg s mm	S275 mm	S355 mm	Resist $N_{Rd,u}$ kN	Crit chck
838x292x226 {2900 kN}	8	713	3	713	1230	713	713	180	10	570	8	4.4	3.4	1320	11
838x292x194 {2610 kN}	8	713	3	713	1080	713	631	180	10	570	8	4.4	3.4	1320	11
838x292x176 {2460 kN}	8	713	3	713	1000	713	590	180	10	570	8	4.4	3.4	1320	11
762x267x197	8	713	3	713	961	713	545	180	10	570	8	4.4	3.4	1320	11
{2530 kN}	7	625	3	625	1100	625	624	180	10	500	8	4.4	3.4	1160	11
762x267x173	8	713	3	713	855	713	486	180	10	570	8	4.4	3.4	1320	11
{2290 kN}	7	625	3	625	976	625	557	180	10	500	8	4.4	3.4	1160	11
762x267x147	8	713	3	713	738	713	418	180	10	570	8	4.4	3.4	1320	11
{2040 kN}	7	625	3	625	843	625	484	180	10	500	8	4.4	3.4	1160	11
762x267x134	8	713	3	713	699	713	388	180	10	570	8	4.4	3.4	1320	11
{1970 kN}	7	625	3	625	798	625	460	180	10	500	8	4.4	3.4	1160	11
686x254x170	8	713	3	713	717	713	386	180	10	570	8	4.4	3.4	1320	11
{2120 kN}	7	625	3	625	819	625	449	180	10	500	8	4.4	3.4	1160	11
	6	538	3	538	954	538	525	180	10	430	8	4.4	3.4	1000	11
686x254x152	8	713	3	713	641	713	326	180	10	570	8	4.4	3.4	1320	11
{1920 kN}	7	625	3	625	732	625	399	180	10	500	8	4.4	3.4	1160	11
	6	538	3	538	852	538	467	180	10	430	8	4.4	3.4	1000	11
686x254x140	8	713	3	713	592	713	283	180	10	570	8	4.4	3.4	1320	11
{1790 kN}	7	625	3	625	676	625	364	180	10	500	8	4.4	3.4	1160	11
	6	538	3	538	788	538	432	180	10	430	8	4.4	3.4	1000	11
686x254x125	8	713	3	713	540	713	236	180	10	570	8	4.4	3.4	1320	11
{1670 kN}	7	625	3	625	618	625	327	180	10	500	8	4.4	3.4	1160	11
	6	538	3	538	721	538	398	180	10	430	8	4.4	3.4	1000	11

For guidance on the use of tables see Explanatory notes in Table G.15

BEAM: S355
FIN PLATES: S275
Table G.21 *Continued* **BOLTS: M20, 8.8**



FIN PLATES, ORDINARY BOLTS
Double line of bolts
180x10 or 160x10 mm Fin Plate

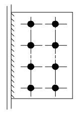


Beam size $\{V_{Rd,beam}\}$	Bolt rows n_1	Un-notched Shear Resist V_{Rd} kN	Un-notched Crit chck	Single Notch Shear Resist V_{Rd} kN	Single Notch Notch limit l_n mm	Double Notch Shear Resist V_{Rd} kN	Double Notch Notch limit l_n mm	Fitting Fin Plate Width b_p mm	Fitting Fin Plate Thck t_p mm	Fitting Fin Plate Length h_p mm	Fillet weld Leg s mm	Min support thickness S275 mm	Min support thickness S355 mm	Tying Resist $N_{Rd,u}$ kN	Tying Crit chck
610x305x238	7	625	3	625	894	625	476	160	10	500	8	4.4	3.4	1160	11
{2460 kN}	6	538	3	538	1040	538	555	160	10	430	8	4.4	3.4	1000	11
610x305x179	7	625	3	625	653	625	334	160	10	500	8	4.4	3.4	1160	11
{1880 kN}	6	538	3	538	761	538	398	160	10	430	8	4.4	3.4	1000	11
610x305x149	7	625	3	625	534	625	230	160	10	500	8	4.4	3.4	1160	11
{1570 kN}	6	538	3	538	622	538	316	160	10	430	8	4.4	3.4	1000	11
610x229x140	7	625	3	625	577	625	293	160	10	500	8	4.4	3.4	1160	11
{1690 kN}	6	538	3	538	672	538	365	160	10	430	8	4.4	3.4	1000	11
610x229x125	7	625	3	625	514	625	234	160	10	500	8	4.4	3.4	1160	11
{1520 kN}	6	538	3	538	599	538	320	160	10	430	8	4.4	3.4	1000	11
610x229x113	7	625	3	625	465	625	185	160	10	500	8	4.4	3.4	1160	11
{1420 kN}	6	538	3	538	546	538	283	160	10	430	8	4.4	3.4	1000	11
610x229x101	7	625	3	625	430	625	145	160	10	500	8	4.4	3.4	1160	11
{1370 kN}	6	538	3	538	516	538	256	160	10	430	8	4.4	3.4	1000	11
610x178x100+	7	625	3	625	453	625	195	160	10	500	8	4.4	3.4	1160	11
{1450 kN}	6	538	3	538	531	538	290	160	10	430	8	4.4	3.4	1000	11
610x178x92+	7	625	3	625	432	625	170	160	10	500	8	4.4	3.4	1160	11
{1410 kN}	6	538	3	538	512	538	275	160	10	430	8	4.4	3.4	1000	11
610x178x82+	7	625	3	625	370	N/A	104	160	10	500	8	4.4	3.4	1160	11
{1290 kN}	6	538	3	538	457	538	225	160	10	430	8	4.4	3.4	1000	11
533x312x272+	6	538	3	538	906	538	422	160	10	430	8	4.4	3.4	1000	11
{2480 kN}	5	450	3	450	1080	450	506	160	10	360	8	4.4	3.4	839	11
533x312x219+	6	538	3	538	793	538	406	160	10	430	8	4.4	3.4	1000	11
{2120 kN}	5	450	3	450	949	450	486	160	10	360	8	4.4	3.4	839	11
533x312x182+	6	538	3	538	639	538	320	160	10	430	8	4.4	3.4	1000	11
{1740 kN}	5	450	3	450	765	450	385	160	10	360	8	4.4	3.4	839	11
533x312x150+	6	538	3	538	518	538	235	160	10	430	8	4.4	3.4	1000	11
{1460 kN}	5	450	3	450	621	450	308	160	10	360	8	4.4	3.4	839	11

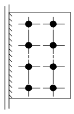
For guidance on the use of tables see Explanatory notes in Table G.15

BEAM: S355
FIN PLATES: S275
BOLTS: M20, 8.8

Table G.21 *Continued*



FIN PLATES, ORDINARY BOLTS
Double line of bolts
180x10 or 160x10 mm Fin Plate

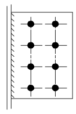


Beam size	Bolt rows	Un-notched		Single Notch		Double Notch		Fitting Fin Plate			Fillet weld	Min support thickness		Tying	
$\{V_{Rd,beam}\}$	n_1	Shear Resist V_{Rd} kN	Crit chck	Shear Resist V_{Rd} kN	Notch limit l_n mm	Shear Resist V_{Rd} kN	Notch limit l_n mm	Width b_p mm	Thck t_p mm	Length h_p mm	Leg s mm	S275 mm	S355 mm	Resist $N_{Rd,u}$ kN	Crit chck
533x210x138+	6	538	3	538	578	538	305	160	10	430	8	4.4	3.4	1000	11
{1680 kN}	5	450	3	450	693	450	369	160	10	360	8	4.4	3.4	839	11
533x210x122	6	538	3	538	494	538	237	160	10	430	8	4.4	3.4	1000	11
{1450 kN}	5	450	3	450	592	450	311	160	10	360	8	4.4	3.4	839	11
533x210x109	6	538	3	538	441	538	186	160	10	430	8	4.4	3.4	1000	11
{1330 kN}	5	450	3	450	529	450	273	160	10	360	8	4.4	3.4	839	11
533x210x101	6	538	3	538	403	538	145	160	10	430	8	4.4	3.4	1000	11
{1240 kN}	5	450	3	450	487	450	243	160	10	360	8	4.4	3.4	839	11
533x210x92	6	538	3	538	372	556	104	160	10	430	8	4.4	3.4	1000	11
{1170 kN}	5	450	3	450	460	450	218	160	10	360	8	4.4	3.4	839	11
533x210x82	6	538	3	538	330	N/A	110	160	10	430	8	4.4	3.4	1000	11
{1120 kN}	5	450	3	450	422	450	188	160	10	360	8	4.4	3.4	839	11
533x165x85+	6	538	3	538	360	538	114	160	10	430	8	4.4	3.4	1000	11
{1170 kN}	5	450	3	450	441	450	221	160	10	360	8	4.4	3.4	839	11
533x165x74+	6	538	3	538	325	N/A	110	160	10	430	8	4.4	3.4	1000	11
{1120 kN}	5	450	3	450	412	450	194	160	10	360	8	4.4	3.4	839	11
533x165x66+	6	538	3	538	265	N/A	110	160	10	430	8	4.4	3.4	1000	11
{1020 kN}	5	450	3	450	364	450	147	160	10	360	8	4.4	3.4	839	11

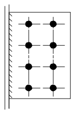
For guidance on the use of tables see Explanatory notes in Table G.15

BEAM: S355
FIN PLATES: S275
BOLTS: M20, 8.8

Table G.21 *Continued*



FIN PLATES, ORDINARY BOLTS
Double line of bolts
180x10 or 160x10 mm Fin Plate

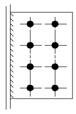


Beam size {$V_{Rd,beam}$}	Bolt rows n_1	Un-notched Shear Resist V_{Rd} kN	Un-notched Crit chck	Single Notch Shear Resist V_{Rd} kN	Single Notch Notch limit l_n mm	Double Notch Shear Resist V_{Rd} kN	Double Notch Notch limit l_n mm	Fitting Fin Plate Width b_p mm	Fitting Fin Plate Thck t_p mm	Fitting Fin Plate Length h_p mm	Fillet weld Leg s mm	Min support thickness S275 mm	Min support thickness S355 mm	Tying Resist $N_{Rd,u}$ kN	Tying Crit chck
457x191x161+	5	450	3	450	665	450	343	160	10	360	8	4.4	3.4	839	11
{1810 kN}	4	363	3	363	828	363	429	160	10	290	8	4.4	3.4	678	11
457x191x133+	5	450	3	450	539	450	273	160	10	360	8	4.4	3.4	839	11
{1510 kN}	4	363	3	363	671	363	342	160	10	290	8	4.4	3.4	678	11
457x191x106+	5	450	3	450	421	450	189	160	10	360	8	4.4	3.4	839	11
{1230 kN}	4	363	3	363	525	363	262	160	10	290	8	4.4	3.4	678	11
457x191x98	5	450	3	450	380	450	146	160	10	360	8	4.4	3.4	839	11
{1110 kN}	4	363	3	363	474	363	233	160	10	290	8	4.4	3.4	678	11
457x191x89	5	450	3	450	339	463	103	160	10	360	8	4.4	3.4	839	11
{1030 kN}	4	363	3	363	428	363	203	160	10	290	8	4.4	3.4	678	11
457x191x82	5	450	3	450	315	415	110	160	10	360	8	4.4	3.4	839	11
{976 kN}	4	363	3	363	407	363	185	160	10	290	8	4.4	3.4	678	11
457x191x74	5	450	3	450	264	N/A	110	160	10	360	8	4.4	3.4	839	11
{895 kN}	4	363	3	363	365	363	147	160	10	290	8	4.4	3.4	678	11
457x191x67	5	450	3	450	225	N/A	110	160	10	360	8	4.4	3.4	817	
{839 kN}	4	363	3	363	335	363	119	160	10	290	8	4.4	3.4	660	12
457x152x82	5	450	3	450	335	483	108	160	10	360	8	4.4	3.4	839	11
{1040 kN}	4	363	3	363	421	363	207	160	10	290	8	4.4	3.4	678	11
457x152x74	5	450	3	450	290	403	110	160	10	360	8	4.4	3.4	839	11
{938 kN}	4	363	3	363	378	363	173	160	10	290	8	4.4	3.4	678	11
457x152x67	5	450	3	450	258	N/A	110	160	10	360	8	4.4	3.4	839	11
{899 kN}	4	363	3	363	355	363	149	160	10	290	8	4.4	3.4	678	11
457x152x60	5	450	3	450	198	N/A	110	160	10	360	8	4.4	3.4	779	12
{806 kN}	4	363	3	363	311	370	102	160	10	290	8	4.4	3.4	629	12
457x152x52	5	450	3	450	147	N/A	110	160	10	360	8	4.4	3.4	731	12
{747 kN}	4	349	2	349	287	327	110	160	10	290	8	4.2	3.3	590	12

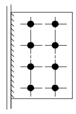
For guidance on the use of tables see Explanatory notes in Table G.15

BEAM: S355
FIN PLATES: S275
BOLTS: M20, 8.8

Table G.21 *Continued*



FIN PLATES, ORDINARY BOLTS
Double line of bolts
180x10 or 160x10 mm Fin Plate



Beam size $\{V_{Rd,beam}\}$	Bolt rows n_1	Un-notched Shear Resist V_{Rd} kN	Un-notched Crit chck	Single Notch Shear Resist V_{Rd} kN	Single Notch Notch limit l_n mm	Double Notch Shear Resist V_{Rd} kN	Double Notch Notch limit l_n mm	Fitting Fin Plate Width b_p mm	Fitting Fin Plate Thck t_p mm	Fitting Fin Plate Length h_p mm	Fillet weld Leg s mm	Min support thickness S275 mm	Min support thickness S355 mm	Tying Resist $N_{Rd,u}$ kN	Tying Crit chck
406x178x85+ {966 kN}	4	363	3	363	349	363	141	160	10	290	8	4.4	3.4	678	11
406x178x74 {858 kN}	4	363	3	363	305	342	110	160	10	290	8	4.4	3.4	678	11
406x178x67 {790 kN}	4	363	3	363	269	311	110	160	10	290	8	4.4	3.4	678	11
406x178x60 {709 kN}	4	363	3	363	220	275	110	160	10	290	8	4.4	3.4	614	12
406x178x54 {683 kN}	4	354	2	354	208	0	110	160	10	290	8	4.3	3.3	598	12
406x140x53+ {709 kN}	4	363	3	363	213	275	110	160	10	290	8	4.4	3.4	614	12
406x140x46 {611 kN}	4	312	2	312	209	0	110	160	10	290	8	3.8	2.9	528	12
406x140x39 {566 kN}	4	294	2	294	192	0	110	160	10	290	8	3.5	2.7	497	12
356x171x67 {733 kN}	3	267	2	267	301	288	109	160	10	220	8	4.2	3.3	517	11
356x171x57 {646 kN}	3	241	2	241	286	230	110	160	10	220	8	3.8	3.0	480	12
356x171x51 {587 kN}	3	220	2	220	280	205	110	160	10	220	8	3.5	2.7	438	12
356x171x45 {549 kN}	3	209	2	209	270	190	110	160	10	220	8	3.3	2.6	415	12
356x127x39 {527 kN}	3	197	2	197	266	181	110	160	10	220	8	3.1	2.4	391	12
356x127x33 {472 kN}	3	179	2	179	254	159	110	160	10	220	8	2.8	2.2	355	12

For guidance on the use of tables see Explanatory notes in Table G.15

BEAM: S355
FIN PLATES: S275
BOLTS: M20, 8.8

Table G.21 *Continued*

FIN PLATES, ORDINARY BOLTS
Double line of bolts
180x10 or 160x10 mm Fin Plate

Beam size {$V_{Rd,beam}$}	Bolt rows n_1	Un-notched Shear Resist V_{Rd} kN	Un-notched Crit chck	Single Notch Shear Resist V_{Rd} kN	Single Notch Notch limit l_n mm	Double Notch Shear Resist V_{Rd} kN	Double Notch Notch limit l_n mm	Fitting Fin Plate Width b_p mm	Fitting Fin Plate Thck t_p mm	Fitting Fin Plate Length h_p mm	Fillet weld Leg s mm	Min support thickness S275 mm	Min support thickness S355 mm	Tying Resist $N_{Rd,u}$ kN	Tying Crit chck
305x165x54 {545 kN}	3	235	2	235	200	N/A	110	160	10	220	8	3.7	2.9	468	12
305x165x46 {461 kN}	3	200	2	200	195	N/A	110	160	10	220	8	3.2	2.5	397	12
305x165x40 {411 kN}	3	179	2	179	189	N/A	110	160	10	220	8	2.8	2.2	355	12
305x127x48 {612 kN}	3	267	2	267	189	N/A	110	160	10	220	8	4.2	3.3	517	11
305x127x42 {542 kN}	3	238	2	238	182	N/A	110	160	10	220	8	3.8	2.9	474	12
305x127x37 {481 kN}	3	212	2	212	178	N/A	110	160	10	220	8	3.4	2.6	420	12
305x102x33 {452 kN}	3	197	2	197	189	N/A	110	160	10	220	8	3.1	2.4	391	12
305x102x28 {407 kN}	3	179	2	179	179	N/A	110	160	10	220	8	2.8	2.2	355	12
305x102x25 {386 kN}	3	173	2	173	167	N/A	110	160	10	220	8	2.7	2.1	343	12
254x146x43 {415 kN}	2	116	2	116	251	91	110	160	10	150	8	2.7	2.1	293	12
254x146x37 {361 kN}	2	101	2	101	244	76	110	160	10	150	8	2.4	1.8	257	12
254x146x31 {336 kN}	2	96	2	96	229	68	110	160	10	150	8	2.2	1.7	245	12
254x102x28 {365 kN}	2	101	2	101	239	80	110	160	10	150	8	2.4	1.8	257	12
254x102x25 {341 kN}	2	96	2	96	229	74	110	160	10	150	8	2.2	1.7	245	12
254x102x22 {320 kN}	2	92	2	92	217	67	110	160	10	150	8	2.1	1.7	232	12

For guidance on the use of tables see Explanatory notes in Table G.15

Table G.22

Explanatory notes – UNIVERSAL COLUMN SPLICES (Bearing Type) Use of Resistance Tables

The following notes apply to Tables G.23 and G.24 which are for bearing type UKC splice connections. The design check numbers noted below refer to those listed in Section 6.5 Design procedures.

1 The ends of the column shaft must be cut to the tolerances required for end bearing, as given in BS EN 1090-2[32] or the National Structural Steelwork Specification[10].

2 Design check 2 for bearing splices should be carried out to see whether the most onerous combination of axial load and bending leads to net tension.

3 If net tension is present then it must not exceed the tensile resistances given in the tables, which are given for the resistance of one flange alone.

4 The tensile resistances given are the minimum from checks 3 and 4. The bearing resistance of the cover plate and the upper column flange have been checked when calculating the quoted resistance.

5 Clause 6.2.7.1(14) of BS EN 1993-1-8 requires that for bearing type splices, splice material must be provided to transmit 25% of the maximum compression force in the column. The compression resistance of the splice material (cover plates and bolts) is provided in the tables, so that this can be compared with 25% of the force in the column.

6 In buildings where it is necessary to comply with structural integrity requirements, the tying resistance of the column splice may be taken as twice the tension value given in each table. This is conservative. Generally (but not always), the tying resistance is twice that quoted for one flange (see Note 3) factored by ${}^{\gamma_{\mathrm{Mu}}}\!/\!{}_{\gamma_{\mathrm{M2}}}$. If the tying resistance is to be calculated precisely, all components should checked as the critical component may be different in the tensile and tying design cases.

7 The tables are based on the use of M20 or M24 property class 8.8 bolts in clearance holes.

Preloaded bolts are required when significant net tension exists (i.e. when the net tensile stress in the upper member exceeds 10% f_y of the upper column) or if joint slip is unacceptable. Bolt spacings and plate thicknesses shown may not be adequate for preloaded bolts and separate checks must be carried out.

When tension is present due only to tying requirements, non-preloaded 8.8 bolts may be used.

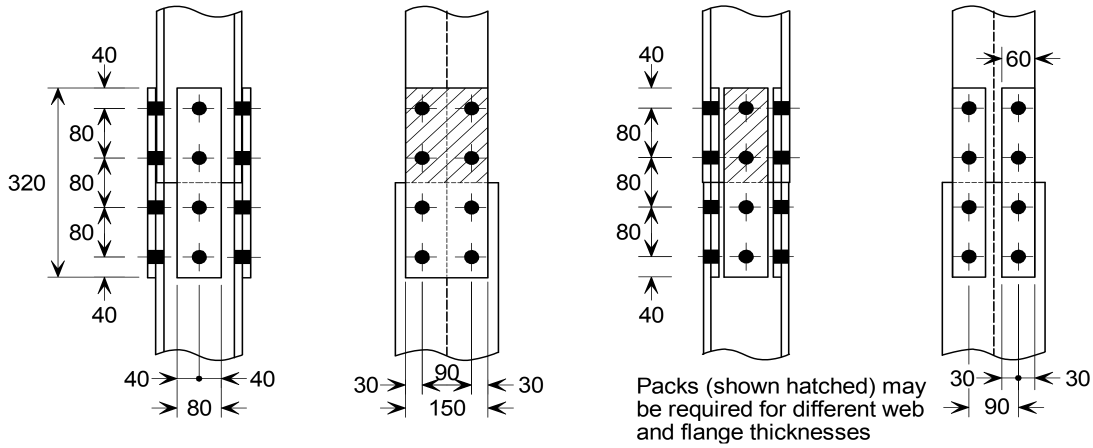
8 The horizontal gauge for the bolts in the column flanges (p_2) has been selected so that the bolts are positioned centrally down an internal flange plate if these are used.

9 Flange and web cover plates are in S275 steel.

10 Where tensile resistance values are marked ‡ the thickness of the flange pack exceeds the commonly accepted limit 4*d*/3 (where '*d*' is the bolt diameter). Although this limit is not in BS EN 1993-1-8, it is anticipated that it will be introduced. If the packs exceed the limit, the pack must be welded to either the column or the cover plate so that the shear resistance of the bolts is not reduced.

Table G.23

Standard Geometry and Tensile Resistances Upper and Lower Columns 152 UKC Series	**UKC 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 UKC	150 x 10 x 320	60 x 10 x 320	80 x 6 x 320

Tensile resistance (one flange)

		Upper Column (kg/m)				
		23	30	37	44	51
Lower Column (kg/m)	23	244				
	30	244	337			
	37	244	337	337		
	44	244	337	337	337	
	51	244	337	337	337	337

Resistance for one external cover plate (kN)

Two internal cover plates provide 256 kN (244 kN with 152 UKC 23 sections).

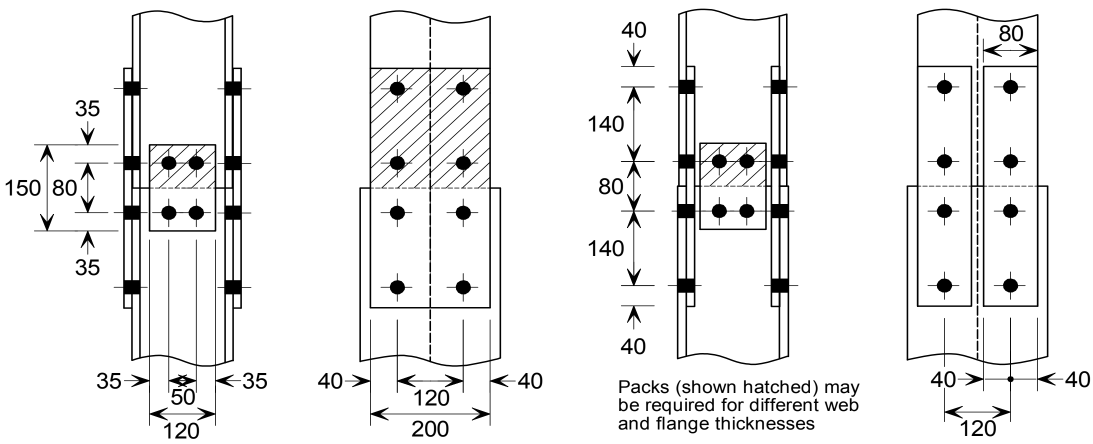
Tying resistance
The tying resistance of the splice may be taken conservatively as twice the tensile resistance of one flange (see Note 6).

Compression resistance
The compression resistance of the splice is twice the tensile resistance of one flange (see Note 5).

For guidance on the use of tables see Explanatory notes in Table G.22

Table G.23 *Continued*

Standard Geometry and Tensile Resistances Upper and Lower Columns 203 UKC Series	**UKC 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 203 x 203 UKC	200 x 10 x 440	80 x 10 x 440	120 x 8 x 150

Tensile resistance (one flange)

		Upper Column (kg/m)							
		46	52	60	71	86	100	113	127
Lower Column (kg/m)	46	376							
	52	376	376						
	60	376	376	376					
	71	376	376	376	376				
	86	359	368	376	376	376			
	100	342	350	359	376	376	376		
	113	326	333	342	359	376	376	376	
	127	312	318	326	341	359	376	376	376

Resistance for one external cover plate (kN)

Two internal cover plates provide 376 kN.

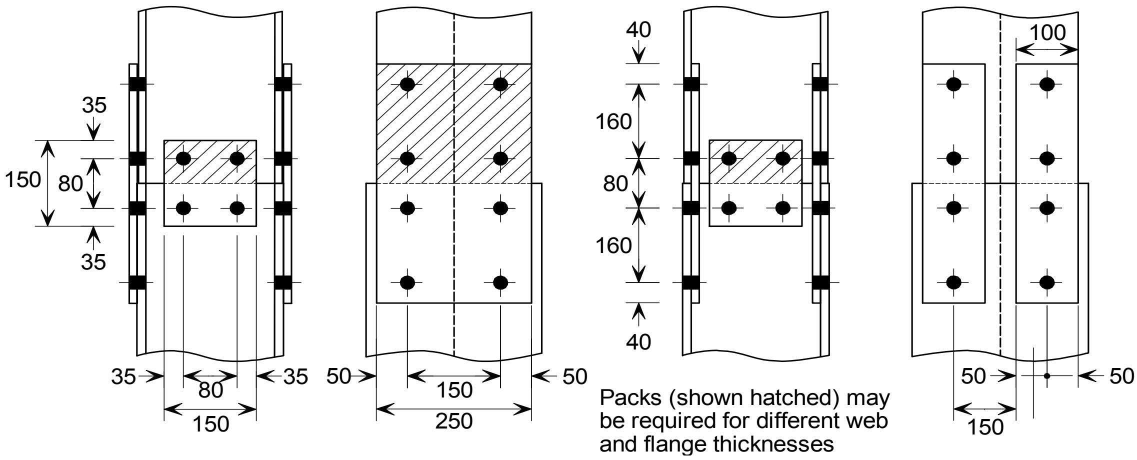
Tying resistance
The tying resistance of the splice may be taken conservatively as twice the tensile resistance of one flange (see Note 6).

Compression resistance
The compression resistance of the splice is twice the tensile resistance of one flange (see Note 5).

For guidance on the use of tables see Explanatory notes in Table G.22

Table G.23 *Continued*

Standard Geometry and Tensile Resistances Upper and Lower Columns 254 UKC Series	**UKC 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)
254 x 254 x 167 UKC	250 x 15 x 480	100 x 15 x 480	150 x 8 x 150
254 x 254 x 132 UKC to 73 UKC	250 x 12 x 480	100 x 12 x 480	150 x 8 x 150

Tensile resistance (one flange)

		Upper Column (kg/m)				
		73	89	107	132	167
Lower Column (kg/m)	73	376				
	89	376	376			
	107	376	376	376		
	132	350	368	376	376	
	167	319	333	350	376	376

Resistance for one external cover plate (kN)

Two internal cover plates provide 376 kN.

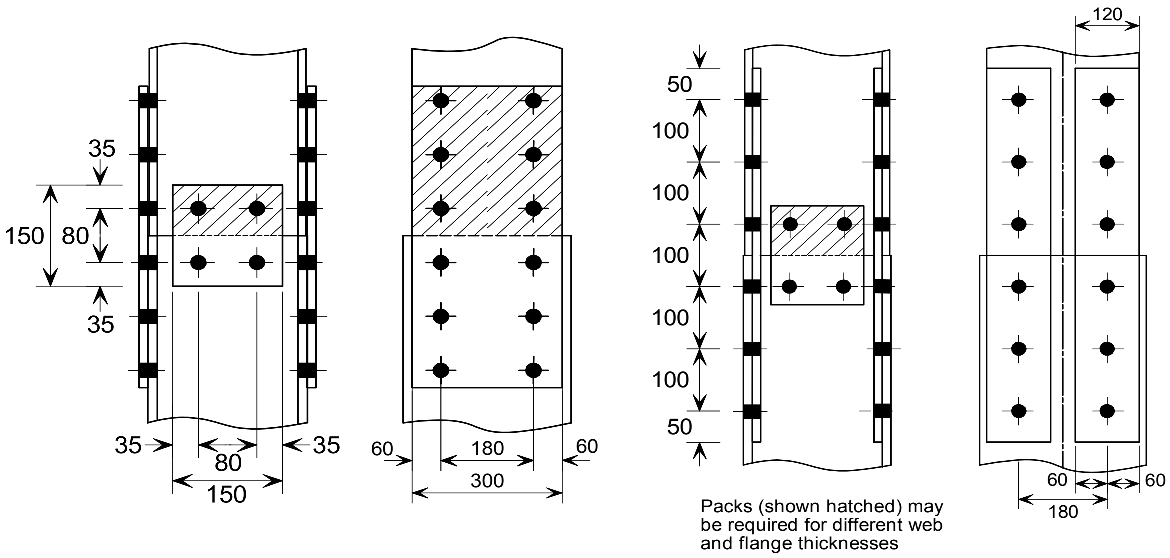
Tying resistance
The tying resistance of the splice may be taken conservatively as twice the tensile resistance of one flange (see Note 6).

Compression resistance
The compression resistance of the splice is twice the tensile resistance of one flange (see Note 5).

For guidance on the use of tables see Explanatory notes in Table G.22

Table G.23 *Continued*

Standard Geometry and Tensile Resistances Upper and Lower Columns 305 UKC Series	**UKC 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 305 UKC ≥ 240kg/m	300 x 20 x 600	120 x 20 x 600	150 x 10 x 150
305 x 305 UKC < 240kg/m	300 x 15 x 600	120 x 15 x 600	150 x 10 x 150

Tensile resistance (one flange)

		Upper Column (kg/m)						
		97	118	137	158	198	240	283
Lower Column (kg/m)	97	813						
	118	813	813					
	137	813	813	813				
	158	796	813	813	813			
	198	732	763	795	813	813		
	240	679	706	732	763	813	813	
	283	632	655	678	705	763	813	813

Resistance for one external cover plate (kN)

Two internal cover plates provide 813 kN.

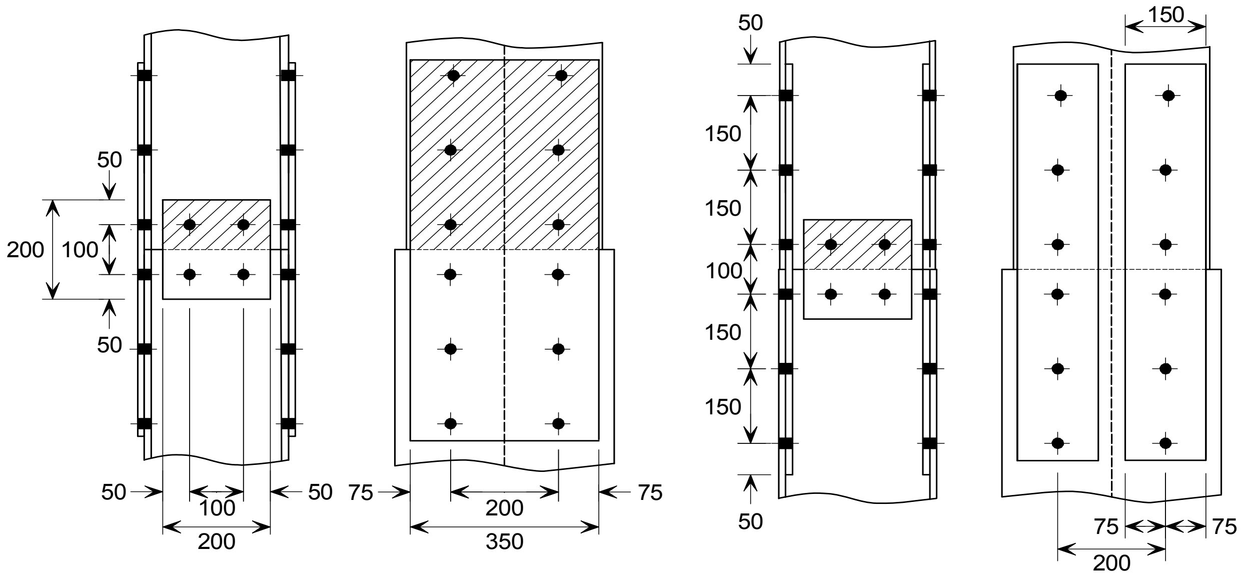
Tying resistance
The tying resistance of the splice may be taken conservatively as twice the tensile resistance of one flange (see Note 6).

Compression resistance
The compression resistance of the splice is twice the tensile resistance of one flange (see Note 5).

For guidance on the use of tables see Explanatory notes in Table G.22

Table G.23 *Continued*

Standard Geometry and Tensile Resistances Upper and Lower Columns 356 x 368 UKC Series	UKC 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)
356 x 368 x 202 UKC	350 x 15 x 800	150 x 15 x 800	200 x 10 x 200
356 x 368 UKC < 202kg/m	350 x 12 x 800	150 x 12 x 800	200 x 10 x 200

Tensile resistance (one flange)

		Upper Column (kg/m)			
		129	153	177	202
Lower Column (kg/m)	129	813			
	153	813	813		
	177	813	813	813	
	202	797	813	813	813

Resistance for one external cover plate (kN)

Two internal cover plates provide 813 kN.

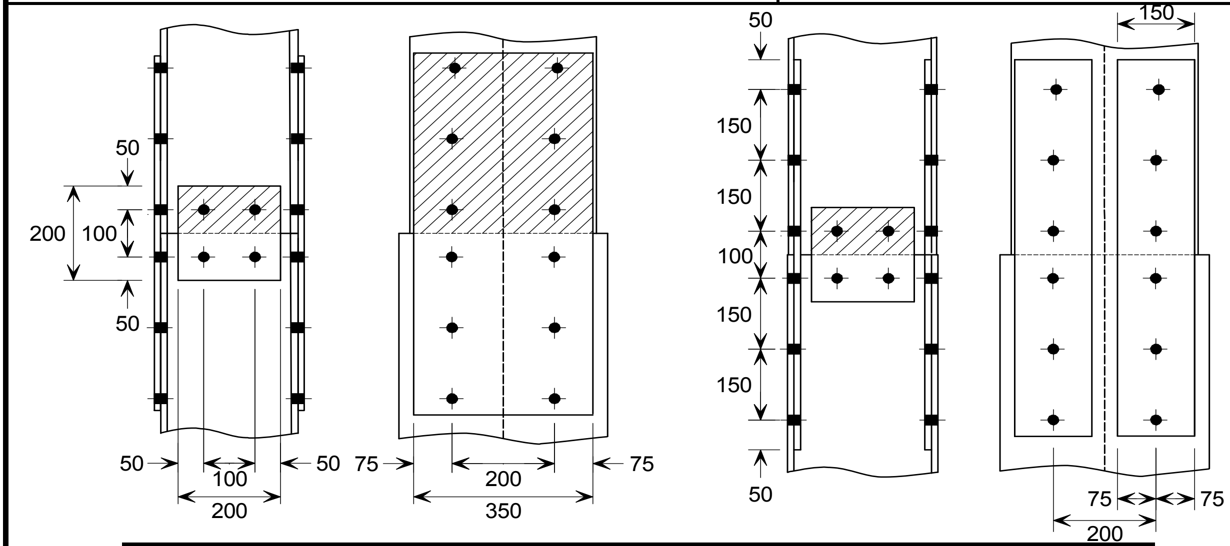
Tying resistance
The tying resistance of the splice may be taken conservatively as twice the tensile resistance of one flange (see Note 6).

Compression resistance
The compression resistance of the splice is twice the tensile resistance of one flange (see Note 5).

For guidance on the use of tables see Explanatory notes in Table G.22

Table G.23 *Continued*

Standard Geometry and Tensile Resistances Upper and Lower Columns 356 x 406 UKC Series	**UKC 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)
*356 x 406 x 634 UKC	350 x 40 x 800	150 x 40 x 800	200 x 15 x 200
*356 x 406 x 551 UKC	350 x 35 x 800	150 x 35 x 800	200 x 15 x 200
*356 x 406 x 467 UKC	350 x 30 x 800	150 x 30 x 800	200 x 12 x 200
*356 x 406 x 393 UKC	350 x 25 x 800	150 x 25 x 800	200 x 12 x 200
*356 x 406 x 340, 287 UKC	350 x 20 x 800	150 x 20 x 800	200 x 10 x 200
356 x 406 x 235 UKC	350 x 15 x 800	150 x 15 x 800	200 x 10 x 200

Tensile resistance (one flange)

		Upper Column (kg/m)						
		235	287	340	393	467	551	634
Lower Column (kg/m)	235	813						
	287	813	813					
	340	763	813	813				
	393	706	763	813	813			
	467	638	685	740	804	813		
	551	578	616	661	712	797	813	
	634	529	560	597	638	706	797	813

Resistance for one external cover plate (kN)

Two internal cover plates provide 813 kN.

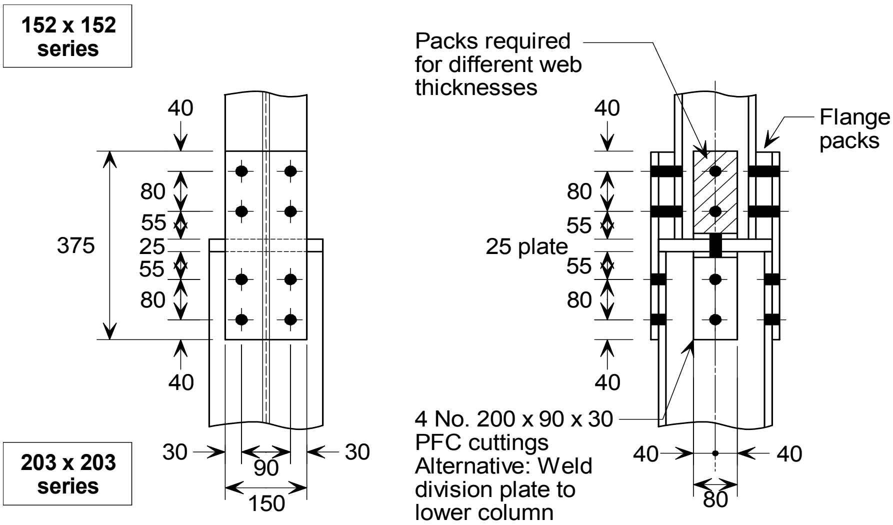
Tying resistance
The tying resistance of the splice may be taken conservatively as twice the tensile resistance of one flange (see Note 6).

Compression resistance
The compression resistance of the splice is twice the tensile resistance of one flange (see Note 5).

For guidance on the use of tables see Explanatory notes in Table G.22

Table G.24

Standard Geometry and Tensile Resistances Upper Column 152 UKC, Lower Column 203 UKC	**UKC COLUMN: S275 or S355 COVER PLATES: S275 BOLTS: M20, 8.8**



Upper Column	Flange Cover Plates (mm)
All 152 x 152 UKC	150 x 10 x 375

Tensile resistance (one flange)

Lower Column 203 x 203 UKC (kg/m)	Upper Column 152 x 152 UKC (kg/m): 23	30	37	44	51
46	287	297	305	314	
52	281‡	291	299	307	317
60	276‡	285	292	301	309
71	266‡	274‡	281‡	289	297
86	256‡	264‡	270‡	277‡	285
100	247‡	254‡	260‡	267‡	274‡
113	239‡	245‡	251‡	257‡	263‡
127	231‡	237‡	242‡	248‡	254‡

Resistance for one external cover plate (kN), ‡ See Note 10

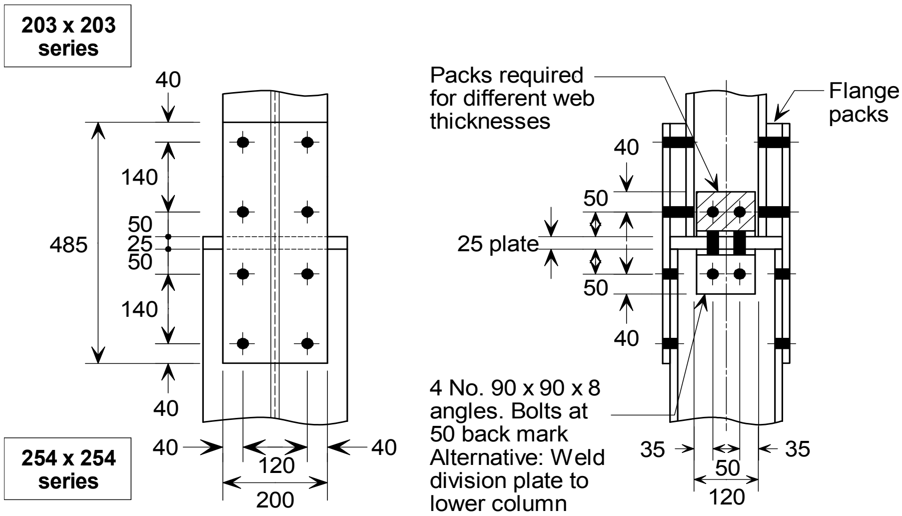
Tying resistance
The tying resistance of the splice may be taken conservatively as twice the tensile resistance of one flange (see Note 6).

Compression resistance
The compression resistance of the splice is twice the tensile resistance of one flange (see Note 5).

For guidance on the use of tables see Explanatory notes in Table G.22

Table G.24 *Continued*

Standard Geometry and Tensile Resistances Upper Column 203 UKC, Lower Column 254 UKC	**UKC COLUMN: S275 or S355 COVER PLATES: S275 BOLTS: M20, 8.8**



Upper Column	Flange Cover Plates (mm)
All 203 x 203 UKC	200 x 10 x 485

Tensile resistance (one flange)

Lower Column 254 x 254 UKC (kg/m)	Upper Column 203 x 203 UKC (kg/m)							
	46	52	60	71	86	100	113	127
73	287	292	299	312				
89	276‡	281‡	287	299	312			
107	265‡	270‡	276‡	287	299			
132	251‡	255‡	260‡	270‡	281‡	293	305	319
167	235‡	238‡	243‡	251‡	260‡	270‡.	281‡	293

Resistance for one external cover plate (kN), ‡ See Note 10

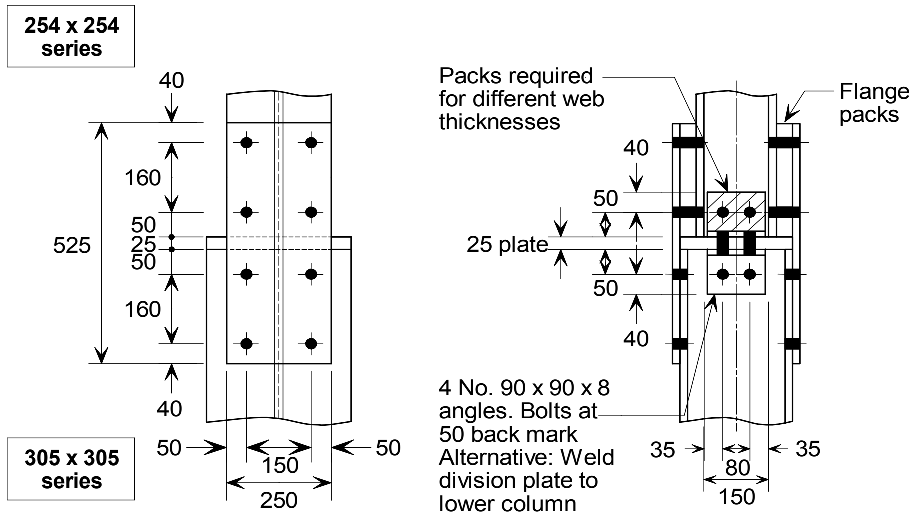
Tying resistance
The tying resistance of the splice may be taken conservatively as twice the tensile resistance of one flange (see Note 6).

Compression resistance
The compression resistance of the splice is twice the tensile resistance of one flange (see Note 5).

For guidance on the use of tables see Explanatory notes in Table G.22

Table G.24 *Continued*

Standard Geometry and Tensile Resistances **Upper Column 254 UKC, Lower Column 305 UKC**	**UKC COLUMN: S275 or S355** **COVER PLATES: S275** **BOLTS: M20, 8.8**



Upper Column	Flange Cover Plates (mm)
254 x 254 UKC 167kg/m	250 x 15 x 525
254 x 254 UKC < 167kg/m	250 x 12 x 525

Tensile resistance (one flange)

		Upper Column 254 x 254 UKC (kg/m)				
		73	89	107	132	167
Lower Column 305 x 305 UKC (kg/m)	97	281‡	293			
	118	270‡	281‡	292		
	137	261‡	271‡	281‡	299	
	158	251‡	260‡	270‡	287	
	198	235‡	242‡	251‡	265‡	287
	240	220‡	227‡	235‡	247‡	266‡
	283	207‡	213‡	220‡	231‡	247‡

Resistance for one external cover plate (kN), ‡ See Note 10

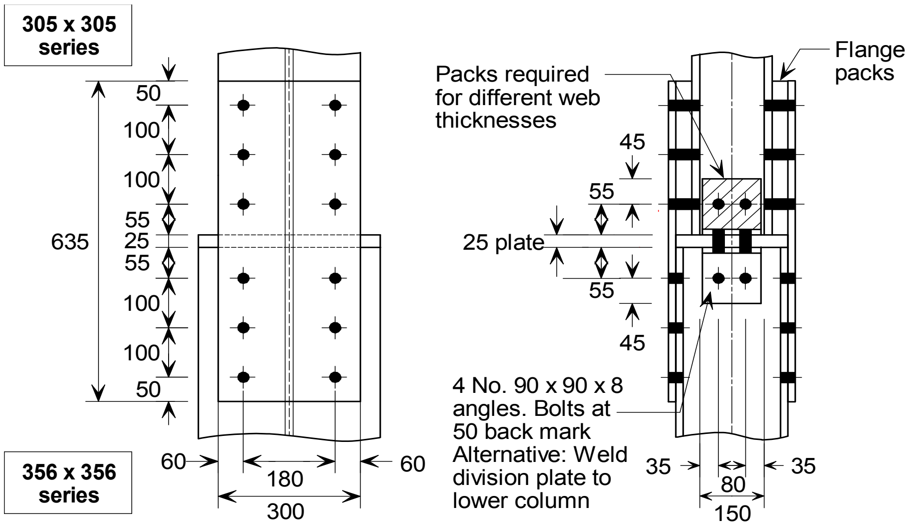
Tying resistance
The tying resistance of the splice may be taken conservatively as twice the tensile resistance of one flange (see Note 6).

Compression resistance
The compression resistance of the splice is twice the tensile resistance of one flange (see Note 5).

For guidance on the use of tables see Explanatory notes in Table G.22

Table G.24 *Continued*

Standard Geometry and Tensile Resistances **Upper Column 305 UKC, Lower Column 356 UKC**	**UKC COLUMN: S275 or S355** **COVER PLATES: S275** **BOLTS: M24, 8.8**



Upper Column	Flange Cover Plates (mm)
305 x 305 UKC ≥ 240kg/m	300 x 20 x 635
305 x 305 UKC < 240kg/m	300 x 15 x 635

Tensile resistance (one flange)

		Upper Column 305 x 305 UKC (kg/m)						
		97	118	137	158	198	240	283
Lower Column 356 x 368 UKC (kg/m)	129	667	693					
	153	643	667	691				
	177	622	645	667	693			
	202	602‡	623	643	667	720		

Resistance for one external cover plate (kN), ‡ See Note 10

Tying resistance
The tying resistance of the splice may be taken conservatively as twice the tensile resistance of one flange (see Note 6).

Compression resistance
The compression resistance of the splice is twice the tensile resistance of one flange (see Note 5).

For guidance on the use of tables see Explanatory notes in Table G.22

Table G.25

Explanatory notes – HOLLOW SECTION TENSION SPLICES
Use of resistance tables

The following notes apply to Tables G.27 to G.29 for hollow section tension splices. The check numbers refer to those listed in Section 6.7 and 6.8.

The connection details are suitable for members in tension.

1 DETAILING

All detailing requirements of Check 1 from Sections 6.7 and 6.8 are adhered to.

The resistance tables are based on the standard connection details that are given in Table G.26 for circular, square and rectangular hollow sections.

Dimension e_1 has been set $\geq$ 51 mm to allow sufficient tightening access when using a torque wrench. (See Table G.61 where clearance *c* = 51 mm for M24 bolts).

2 TENSION RESISTANCE OF THE CONNECTION

The tables are based on M24 property class 8.8 bolts and S275 plates.

Tabulated tension resistances are valid for both S275 and S355 members.

The tension resistances are conservatively based on the dimensions of the heaviest section available for the given external dimensions. Lighter sections will have higher connection resistances than those tabulated.

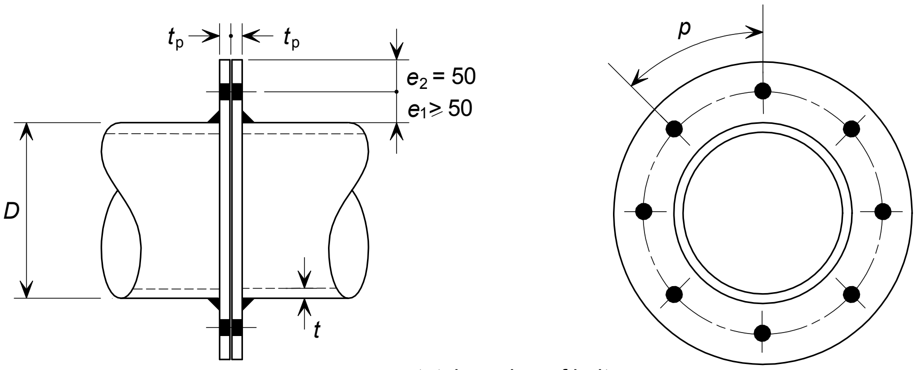
The tension resistances are based on the minimum of Checks 2, 3 and 4 from Section 6.7 for square and rectangular hollow sections and from Section 6.8 for circular hollow sections.

3 CRITICAL DESIGN CHECK

The critical design check indicates whether it is Check 2 (complete end plate yielding) or Check 3 (bolt failure with end plate yielding) which controls the connection resistance.

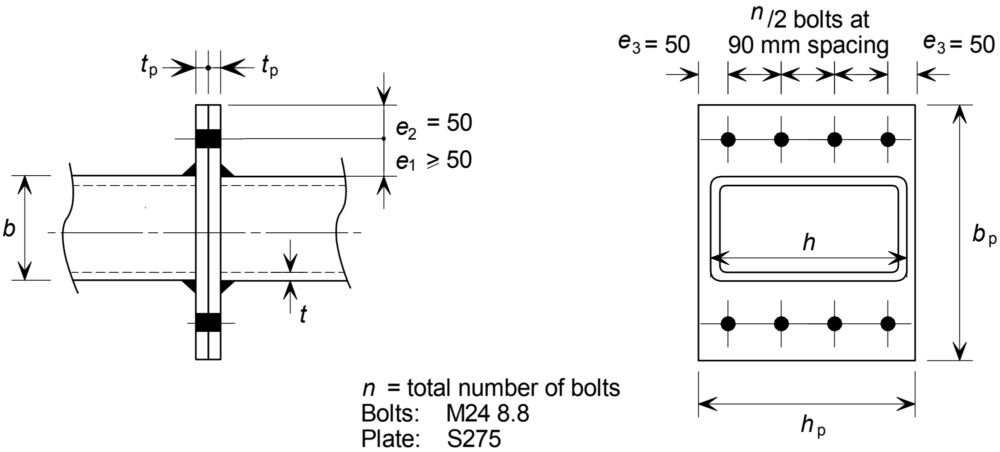
Table G.26

HOLLOW SECTION TENSION SPLICES
Standard Details used in Resistance Tables



n = total number of bolts
Bolts: M24 8.8
Plate: S275

CIRCULAR HOLLOW SECTION SPLICE DETAILS



n = total number of bolts
Bolts: M24 8.8
Plate: S275

SQUARE AND RECTANGULAR HOLLOW SECTION SPLICE DETAILS

CHS: S275 or S355
PLATES: S275
BOLTS: M24, 8.8

Table G.27

TENSION SPLICES
CIRCULAR HOLLOW SECTIONS

Section Diameter	Number of bolts	Bolt spacing p	Plate dimensions: Diameter D_p	Plate dimensions: Thickness t_p	End distance e_1	Edge distance e_2	Tension resistance $N_{t,Rd}$	Critical design check
mm		mm	mm	mm	mm	mm	kN	
114.3	4	173	320	12	53	50	203	2
114.3	4	173	320	15	53	50	318	2
114.3	4	173	320	20	53	50	538	3
139.7	4	196	350	15	55	50	344	2
139.7	4	196	350	20	55	50	522	3
139.7	6	131	350	20	55	50	590	2
139.7	6	131	350	25	55	50	782	3
168.3	6	141	370	15	51	50	409	2
168.3	6	141	370	20	51	50	700	2
168.3	6	141	370	25	51	50	803	3
168.3	8	106	370	25	51	50	1070	3
193.7	6	157	400	15	53	50	426	2
193.7	6	157	400	20	53	50	730	2
193.7	8	118	400	25	53	50	1040	3
219.1	6	173	430	15	55	50	455	2
219.1	6	173	430	20	55	50	764	3
219.1	8	130	430	25	55	50	1020	3
219.1	10	104	430	25	55	50	1220	2
244.5	6	183	450	15	53	50	510	2
244.5	8	137	450	20	53	50	873	2
244.5	10	110	450	25	53	50	1290	3
244.5	12	92	450	25	53	50	1360	2
273	6	199	480	15	54	50	550	2
273	8	149	480	20	54	50	943	2
273	10	119	480	25	54	50	1290	3
273	12	99	480	25	54	50	1470	2
323.9	8	169	530	20	53	50	1030	3
323.9	10	135	530	25	53	50	1290	3
323.9	12	113	530	25	53	50	1540	3
323.9	14	96	530	25	53	50	1700	2
355.6	8	181	560	20	52	50	1030	3
355.6	10	145	560	25	52	50	1290	3
355.6	12	120	560	25	52	50	1550	3
355.6	14	103	560	25	52	50	1810	3
406.4	8	200	610	20	52	50	1030	3
406.4	10	160	610	25	52	50	1290	3
406.4	14	114	610	25	52	50	1810	3
406.4	16	100	610	25	52	50	2070	3
457	10	176	660	20	52	50	1290	3
457	14	126	660	25	52	50	1810	3
457	16	110	660	25	52	50	2070	3
457	20	88	660	25	52	50	2320	2
508	12	160	710	20	51	50	1550	3
508	16	120	710	25	51	50	2070	3
508	18	106	710	25	51	50	2330	3
508	20	96	710	25	51	50	2560	2

For further information on standard details see Table G.26
For guidance on the use of tables see Explanatory notes in Table G.25

SHS: S275 or S355
PLATES: S275
BOLTS: M24, 8.8

Table G.28

TENSION SPLICES
SQUARE HOLLOW SECTIONS

Section size h x b	Number of bolts	Plate dimensions Size h_p mm	b_p mm	Thickness t_p mm	End distance e_1 mm	Edge distance e_2 mm	Tension resistance $N_{t,Rd}$ kN	Critical design check
100 x 100	4	190	310	15	55	50	476	3
100 x 100	4	190	310	20	55	50	510	3
100 x 100	4	190	310	25	55	50	555	3
120 x 120	4	190	330	15	55	50	476	3
120 x 120	4	190	330	20	55	50	510	3
120 x 120	4	190	330	25	55	50	555	3
140 x 140	4	190	350	15	55	50	476	3
140 x 140	4	190	350	20	55	50	510	3
140 x 140	4	190	350	25	55	50	555	3
150 x 150	4	190	360	15	55	50	462	3
150 x 150	4	190	360	20	55	50	495	3
150 x 150	4	190	360	25	55	50	539	3
160 x 160	6	280	370	15	55	50	693	3
160 x 160	6	280	370	20	55	50	742	3
160 x 160	6	280	370	25	55	50	809	3
180 x 180	6	280	390	15	55	50	693	3
180 x 180	6	280	390	20	55	50	742	3
180 x 180	6	280	390	25	55	50	809	3
200 x 200	6	280	410	15	55	50	693	3
200 x 200	6	280	410	20	55	50	742	3
200 x 200	6	280	410	25	55	50	809	3
250 x 250	6	280	460	15	55	50	693	3
250 x 250	6	280	460	20	55	50	742	3
250 x 250	6	280	460	25	55	50	809	3
260 x 260	6	280	470	15	55	50	693	3
260 x 260	6	280	470	20	55	50	742	3
260 x 260	6	280	470	25	55	50	809	3
300 x 300	8	370	510	15	55	50	924	3
300 x 300	8	370	510	20	55	50	990	3
300 x 300	8	370	510	25	55	50	1080	3
350 x 350	8	370	560	15	55	50	924	3
350 x 350	8	370	560	20	55	50	990	3
350 x 350	8	370	560	25	55	50	1080	3
400 x 400	8	370	610	15	55	50	895	3
400 x 400	8	370	610	20	55	50	958	3
400 x 400	8	370	610	25	55	50	1040	3

For further information on standard details see Table G.26
For guidance on the use of tables see Explanatory notes in Table G.25

RHS: S275 or S355
PLATES: S275
BOLTS: M24, 8.8

Table G.29

TENSION SPLICES
RECTANGULAR HOLLOW SECTIONS

Section size	Number of bolts	Plate dimensions			End distance	Edge distance	Tension resistance	Critical design check
		Size		Thickness				
$h \times b$	bolts	h_p	b_p	t_p	e_1	e_2	$N_{t,Rd}$	check
		mm	mm	mm	mm	mm	kN	
200 x 100	6	280	310	15	55	50	693	3
200 x 100	6	280	310	20	55	50	742	3
200 x 100	6	280	310	25	55	50	809	3
200 x 120	6	280	330	15	55	50	693	3
200 x 120	6	280	330	20	55	50	742	3
200 x 120	6	280	330	25	55	50	809	3
200 x 150	6	280	360	15	55	50	693	3
200 x 150	6	280	360	20	55	50	742	3
200 x 150	6	280	360	25	55	50	809	3
220 x 120	6	280	330	15	55	50	693	3
220 x 120	6	280	330	20	55	50	742	3
220 x 120	6	280	330	25	55	50	809	3
250 x 100	6	280	310	15	55	50	693	3
250 x 100	6	280	310	20	55	50	742	3
250 x 100	6	280	310	25	55	50	809	3
250 x 150	6	280	360	15	55	50	693	3
250 x 150	6	280	360	20	55	50	742	3
250 x 150	6	280	360	25	55	50	809	3
260 x 140	6	280	350	15	55	50	693	3
260 x 140	6	280	350	20	55	50	742	3
260 x 140	6	280	350	25	55	50	809	3
300 x 100	8	370	310	15	55	50	924	3
300 x 100	8	370	310	20	55	50	990	3
300 x 100	8	370	310	25	55	50	1080	3
300 x 150	8	370	360	15	55	50	924	3
300 x 150	8	370	360	20	55	50	990	3
300 x 150	8	370	360	25	55	50	1080	3
300 x 200	8	370	410	15	55	50	924	3
300 x 200	8	370	410	20	55	50	990	3
300 x 200	8	370	410	25	55	50	1080	3
300 x 250	8	370	460	15	55	50	924	3
300 x 250	8	370	460	20	55	50	990	3
300 x 250	8	370	460	25	55	50	1080	3
340 x 100	8	370	310	15	55	50	972	3
340 x 100	8	370	310	20	55	50	1040	3
340 x 100	8	370	310	25	55	50	1130	3

For further information on standard details see Table G.26

For guidance on the use of tables see Explanatory notes in Table G.25

RHS: S275 or S355
PLATES: S275
BOLTS: M24, 8.8

Table G.29 *Continued*

TENSION SPLICES
RECTANGULAR HOLLOW SECTIONS

Section size $h \times b$	Number of bolts	Plate dimensions			End distance	Edge distance	Tension resistance	Critical design check
		Size		Thickness				
		h_p mm	b_p mm	t_p mm	e_1 mm	e_2 mm	$N_{t,Rd}$ kN	
350 x 150	8	370	360	15	55	50	924	3
350 x 150	8	370	360	20	55	50	990	3
350 x 150	8	370	360	25	55	50	1080	3
350 x 250	8	370	460	15	55	50	924	3
350 x 250	8	370	460	20	55	50	990	3
350 x 250	8	370	460	25	55	50	1080	3
400 x 150	10	460	360	15	55	50	1160	3
400 x 150	10	460	360	20	55	50	1240	3
400 x 150	10	460	360	25	55	50	1350	3
400 x 200	10	460	410	15	55	50	1160	3
400 x 200	10	460	410	20	55	50	1240	3
400 x 200	10	460	410	25	55	50	1350	3
400 x 300	10	460	510	15	55	50	1160	3
400 x 300	10	460	510	20	55	50	1240	3
400 x 300	10	460	510	25	55	50	1350	3
450 x 250	12	550	460	15	55	50	1390	3
450 x 250	12	550	460	20	55	50	1480	3
450 x 250	12	550	460	25	55	50	1620	3
500 x 200	12	550	410	15	55	50	1390	3
500 x 200	12	550	410	20	55	50	1480	3
500 x 200	12	550	410	25	55	50	1620	3
500 x 300	12	550	510	15	55	50	1340	3
500 x 300	12	550	510	20	55	50	1440	3
500 x 300	12	550	510	25	55	50	1570	3

For further information on standard details see Table G.26
For guidance on the use of tables see Explanatory notes in Table G.25

Table G.30

Explanatory notes – COLUMN BASES
Use of resistance tables

The following notes apply to Tables G.32 to G.35. The check numbers refer to those listed in Section 7.5.

1 BASE PLATE SIZE

See Table G.31 for standard base plate details.

2 AXIAL RESISTANCE OF BASE PLATE

The tabulated resistances are axial compression resistances based on the minimum resistance from Checks 1 and 2 in Section 8.5 (effective area method).

The resistances 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 resistances than those tabulated.

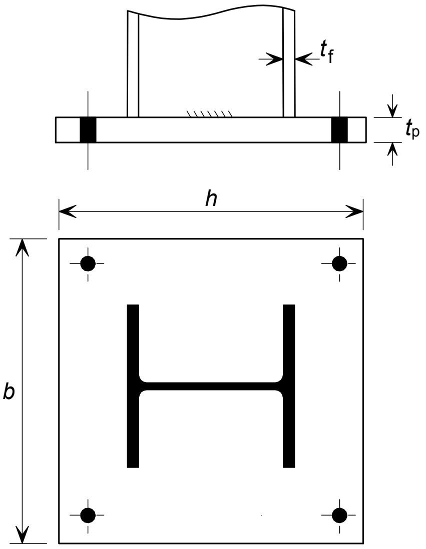
Resistances are tabulated for a range of cube strengths of bedding material/foundation concrete. The resistance based on the cube strength of the weaker material should be used.

The calculated resistances are for S275 base plates.

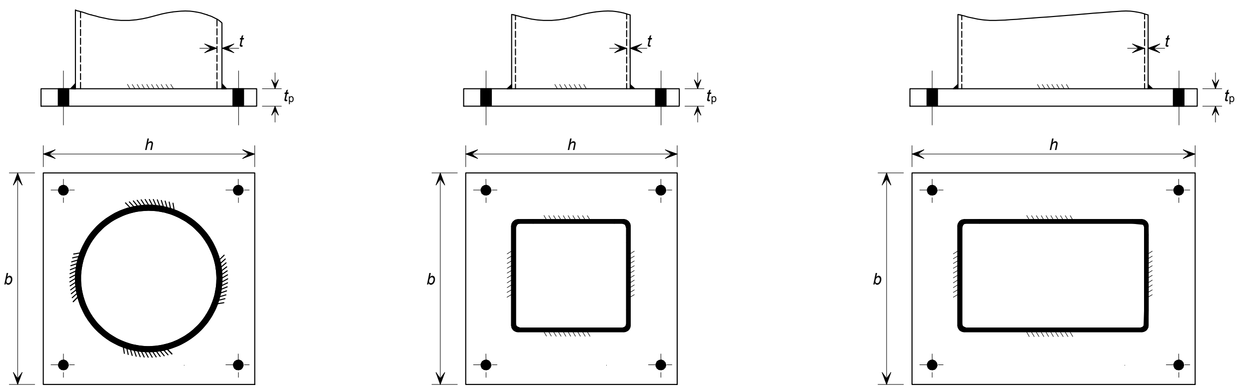
Tabulated base plate resistances are valid for both S275 and S355 columns.

Table G.31

COLUMN BASES
Standard Details used in Resistance Tables



UNIVERSAL COLUMN BASE PLATE DETAILS



HOLLOW SECTION COLUMN BASE PLATE DETAILS

COLUMN: S275 or S355

Table G.32 **PLATES: S275**

COLUMN BASES UNIVERSAL COLUMNS							
Column size h x b mm	Base plate size h_p x b_p x t_p mm	Base Plate Mass kg	Axial resistance (kN)				
			Concrete grade				
			C16/20	C20/25	C25/30	C30/37	C35/45
			Cylinder strengths f_{ck} (N/mm^2)				
			16	20	25	30	35
152x152	300 x 300 x 15	11	493	545	604	659	709
	20	14	671	737	812	880	943
	25	18	816	966	1050	1130	1210
	30	21	816	1020	1280	1400	1490
	350 x 350 x 20	19	671	737	812	880	943
	25	24	862	966	1050	1130	1210
	30	29	1050	1160	1290	1400	1490
	35	34	1110	1370	1520	1650	1790
203x203	400 x 400 x 20	25	868	961	1070	1160	1250
	25	31	1120	1230	1360	1470	1580
	30	38	1380	1510	1660	1800	1930
	35	44	1450	1800	1990	2150	2290
	40	50	1450	1810	2270	2530	2670
	450 x 450 x 25	40	1120	1230	1360	1470	1580
	30	48	1380	1510	1660	1800	1930
	35	56	1610	1800	1990	2150	2290
	40	64	1840	2060	2300	2530	2670
	45	72	1840	2290	2550	2780	3010
	500 x 500 x 25	49	1120	1230	1360	1470	1580
	30	59	1380	1510	1660	1800	1930
	40	79	1860	2060	2300	2530	2670
	50	98	2270	2580	2850	3100	3340
	60	118	2270	2830	3500	3780	4050
254x254	450 x 450 x 25	40	1360	1500	1670	1820	1960
	30	48	1670	1840	2030	2210	2370
	35	56	1820	2210	2410	2610	2800
	40	64	1840	2290	2780	3030	3240
	45	72	1840	2300	2850	3340	3630
	50	79	1840	2300	2870	3430	3930
	500 x 500 x 25	49	1360	1500	1670	1820	1960
	30	59	1670	1840	2030	2210	2370
	40	79	2270	2560	2810	3030	3240
	50	98	2270	2830	3530	3840	4090
	60	118	2270	2830	3540	4250	4960
	600 x 600 x 30	85	1670	1840	2030	2210	2370
	40	113	2300	2560	2810	3030	3240
	50	141	2850	3160	3530	3840	4090
	60	170	3260	3850	4260	4640	5010
	70	198	3260	4080	4940	5350	5750
	700 x 700 x 40	154	2300	2560	2810	3030	3240
	50	192	2850	3160	3530	3840	4090
	60	231	3510	3850	4260	4640	5010
	70	269	4130	4500	4940	5350	5750
	80	308	4440	5300	5780	6220	6650

For further information on standard details see Table G.31

For guidance on the use of tables see Explanatory notes in Table G.30

COLUMN: S275 or S355

Table G.32 *Continued* **PLATES: S275**

COLUMN BASES UNIVERSAL COLUMNS							
			Axial resistance (kN)				
			Concrete grade				
			C16/20	C20/25	C25/30	C30/37	C35/45
Column size h x b mm	Base plate size h_p x b_p x t_p mm	Base Plate Mass kg	Cylinder strengths f_{ck} (N/mm^2)				
			16	20	25	30	35
305 x 305	500 x 500 x 25	49	1600	1780	1980	2160	2340
	30	59	1950	2160	2390	2610	2810
	40	79	2210	2670	3220	3550	3810
	50	98	2270	2830	3430	4010	4560
	60	118	2270	2830	3540	4220	4820
	600 x 600 x 30	85	1950	2160	2390	2610	2810
	40	113	2700	2970	3270	3550	3810
	50	141	3260	3770	4130	4460	4770
	60	170	3260	4080	5090	5520	5880
	70	198	3260	4080	5100	6120	6920
	700 x 700 x 40	154	2700	2970	3270	3550	3810
	50	192	3400	3770	4130	4460	4770
	60	231	4120	4570	5110	5520	5880
	70	269	4440	5280	5850	6400	6920
	80	308	4440	5550	6760	7340	7900
	800 x 800 x 40	201	2700	2970	3270	3550	3810
	50	251	3400	3770	4130	4460	4770
	60	301	4120	4570	5110	5520	5880
	70	352	4790	5280	5850	6400	6920
	80	402	5610	6140	6760	7340	7900
	90	452	5800	6890	7550	8160	8750
356 x 368	600 x 600 x 30	85	2280	2540	2820	3080	3330
	40	113	3050	3460	3830	4160	4470
	50	141	3230	3910	4730	5200	5570
	60	170	3260	4080	4990	5830	6650
	70	198	3260	4080	5100	6050	6920
	700 x 700 x 40	154	3140	3460	3830	4160	4470
	50	192	3970	4360	4800	5200	5570
	60	231	4440	5410	5910	6380	6820
	70	269	4440	5550	6860	7460	7950
	800 x 800 x 40	201	3140	3460	3830	4160	4470
	50	251	3970	4360	4800	5200	5570
	60	301	4820	5410	5910	6380	6820
	70	352	5550	6170	6910	7460	7950
	80	402	5800	7100	7890	8640	9350
	90	452	5800	7250	8740	9520	10300

For further information on standard details see Table G.31

For guidance on the use of tables see Explanatory notes in Table G.30

COLUMN: S275 or S355

Table G.32 *Continued* **PLATES: S275**

COLUMN BASES UNIVERSAL COLUMNS							
			Axial resistance (kN)				
			Concrete grade				
			C16/20	C20/25	C25/30	C30/37	C35/45
Column size h x b mm	Base plate size h_p x b_p x t_p mm	Base Plate Mass kg	Cylinder strengths f_{ck} (N/mm^2)				
			16	20	25	30	35
356 x 406	800 x 800 x 40	201	3420	3790	4220	4610	4980
(≤393 kg/m)	50	251	4290	4730	5230	5690	6120
	60	301	5170	5820	6390	6930	7430
	70	352	5800	6610	7430	8060	8620
	80	402	5800	7250	8440	9270	10100
	90	452	5800	7250	9070	10200	11000
	900 x 900 x 50	318	4290	4730	5230	5690	6120
	60	382	5170	5820	6390	6930	7430
	70	445	5920	6610	7430	8060	8620
	80	509	6830	7570	8440	9270	10100
	90	572	7340	8400	9320	10200	11000
	100	636	7340	9180	10400	11400	12200
	1000 x 1000 x 50	393	4290	4730	5230	5690	6120
	60	471	5170	5820	6390	6930	7430
	70	550	5920	6610	7430	8060	8620
	80	628	6830	7570	8440	9270	10100
	90	707	7620	8400	9320	10200	11000
	100	785	8630	9460	10400	11400	12200
	110	864	9070	10300	11300	12300	13200
	120	942	9070	11300	12500	13500	14500
356 x 406	900 x 900 x 50	318	4750	5280	5890	6460	7000
(>393 kg/m)	60	382	5680	6420	7110	7760	8370
	70	445	6470	7260	8190	8920	9600
	80	509	7280	8260	9260	10200	11100
	90	572	7340	9030	10200	11200	12100
	100	636	7340	9180	11300	12400	13400
	1000 x 1000 x 50	393	4750	5280	5890	6460	7000
	60	471	5680	6420	7110	7760	8370
	70	550	6470	7260	8190	8920	9600
	80	628	7420	8260	9260	10200	11100
	90	707	8240	9130	10200	11200	12100
	100	785	9070	10200	11300	12400	13400
	110	864	9070	11100	12300	13400	14400
	120	942	9070	11300	13500	14700	15700
	1100 x 1100 x 70	665	6470	7260	8190	8920	9600
	80	760	7420	8260	9260	10200	11100
	90	855	8240	9130	10200	11200	12100
	100	950	9290	10200	11300	12400	13400
	110	1045	10100	11100	12300	13400	14400
	120	1140	11000	12300	13500	14700	15700
	1250 x 1250 x 80	981	7420	8260	9260	10200	11100
	90	1104	8240	9130	10200	11200	12100
	100	1227	9290	10200	11300	12400	13400
	120	1472	11300	12300	13500	14700	15700
	140	1717	13700	14800	16200	17400	18600
	150	1840	14200	16200	17600	18900	20100

For further information on standard details see Table G.31

For guidance on the use of tables see Explanatory notes in Table G.30

Table G.33 — **COLUMN: S275 or S355** / **PLATES: S275**

COLUMN BASES
CIRCULAR HOLLOW SECTIONS

Column size d mm	Base plate size $h_p \times b_p \times t_p$ mm	Base Plate Mass kg	Axial resistance (kN)				
			Concrete grade				
			C16/20	C20/25	C25/30	C30/37	C35/45
			Cylinder strength (N/mm^2)				
			16	20	25	30	35
139.7	200 x 200 x 15	4.7	330	391	458	518	571
	20	6.3	359	437	526	606	680
	250 x 250 x 15	7.4	383	430	483	531	575
	20	9.8	482	557	626	688	745
	25	12.3	534	638	754	850	924
	300 x 300 x 20	14.1	498	558	626	688	745
	25	17.7	623	694	778	854	924
	30	21.2	718	832	932	1020	1100
	350 x 350 x 20	19.2	498	558	626	688	745
	25	24.0	623	694	778	854	924
	30	28.8	761	840	932	1020	1100
168.3	250 x 250 x 15	7.4	453	523	591	651	707
	20	9.8	523	620	728	823	907
	25	12.3	558	680	817	940	1050
	300 x 300 x 15	10.6	467	525	591	651	707
	20	14.1	604	678	762	838	909
	25	17.7	712	832	943	1040	1120
	30	21.2	769	921	1090	1230	1340
	350 x 350 x 20	19.2	604	678	762	838	909
	25	24.0	749	840	943	1040	1120
	30	28.8	896	1000	1120	1240	1340
	35	33.7	991	1160	1310	1430	1550
	400 x 400 x 20	25.1	604	678	762	838	909
	25	31.4	749	840	943	1040	1120
	30	37.7	900	1000	1120	1240	1340
	35	44.0	1060	1180	1310	1430	1550
	40	50.2	1220	1360	1510	1640	1770
193.7	300 x 300 x 15	10.6	539	607	683	752	817
	20	14.1	673	779	880	969	1050
	25	17.7	755	897	1050	1190	1300
	30	21.2	798	969	1160	1330	1490
	350 x 350 x 20	19.2	698	783	880	969	1050
	25	24.0	865	971	1090	1200	1300
	30	28.8	976	1140	1300	1430	1550
	35	33.7	1040	1250	1470	1650	1790
	400 x 400 x 20	25.1	698	783	880	969	1050
	25	31.4	865	971	1090	1200	1300
	30	37.7	1030	1160	1300	1430	1550
	35	44.0	1190	1350	1510	1660	1790
	40	50.2	1300	1520	1720	1890	2040
	450 x 450 x 25	39.7	865	971	1090	1200	1300
	30	47.7	1030	1160	1300	1430	1550
	35	55.6	1210	1350	1510	1660	1790
	40	63.6	1400	1550	1720	1890	2040
	45	71.5	1540	1720	1910	2080	2250

For further information on standard details see Table G.31

For guidance on the use of tables see Explanatory notes in Table G.30

COLUMN: S275 or S355
PLATES: S275

Table G.33 *Continued*

COLUMN BASES CIRCULAR HOLLOW SECTIONS

Column size d mm	Base plate size $h_p \times b_p \times t_p$ mm	Base Plate Mass kg	Axial resistance (kN) Concrete grade C16/20, Cylinder strength (N/mm^2) 16	C20/25, 20	C25/30, 25	C30/37, 30	C35/45, 35
219.1	350 x 350 x 20	19.2	790	887	996	1100	1190
	25	24.0	940	1090	1230	1360	1470
	30	28.8	1030	1220	1430	1610	1750
	35	33.7	1080	1310	1560	1780	1990
	400 x 400 x 20	25.1	790	887	996	1100	1190
	25	31.4	981	1100	1230	1360	1470
	30	37.7	1170	1310	1470	1620	1750
	35	44.0	1280	1500	1710	1880	2030
	40	50.2	1360	1620	1910	2140	2320
	450 x 450 x 25	39.7	981	1100	1230	1360	1470
	30	47.7	1170	1310	1470	1620	1750
	35	55.6	1360	1530	1710	1880	2030
	40	63.6	1530	1740	1950	2140	2320
	45	71.5	1630	1900	2150	2360	2550
	500 x 500 x 25	49.1	981	1100	1230	1360	1470
	30	58.9	1170	1310	1470	1620	1750
	40	78.5	1560	1740	1950	2140	2320
	50	98.1	1910	2160	2390	2620	2830
	60	118	2120	2510	2900	3170	3400
244.5	400 x 400 x 20	25.1	885	994	1120	1230	1330
	25	31.4	1100	1230	1380	1520	1650
	30	37.7	1250	1450	1650	1810	1960
	35	44.0	1340	1600	1870	2100	2280
	40	50.2	1400	1690	2020	2300	2560
	450 x 450 x 25	39.7	1100	1230	1380	1520	1650
	30	47.7	1310	1470	1650	1810	1960
	35	55.6	1510	1710	1920	2100	2280
	40	63.6	1630	1910	2180	2400	2590
	45	71.5	1700	2020	2370	2640	2850
	500 x 500 x 25	49.1	1100	1230	1380	1520	1650
	30	58.9	1310	1470	1650	1810	1960
	40	78.5	1740	1950	2180	2400	2590
	50	98.1	2020	2350	2660	2920	3160
	60	118	2180	2610	3080	3480	3780
	600 x 600 x 30	84.8	1310	1470	1650	1810	1960
	40	113	1740	1950	2180	2400	2590
	50	141	2160	2390	2660	2920	3160
	60	170	2650	2920	3220	3510	3780

For further information on standard details see Table G.31

For guidance on the use of tables see Explanatory notes in Table G.30

COLUMN: S275 or S355
PLATES: S275

Table G.33 *Continued*

COLUMN BASES CIRCULAR HOLLOW SECTIONS							
Column size d mm	Base plate size $h_p \times b_p \times t_p$ mm	Base Plate Mass kg	Axial resistance (kN)				
			Concrete grade				
			C16/20	C20/25	C25/30	C30/37	C35/45
			Cylinder strength (N/mm^2)				
			16	20	25	30	35
273	400 x 400 x 20	25.1	991	1110	1250	1380	1490
	25	31.4	1170	1360	1550	1700	1840
	30	37.7	1300	1530	1790	2010	2200
	35	44.0	1380	1660	1960	2230	2480
	40	50.2	1430	1740	2090	2400	2690
	450 x 450 x 25	39.7	1230	1380	1550	1700	1840
	30	47.7	1460	1650	1850	2030	2200
	35	55.6	1610	1880	2140	2350	2550
	40	63.6	1710	2030	2380	2670	2900
	45	71.5	1760	2120	2520	2870	3170
	500 x 500 x 25	49.1	1230	1380	1550	1700	1840
	30	58.9	1470	1650	1850	2030	2200
	40	78.5	1900	2180	2440	2680	2900
	50	98.1	2100	2500	2930	3270	3540
	60	118	2220	2700	3230	3710	4150
	600 x 600 x 30	84.8	1470	1650	1850	2030	2200
	40	113	1940	2180	2440	2680	2900
	50	141	2390	2660	2980	3270	3540
	60	170	2810	3220	3570	3910	4230
	70	198	3020	3560	4110	4490	4830
	700 x 700 x 40	154	1940	2180	2440	2680	2900
	50	192	2390	2660	2980	3270	3540
	60	231	2920	3220	3570	3910	4230
	70	269	3420	3750	4130	4490	4830
323.9	450 x 450 x 25	39.7	1400	1620	1840	2030	2190
	30	47.7	1560	1830	2130	2390	2610
	35	55.6	1680	2000	2350	2670	2950
	40	63.6	1770	2120	2530	2890	3220
	45	71.5	1810	2200	2640	3040	3410
	500 x 500 x 25	49.1	1460	1640	1840	2030	2190
	30	58.9	1730	1960	2200	2410	2610
	40	78.5	2060	2430	2830	3180	3450
	50	98.1	2210	2670	3190	3650	4060
	60	118	2260	2800	3420	3980	4490
	600 x 600 x 30	84.8	1750	1960	2200	2410	2610
	40	113	2310	2590	2900	3190	3450
	50	141	2750	3160	3550	3890	4210
	60	170	3020	3580	4190	4650	5030
	70	198	3160	3810	4530	5170	5720
	700 x 700 x 40	154	2310	2590	2900	3190	3450
	50	192	2830	3160	3550	3890	4210
	60	231	3400	3790	4240	4650	5030
	70	269	3810	4360	4840	5310	5740
	80	308	4100	4820	5560	6070	6550

For further information on standard details see Table G.31
For guidance on the use of tables see Explanatory notes in Table G.30

Table G.33 *Continued*

COLUMN: S275 or S355
PLATES: S275

COLUMN BASES CIRCULAR HOLLOW SECTIONS							
Column size	Base plate size	Base Plate Mass	Axial resistance (kN)				
			Concrete grade				
			C16/20	C20/25	C25/30	C30/37	C35/45
d	$h_p \times b_p \times t_p$	kg	Cylinder strength (N/mm^2)				
mm	mm		16	20	25	30	35
355.6	500 x 500 x 25	49.1	1600	1810	2040	2240	2430
	30	58.9	1810	2110	2420	2670	2890
	40	78.5	2100	2500	2950	3350	3710
	50	98.1	2240	2720	3260	3750	4210
	60	118	2270	2820	3460	4050	4590
	600 x 600 x 30	84.8	1930	2160	2430	2670	2890
	40	113	2540	2850	3200	3520	3810
	50	141	2900	3400	3900	4290	4640
	60	170	3110	3740	4430	5030	5530
	70	198	3210	3910	4720	5440	6080
	700 x 700 x 40	154	2550	2850	3200	3520	3810
	50	192	3110	3480	3900	4290	4640
	60	231	3670	4160	4670	5120	5540
	70	269	3970	4660	5320	5840	6320
	80	308	4210	5020	5910	6640	7200
	800 x 800 x 40	201	2550	2850	3200	3520	3810
	50	251	3110	3480	3900	4290	4640
	60	301	3720	4160	4670	5120	5540
	70	352	4280	4760	5320	5840	6320
	80	402	4870	5480	6090	6660	7200
406.4	500 x 500 x 25	49.1	1650	1920	2230	2500	2750
	30	58.9	1840	2160	2520	2850	3150
	40	78.5	2090	2500	2970	3390	3790
	50	98.1	2210	2690	3250	3760	4230
	60	118	2260	2800	3430	4020	4570
	600 x 600 x 30	84.8	2200	2470	2780	3060	3310
	40	113	2730	3180	3660	4030	4360
	50	141	3030	3600	4220	4780	5270
	60	170	3200	3890	4650	5340	5960
	70	198	3260	4020	4890	5680	6410
	700 x 700 x 40	154	2920	3270	3670	4030	4360
	50	192	3540	3990	4470	4910	5320
	60	231	3960	4650	5340	5870	6350
	70	269	4180	5000	5890	6650	7240
	80	308	4340	5260	6310	7250	8070
	800 x 800 x 40	201	2920	3270	3670	4030	4360
	50	251	3560	3990	4470	4910	5320
	60	301	4260	4770	5340	5870	6350
	70	352	4800	5440	6090	6690	7240
	80	402	5190	6080	6950	7620	8250

For further information on standard details see Table G.31

For guidance on the use of tables see Explanatory notes in Table G.30

COLUMN: S275 or S355

Table G.33 *Continued* **PLATES: S275**

COLUMN BASES CIRCULAR HOLLOW SECTIONS							
Column size	Base plate size	Base Plate Mass	Axial resistance (kN)				
			Concrete grade				
			C16/20	C20/25	C25/30	C30/37	C35/45
d	$h_p \times b_p \times t_p$	kg	Cylinder strength (N/mm^2)				
mm	mm		16	20	25	30	35
457	600 x 600 x 30	84.8	2360	2720	3120	3440	3730
	40	113	2800	3290	3840	4340	4790
	50	141	3060	3650	4320	4940	5500
	60	170	3210	3910	4700	5430	6100
	70	198	3260	4030	4920	5740	6490
	700 x 700 x 40	154	3280	3680	4130	4540	4920
	50	192	3780	4400	5040	5530	5990
	60	231	4130	4910	5760	6500	7140
	70	269	4310	5210	6200	7090	7880
	80	308	4410	5420	6560	7590	8530
	800 x 800 x 40	201	3280	3680	4130	4540	4920
	50	251	4010	4490	5040	5530	5990
	60	301	4740	5370	6020	6610	7150
	70	352	5130	6010	6870	7530	8150
	80	402	5430	6490	7630	8570	9290
	900 x 900 x 50	318	4010	4490	5040	5530	5990
	60	382	4800	5370	6020	6610	7150
	70	445	5470	6130	6870	7530	8150
	80	509	6130	6990	7830	8590	9290
	90	572	6500	7590	8610	9450	10200
508	700 x 700 x 40	154	3450	4000	4580	5050	5470
	50	192	3880	4570	5330	6000	6610
	60	231	4200	5020	5940	6770	7530
	70	269	4360	5290	6330	7280	8160
	80	308	4440	5480	6660	7740	8730
	800 x 800 x 40	201	3660	4100	4600	5050	5470
	50	251	4440	5000	5610	6160	6670
	60	301	5010	5840	6700	7350	7960
	70	352	5350	6340	7420	8330	9070
	80	402	5600	6750	8020	9140	10100
	900 x 900 x 50	318	4460	5000	5610	6160	6670
	60	382	5340	5980	6700	7350	7960
	70	445	6010	6820	7640	8390	9070
	80	509	6510	7630	8710	9560	10300
	90	572	6790	8080	9440	10500	11400
	100	636	7040	8470	10100	11400	12600

For further information on standard details see Table G.31

For guidance on the use of tables see Explanatory notes in Table G.30

COLUMN: S275 or S355

Table G.34 **PLATES: S275**

COLUMN BASES
SQUARE HOLLOW SECTIONS

Column size $h \times b$ mm	Base plate size $h_p \times b_p \times t_p$ mm	Base Plate Mass kg	Axial resistance (kN)				
			Concrete grade				
			C16/20	C20/25	C25/30	C30/37	C35/45
			Cylinder strength (N/mm^2)				
			16	20	25	30	35
150 x 150	300 x 300 x 15	10.6	528	594	668	736	799
	20	14.1	683	767	862	948	1030
	25	17.7	816	950	1070	1170	1270
	30	21.2	816	1020	1270	1400	1510
	350 x 350 x 20	19.2	683	767	862	948	1030
	25	24.0	849	950	1070	1170	1270
	30	28.8	1030	1140	1270	1400	1510
	35	33.7	1110	1350	1490	1630	1760
	400 x 400 x 20	25.1	683	767	862	948	1030
	25	31.4	849	950	1070	1170	1270
	30	37.7	1030	1140	1270	1400	1510
	35	44.0	1230	1350	1490	1630	1760
	40	50.2	1450	1580	1730	1880	2020
	450 x 450 x 25	39.7	849	950	1070	1170	1270
	30	47.7	1030	1140	1270	1400	1510
	35	55.6	1230	1350	1490	1630	1760
	40	63.6	1450	1580	1730	1880	2020
	41	65.2	1460	1590	1740	1890	2030
160 x 160	300 x 300 x 15	10.6	564	635	714	787	855
	20	14.1	730	820	921	1010	1100
	25	17.7	816	1020	1140	1250	1360
	30	21.2	816	1020	1270	1490	1620
	350 x 350 x 20	19.2	730	820	921	1010	1100
	25	24.0	906	1020	1140	1250	1360
	30	28.8	1090	1220	1360	1490	1620
	35	33.7	1110	1390	1590	1740	1880
	400 x 400 x 20	25.1	730	820	921	1010	1100
	25	31.4	906	1020	1140	1250	1360
	30	37.7	1090	1220	1360	1490	1620
	35	44.0	1300	1430	1590	1740	1880
	40	50.2	1450	1670	1830	1990	2140
	450 x 450 x 25	39.7	906	1020	1140	1250	1360
	30	47.7	1090	1220	1360	1490	1620
	35	55.6	1300	1430	1590	1740	1880
	40	63.6	1520	1670	1830	1990	2140
	45	71.5	1720	1870	2050	2220	2380

For further information on standard details see Table G.31

For guidance on the use of tables see Explanatory notes in Table G.30

COLUMN: S275 or S355
PLATES: S275

Table G.34 *Continued*

COLUMN BASES
SQUARE HOLLOW SECTIONS

Column size *h* x *b* mm	Base plate size h_p x b_p x t_p mm	Base Plate Mass kg	Axial resistance (kN)				
			Concrete grade				
			C16/20	C20/25	C25/30	C30/37	C35/45
			Cylinder strength (N/mm^2)				
			16	20	25	30	35
180 x 180	350 x 350 x 20	19.2	824	926	1040	1140	1240
	25	24.0	1020	1150	1290	1420	1530
	30	28.8	1110	1370	1540	1690	1830
	35	33.7	1110	1390	1740	1960	2120
	400 x 400 x 20	25.1	824	926	1040	1140	1240
	25	31.4	1020	1150	1290	1420	1530
	30	37.7	1220	1370	1540	1690	1830
	35	44.0	1440	1600	1780	1960	2120
	40	50.2	1450	1810	2040	2230	2410
	450 x 450 x 25	39.7	1020	1150	1290	1420	1530
	30	47.7	1220	1370	1540	1690	1830
	35	55.6	1440	1600	1780	1960	2120
	40	63.6	1670	1840	2040	2230	2410
	45	71.5	1840	2060	2270	2470	2660
	500 x 500 x 25	49.1	1020	1150	1290	1420	1530
	30	58.9	1220	1370	1540	1690	1830
	40	78.5	1670	1840	2040	2230	2410
	50	98.1	2140	2330	2560	2770	2970
	60	118	2270	2830	3180	3420	3640
200 x 200	400 x 400 x 20	25.1	918	1030	1160	1280	1380
	25	31.4	1140	1280	1430	1580	1710
	30	37.7	1360	1520	1710	1880	2040
	35	44.0	1450	1770	1990	2180	2360
	40	50.2	1450	1810	2260	2480	2690
	450 x 450 x 25	39.7	1140	1280	1430	1580	1710
	30	47.7	1360	1520	1710	1880	2040
	35	55.6	1590	1770	1990	2180	2360
	40	63.6	1830	2030	2260	2480	2690
	45	71.5	1840	2260	2500	2740	2960
	500 x 500 x 25	49.1	1140	1280	1430	1580	1710
	30	58.9	1360	1520	1710	1880	2040
	40	78.5	1830	2030	2260	2480	2690
	50	98.1	2270	2540	2800	3050	3290
	60	118	2270	2830	3460	3730	3990

For further information on standard details see Table G.31
For guidance on the use of tables see Explanatory notes in Table G.30

COLUMN: S275 or S355
PLATES: S275

Table G.34 *Continued*

COLUMN BASES SQUARE HOLLOW SECTIONS							
Column size h x b mm	Base plate size h_p x b_p x t_p mm	Base Plate Mass kg	Axial resistance (kN)				
			Concrete grade				
			C16/20	C20/25	C25/30	C30/37	C35/45
			Cylinder strength (N/mm^2)				
			16	20	25	30	35
250 x 250	450 x 450 x 25	39.7	1430	1610	1800	1980	2150
	30	47.7	1710	1920	2150	2360	2560
	35	55.6	1830	2230	2500	2740	2970
	40	63.6	1840	2290	2840	3120	3380
	45	71.5	1840	2300	2860	3420	3720
	500 x 500 x 25	49.1	1430	1610	1800	1980	2150
	30	58.9	1710	1920	2150	2360	2560
	40	78.5	2260	2540	2840	3120	3380
	50	98.1	2270	2830	3470	3810	4120
	60	118	2270	2830	3540	4250	4930
	600 x 600 x 30	84.8	1710	1920	2150	2360	2560
	40	113	2260	2540	2840	3120	3380
	50	141	2800	3110	3470	3810	4120
	60	170	3260	3790	4190	4570	4930
	70	198	3260	4080	4870	5270	5660
260 x 260	450 x 450 x 40	113	3200	3590	4030	4430	4800
	50	141	3260	4040	4900	5390	5830
	60	170	3260	4080	5080	6050	6960
	40	154	3200	3590	4030	4430	4800
	50	192	3900	4370	4900	5390	5830
	500 x 500 x 60	231	4440	5220	5850	6430	6960
	70	269	4440	5550	6670	7320	7920
	80	308	4440	5550	6940	8330	9030
	30	84.8	2780	3120	3510	3860	4190
	40	113	3100	3790	4620	5070	5500
	600 x 600 x 50	141	3210	3940	4830	5680	6520
	60	170	3260	4050	4990	5890	6780
	70	198	3260	4080	5070	6020	6940
	40	154	3660	4110	4620	5070	5500
	50	192	4390	5010	5620	6170	6690

For further information on standard details see Table G.31
For guidance on the use of tables see Explanatory notes in Table G.30

COLUMN: S275 or S355
PLATES: S275

Table G.34 *Continued*

COLUMN BASES
SQUARE HOLLOW SECTIONS

Column size *h* x *b* mm	Base plate size h_p x b_p x t_p mm	Base Plate Mass kg	Axial resistance (kN)				
			Concrete grade				
			C16/20	C20/25	C25/30	C30/37	C35/45
			Cylinder strength (N/mm^2)				
			16	20	25	30	35
300 x 300	500 x 500 x 25	49.1	1730	1940	2180	2400	2610
	30	58.9	2060	2310	2600	2860	3100
	40	78.5	2250	2790	3430	3770	4080
	50	98.1	2270	2830	3520	4180	4830
	60	118	2270	2830	3540	4240	4930
	600 x 600 x 30	84.8	2060	2310	2600	2860	3100
	40	113	2730	3060	3430	3770	4080
	50	141	3260	3730	4180	4590	4970
	60	170	3260	4080	5000	5480	5940
	700 x 700 x 40	154	2730	3060	3430	3770	4080
	50	192	3330	3730	4180	4590	4970
	60	231	4040	4480	5000	5480	5940
	70	269	4440	5180	5730	6260	6770
	80	308	4440	5550	6630	7200	7740
350 x 350	500 x 500 x 25	49.1	1980	2290	2570	2840	3080
	30	58.9	2070	2520	3060	3370	3650
	40	78.5	2200	2690	3290	3860	4420
	50	98.1	2260	2790	3430	4040	4640
	60	118	2270	2830	3520	4180	4810
	600 x 600 x 30	84.8	2420	2720	3060	3370	3650
	40	113	3200	3590	4030	4430	4800
	50	141	3260	4040	4900	5390	5830
	60	170	3260	4080	5080	6050	6960
	700 x 700 x 40	154	3200	3590	4030	4430	4800
	50	192	3900	4370	4900	5390	5830
	60	231	4440	5220	5850	6430	6960
	70	269	4440	5550	6670	7320	7920
	80	308	4440	5550	6940	8330	9030
400 x 400	600 x 600 x 30	84.8	2780	3120	3510	3860	4190
	40	113	3100	3790	4620	5070	5500
	50	141	3210	3940	4830	5680	6520
	60	170	3260	4050	4990	5890	6780
	70	198	3260	4080	5070	6020	6940
	700 x 700 x 40	154	3660	4110	4620	5070	5500
	50	192	4390	5010	5620	6170	6690
	60	231	4440	5520	6710	7370	7970
	70	269	4440	5550	6910	8230	9080
	80	308	4440	5550	6940	8310	9650
	800 x 800 x 40	201	3660	4110	4620	5070	5500
	50	251	4470	5010	5620	6170	6690
	60	301	5340	5980	6710	7370	7970
	70	352	5800	6820	7640	8390	9080
	80	402	5800	7250	8710	9560	10300
	90	452	5800	7250	9070	10500	11400

For further information on standard details see Table G.31
For guidance on the use of tables see Explanatory notes in Table G.30

COLUMN: S275 or S355
Table G.35 **PLATES: S275**

COLUMN BASES
RECTANGULAR HOLLOW SECTIONS

Column size $h \times b$ mm	Base plate size $h_p \times b_p \times t_p$ mm	Base Plate Mass kg	Axial resistance (kN)				
			Concrete grade				
			C16/20	C20/25	C25/30	C30/37	C35/45
			Cylinder strength (N/mm^2)				
			16	20	25	30	35
200 x 100	400 x 300 x 20	19	662	748	849	945	1024
	25	24	827	922	1035	1142	1245
	30	28	1009	1114	1238	1356	1468
	35	33	1088	1324	1459	1587	1708
	40	38	1088	1360	1698	1835	1967
	450 x 350 x 25	31	827	922	1035	1142	1245
	30	37	1009	1114	1238	1356	1468
	35	43	1208	1324	1459	1587	1708
	40	49	1426	1551	1698	1835	1967
	45	56	1428	1753	1909	2055	2194
200 x 120	400 x 300 x 20	19	721	818	921	1014	1100
	25	24	891	999	1127	1249	1359
	30	28	1053	1197	1337	1470	1598
	35	33	1088	1344	1565	1709	1846
	40	38	1088	1360	1699	1950	2113
	450 x 350 x 25	31	891	999	1127	1249	1359
	30	37	1079	1197	1337	1470	1598
	35	43	1284	1414	1565	1709	1846
	40	49	1427	1647	1811	1965	2113
	45	56	1428	1771	2028	2191	2347
200 x 150	400 x 300 x 20	19	801	899	1010	1112	1206
	25	24	968	1114	1251	1375	1491
	30	28	1053	1250	1486	1639	1775
	35	33	1088	1344	1593	1834	2053
	40	38	1088	1360	1699	1950	2195
	450 x 350 x 35	43	1328	1548	1725	1892	2053
	40	49	1427	1679	1981	2160	2332
	45	56	1428	1771	2085	2387	2577
220 x 120	400 x 300 x 20	19	766	871	981	1079	1170
	25	24	941	1058	1196	1328	1447
	30	28	1088	1263	1414	1557	1695
	35	33	1088	1360	1649	1804	1953
	40	38	1088	1360	1700	2040	2227
	450 x 350 x 25	31	941	1058	1196	1328	1447
	30	37	1135	1263	1414	1557	1695
	35	43	1346	1485	1649	1804	1953
	40	49	1428	1725	1902	2068	2227
	45	56	1428	1785	2124	2301	2469
	500 x 400 x 25	39	941	1058	1196	1328	1447
	30	47	1135	1263	1414	1557	1695
	35	55	1346	1485	1649	1804	1953
	40	63	1574	1725	1902	2068	2227
	45	71	1777	1937	2124	2301	2469

For further information on standard details see Table G.31
For guidance on the use of tables see Explanatory notes in Table G.30

COLUMN: S275 or S355
PLATES: S275

Table G.35 *Continued*

COLUMN BASES
RECTANGULAR HOLLOW SECTIONS

Column size h x b mm	Base plate size h_p x b_p x t_p mm	Base Plate Mass kg	Axial resistance (kN)				
			Concrete grade				
			C16/20	C20/25	C25/30	C30/37	C35/45
			Cylinder strength (N/mm^2)				
			16	20	25	30	35
250 x 100	450 x 300 x 25	26	943	1058	1194	1324	1449
	30	32	1139	1266	1415	1557	1693
	35	37	1224	1491	1654	1807	1954
	40	42	1224	1530	1910	2075	2233
	45	48	1224	1530	1913	2295	2478
	500 x 350 x 25	34	943	1058	1194	1324	1449
	30	41	1139	1266	1415	1557	1693
	40	55	1584	1735	1910	2075	2233
	50	69	1587	1983	2419	2605	2783
	60	82	1587	1983	2479	2975	3444
250 x 150	450 x 350 x 25	31	1127	1278	1435	1578	1710
	30	37	1337	1502	1698	1880	2036
	35	43	1428	1744	1955	2155	2349
	40	49	1428	1785	2229	2443	2648
	45	56	1428	1785	2231	2678	2910
	500 x 400 x 25	39	1127	1278	1435	1578	1710
	30	47	1337	1502	1698	1880	2036
	40	63	1811	2003	2229	2443	2648
	50	79	1813	2267	2769	3008	3236
	60	94	1813	2267	2833	3400	3939
	600 x 500 x 30	71	1337	1502	1698	1880	2036
	40	94	1811	2003	2229	2443	2648
	50	118	2300	2517	2769	3008	3236
	60	141	2720	3139	3421	3686	3939
260 x 140	450 x 300 x 25	26	1117	1266	1435	1578	1710
	30	32	1217	1454	1683	1867	2036
	35	37	1224	1530	1848	2137	2327
	40	42	1224	1530	1913	2256	2552
	45	48	1224	1530	1913	2295	2656
	500 x 350 x 25	34	1117	1266	1435	1578	1710
	30	41	1327	1490	1683	1867	2036
	35	48	1518	1731	1939	2137	2327
	40	55	1587	1917	2213	2424	2627
	45	62	1587	1983	2382	2676	2889
	600 x 400 x 30	57	1327	1490	1683	1867	2036
	40	75	1801	1990	2213	2424	2627
	50	94	2053	2420	2754	2989	3214
	60	113	2176	2668	3139	3593	3917
	70	132	2176	2720	3378	3855	4316

For further information on standard details see Table G.31
For guidance on the use of tables see Explanatory notes in Table G.30

COLUMN: S275 or S355
PLATES: S275

Table G.35 *Continued*

COLUMN BASES
RECTANGULAR HOLLOW SECTIONS

Column size $h \times b$ mm	Base plate size $h_p \times b_p \times t_p$ mm	Base Plate Mass kg	Axial resistance (kN)				
			Concrete grade				
			C16/20	C20/25	C25/30	C30/37	C35/45
			Cylinder strength (N/mm^2)				
			16	20	25	30	35
300 x 100	500 x 300 x 25	29	1059	1194	1353	1506	1653
	30	35	1269	1417	1592	1758	1918
	40	47	1360	1700	2122	2315	2500
	50	59	1360	1700	2125	2550	2975
	60	71	1360	1700	2125	2550	2975
	600 x 400 x 30	57	1269	1417	1592	1758	1918
	40	75	1743	1918	2122	2315	2500
	50	94	2176	2432	2663	2881	3087
	60	113	2176	2720	3315	3559	3790
	70	132	2176	2720	3400	4080	4449
300 x 150	500 x 300 x 25	29	1240	1443	1634	1800	1954
	30	35	1325	1590	1911	2129	2318
	40	47	1360	1700	2124	2460	2790
	50	59	1360	1700	2125	2550	2975
	60	71	1360	1700	2125	2550	2975
	500 x 350 x 25	34	1266	1443	1634	1800	1954
	30	41	1490	1682	1911	2129	2318
	35	48	1587	1940	2185	2419	2644
	40	55	1587	1983	2476	2725	2965
	45	62	1587	1983	2479	2975	3244
	600 x 400 x 30	57	1490	1682	1911	2129	2318
	40	75	1992	2214	2476	2725	2965
	50	94	2176	2602	3049	3326	3590
	60	113	2176	2720	3366	3865	4334
	70	132	2176	2720	3400	4080	4634
	700 x 500 x 30	82	1490	1682	1911	2129	2318
	40	110	1992	2214	2476	2725	2965
	50	137	2507	2757	3049	3326	3590
	60	165	3026	3410	3735	4042	4334
	70	192	3173	3830	4381	4713	5029
300 x 200	500 x 400 x 25	39	1431	1606	1803	1982	2148
	30	47	1708	1916	2149	2362	2558
	40	63	1813	2267	2830	3121	3379
	50	79	1813	2267	2833	3400	3967
	60	94	1813	2267	2833	3400	3967
	600 x 500 x 30	71	1708	1916	2149	2362	2558
	40	94	2241	2511	2830	3121	3379
	50	118	2720	3082	3435	3771	4093
	60	141	2720	3400	4156	4525	4878
	70	165	2720	3400	4250	5100	5608
	700 x 600 x 40	132	2241	2511	2830	3121	3379
	50	165	2782	3082	3435	3771	4093
	60	198	3434	3766	4156	4525	4878
	70	231	3808	4410	4832	5229	5608

For further information on standard details see Table G.31
For guidance on the use of tables see Explanatory notes in Table G.30

COLUMN: S275 or S355
PLATES: S275

Table G.35 *Continued*

COLUMN BASES
RECTANGULAR HOLLOW SECTIONS

Column size $h \times b$ mm	Base plate size $h_p \times b_p \times t_p$ mm	Base Plate Mass kg	Axial resistance (kN)				
			Concrete grade				
			C16/20	C20/25	C25/30	C30/37	C35/45
			Cylinder strength (N/mm^2)				
			16	20	25	30	35
300 x 250	500 x 450 x 25	44	1577	1770	1987	2184	2367
	30	53	1883	2111	2369	2603	2819
	35	62	2026	2453	2751	3021	3271
	40	71	2040	2537	3133	3440	3723
	45	79	2040	2550	3168	3767	4098
	600 x 500 x 30	71	1883	2111	2369	2603	2819
	40	94	2491	2795	3133	3440	3723
	50	118	2720	3252	3821	4197	4541
	60	141	2720	3400	4207	4831	5421
	70	165	2720	3400	4250	5100	5792
	700 x 600 x 30	99	1883	2111	2369	2603	2819
	40	132	2491	2795	3133	3440	3723
	50	165	3057	3407	3821	4197	4541
	60	198	3631	4122	4577	5008	5421
	70	231	3808	4596	5282	5745	6187
340 x 100	500 x 300 x 25	29	1152	1302	1481	1651	1816
	30	35	1302	1516	1734	1919	2097
	40	47	1360	1700	2123	2400	2666
	50	59	1360	1700	2125	2550	2975
	60	71	1360	1700	2125	2550	2975
	550 x 300 x 25	32	1152	1302	1481	1651	1816
	30	39	1373	1539	1734	1919	2097
	35	45	1496	1793	2004	2204	2396
	40	52	1496	1870	2292	2507	2713
	45	58	1496	1870	2338	2760	2989
	600 x 400 x 30	57	1373	1539	1734	1919	2097
	40	75	1870	2064	2292	2507	2713
	50	94	2176	2542	2859	3101	3331
	60	113	2176	2720	3348	3757	4066
	70	132	2176	2720	3400	4080	4571
	700 x 500 x 30	82	1373	1539	1734	1919	2097
	40	110	1870	2064	2292	2507	2713
	50	137	2380	2601	2859	3101	3331
	60	165	2967	3248	3538	3809	4066
	70	192	3173	3775	4178	4474	4754

For further information on standard details see Table G.31

For guidance on the use of tables see Explanatory notes in Table G.30

COLUMN: S275 or S355
PLATES: S275

Table G.35 *Continued*

COLUMN BASES
RECTANGULAR HOLLOW SECTIONS

Column size $h \times b$ mm	Base plate size $h_p \times b_p \times t_p$ mm	Base Plate Mass kg	Axial resistance (kN) Concrete grade C16/20	C20/25	C25/30	C30/37	C35/45
			Cylinder strength (N/mm^2) 16	20	25	30	35
350 x 150	550 x 350 x 25	38	1404	1606	1803	1982	2148
	30	45	1643	1862	2123	2362	2558
	35	53	1745	2135	2415	2682	2939
	40	60	1745	2182	2724	3008	3281
	45	68	1745	2182	2727	3273	3577
	600 x 400 x 30	57	1643	1862	2123	2362	2558
	40	75	2173	2426	2724	3008	3281
	50	94	2176	2720	3329	3643	3944
	60	113	2176	2720	3400	4080	4729
	70	132	2176	2720	3400	4080	4760
	700 x 500 x 30	82	1643	1862	2123	2362	2558
	40	110	2173	2426	2724	3008	3281
	50	137	2714	2997	3329	3643	3944
	60	165	3173	3681	4050	4397	4729
	70	192	3173	3967	4725	5101	5459
350 x 250	500 x 300 x 25	29	1226	1445	1679	1897	2104
	30	35	1292	1565	1892	2139	2373
	40	47	1360	1682	2052	2405	2746
	50	59	1360	1700	2125	2523	2896
	60	71	1360	1700	2125	2550	2975
	550 x 450 x 25	49	1729	1942	2183	2402	2605
	30	58	2062	2314	2598	2857	3097
	35	68	2224	2686	3014	3313	3589
	40	78	2244	2787	3430	3768	4081
	45	87	2244	2805	3478	4124	4488
	600 x 500 x 30	71	2062	2314	2598	2857	3097
	40	94	2717	3058	3430	3768	4081
	50	118	2720	3400	4172	4592	4970
	60	141	2720	3400	4250	5100	5916
	70	165	2720	3400	4250	5100	5950
	700 x 600 x 30	99	2062	2314	2598	2857	3097
	40	132	2717	3058	3430	3768	4081
	50	165	3309	3704	4172	4592	4970
	60	198	3808	4450	4962	5448	5916
	70	231	3808	4760	5697	6218	6717

For further information on standard details see Table G.31
For guidance on the use of tables see Explanatory notes in Table G.30

COLUMN: S275 or S355
PLATES: S275

Table G.35 *Continued*

COLUMN BASES
RECTANGULAR HOLLOW SECTIONS

Column size $h \times b$ mm	Base plate size $h_p \times b_p \times t_p$ mm	Base Plate Mass kg	Axial resistance (kN) Concrete grade C16/20	C20/25	C25/30	C30/37	C35/45
			Cylinder strength (N/mm^2) 16	20	25	30	35
400 x 150	600 x 350 x 25	41	1543	1771	1997	2198	2384
	30	49	1796	2042	2335	2614	2834
	40	66	1904	2380	2972	3290	3597
	50	82	1904	2380	2975	3570	4165
	60	99	1904	2380	2975	3570	4165
	700 x 500 x 30	82	1796	2042	2335	2614	2834
	40	110	2355	2638	2972	3290	3597
	50	137	2895	3237	3609	3961	4298
	60	165	3173	3797	4364	4753	5124
	70	192	3173	3967	4821	5437	5890
400 x 200	600 x 400 x 30	57	2062	2314	2598	2857	3097
	40	75	2176	2720	3397	3768	4081
	50	94	2176	2720	3400	4080	4760
	60	113	2176	2720	3400	4080	4760
	70	132	2176	2720	3400	4080	4760
	700 x 500 x 40	110	2649	2991	3397	3768	4081
	50	137	3173	3619	4065	4491	4900
	60	165	3173	3967	4856	5321	5767
	70	192	3173	3967	4958	5950	6568
	80	220	3173	3967	4958	5950	6942
	800 x 600 x 40	151	2649	2991	3397	3768	4081
	50	188	3241	3619	4065	4491	4900
	60	226	3948	4365	4856	5321	5767
	70	264	4352	5063	5591	6091	6568
	80	301	4352	5440	6489	7028	7541
400 x 300	600 x 500 x 25	59	2035	2288	2575	2837	3080
	30	71	2422	2721	3059	3367	3653
	40	94	2678	3290	4026	4427	4798
	50	118	2720	3387	4173	4935	5680
	60	141	2720	3400	4250	5069	5854
	700 x 600 x 30	99	2422	2721	3059	3367	3653
	40	132	3196	3587	4026	4427	4798
	50	165	3808	4370	4902	5386	5834
	60	198	3808	4760	5839	6426	6957
	70	231	3808	4760	5950	7140	7925
	800 x 700 x 30	132	2422	2721	3059	3367	3653
	40	176	3196	3587	4026	4427	4798
	50	220	3881	4370	4902	5386	5834
	60	264	4644	5191	5839	6426	6957
	70	308	5077	5942	6634	7292	7925

For further information on standard details see Table G.31
For guidance on the use of tables see Explanatory notes in Table G.30

COLUMN: S275 or S355
PLATES: S275

Table G.35 *Continued*

COLUMN BASES
RECTANGULAR HOLLOW SECTIONS

Column size $h \times b$ mm	Base plate size $h_p \times b_p \times t_p$ mm	Base Plate Mass kg	Axial resistance (kN) Concrete grade C16/20 Cylinder strength (N/mm^2) 16	C20/25 20	C25/30 25	C30/37 30	C35/45 35
450 x 250	650 x 450 x 30	69	2422	2721	3059	3367	3653
	40	92	2652	3290	4026	4427	4798
	50	115	2652	3315	4144	4935	5680
	60	138	2652	3315	4144	4973	5801
	70	161	2652	3315	4144	4973	5801
	700 x 500 x 30	82	2422	2721	3059	3367	3653
	40	110	3170	3587	4026	4427	4798
	50	137	3173	3967	4873	5386	5834
	60	165	3173	3967	4958	5950	6904
	70	192	3173	3967	4958	5950	6942
	800 x 600 x 30	113	2422	2721	3059	3367	3653
	40	151	3170	3587	4026	4427	4798
	50	188	3813	4298	4873	5386	5834
	60	226	4352	5106	5733	6330	6904
	70	264	4352	5440	6528	7165	7776
500 x 200	700 x 400 x 30	66	2413	2721	3059	3367	3653
	40	88	2539	3173	3963	4427	4798
	50	110	2539	3173	3967	4760	5553
	60	132	2539	3173	3967	4760	5553
	70	154	2539	3173	3967	4760	5553
	800 x 500 x 40	126	3057	3471	3963	4427	4798
	50	157	3627	4156	4696	5211	5707
	60	188	3627	4533	5556	6117	6656
	70	220	3627	4533	5667	6800	7528
	80	251	3627	4533	5667	6800	7933
	900 x 600 x 40	170	3057	3471	3963	4427	4798
	50	212	3700	4156	4696	5211	5707
	60	254	4463	4964	5556	6117	6656
	70	297	4896	5715	6351	6952	7528
	80	339	4896	6120	7317	7964	8582

For further information on standard details see Table G.31

For guidance on the use of tables see Explanatory notes in Table G.30

COLUMN: S275 or S355
PLATES: S275

Table G.35 *Continued*

COLUMN BASES
RECTANGULAR HOLLOW SECTIONS

Column size $h \times b$ mm	Base plate size $h_p \times b_p \times t_p$ mm	Base Plate Mass kg	Axial resistance (kN)				
			Concrete grade				
			C16/20	C20/25	C25/30	C30/37	C35/45
			Cylinder strength (N/mm^2)				
			16	20	25	30	35
500 x 300	700 x 500 x 30	82	2776	3119	3506	3859	4187
	40	110	3100	3787	4615	5074	5499
	50	137	3173	3943	4826	5683	6519
	60	165	3173	3967	4958	5893	6775
	70	192	3173	3967	4958	5950	6942
	800 x 600 x 40	151	3664	4111	4615	5074	5499
	50	188	4352	5009	5619	6174	6687
	60	226	4352	5440	6680	7366	7974
	70	264	4352	5440	6800	8160	9083
	80	301	4352	5440	6800	8160	9520
	900 x 700 x 40	198	3664	4111	4615	5074	5499
	50	247	4431	5009	5619	6174	6687
	60	297	5249	5904	6680	7366	7974
	70	346	5712	6708	7535	8324	9083
	80	396	5712	7140	8569	9410	10217

For further information on standard details see Table G.31

For guidance on the use of tables see Explanatory notes in Table G.30

Table G.36 Material Strengths

The following extracts from BS EN 10210-1[(6)] and BS EN 10025[(5)] have been included for the convenience of the connection designer:

Design strengths for hot rolled sections, plates and hot rolled hollow sections				
Steel Grade	**Thickness less than or equal to (mm)**		**Design yield strength, f_y (N/mm²)**	**Design ultimate strength, f_u (N/mm²)**
	BS EN 10025	BS EN 10210		
S275	16		275	410
	40		265	
	63		255	
	80		245	
	100		235	
	150	120	225	400
S355	16		355	470
	40		345	
	63		335	
	80		325	
	100		315	
	150	120	295	450

Values taken from BS EN 10025[(5)] Table 7 for hot rolled sections and plates and BS EN 10210-1[(6)] Table A.3 for hot rolled hollow sections. In BS EN 10025 and BS EN 10210-1 f_y and f_u are designated R_{eH} and R_m respectively.

Table G.37 Weld Strengths

Fillet weld design shear strength, $f_{vw,d}$ (N/mm²)		
Steel Grade	Longitudinal loading	Transverse loading
S275	223	273
S355	241	295

Weld strengths have been calculated in accordance with BS EN 1993-1-8, clause 4.5.3.3(3) and the UK National Annex.

The specified yield strength, ultimate tensile strength, elongation at failure and minimum Charpy V-notch energy value of the filler material should be equivalent to, or better than that specified for the parent material.

It is generally safe to use electrodes that are overmatched with regard to the steel grades being used.

The values presented above assume that the elements being welded are at 90°.

Table G.38 Bolt Strengths

	Bolt property class		
	4.6	**8.8**	**10.9**
Design yield strength, f_{yb} (N/mm²)	240	640	900
Design ultimate strength, f_{ub} (N/mm²)	400	800	1000

Values taken from BS EN 1993-1-8:2005, Table 3.1.

BS EN 1993-1-8:2005
BS EN ISO 4016
BS EN ISO 4018

Table G.40 Non preloaded hexagon head bolts, property class 4.6, in S275

Diameter of Bolt	Tensile Stress Area	Tension Resistance	Shear Resistance		Bolts in tension
			Single Shear	Double Shear	Min thickness for punching shear
d	A_s	$F_{t,Rd}$	$F_{v,Rd}$	$2 \times F_{v,Rd}$	t_{min}
mm	mm^2	kN	kN	kN	mm
12	84.3	24.3	16.2	32.4	2.1
16	157	45.2	30.1	60.3	3.2
20	245	70.6	47.0	94.1	3.9
24	353	102	67.8	136	4.7
30	561	162	108	215	5.8

See notes below

Diameter of Bolt	Minimum				Bearing Resistance (kN)										
	Edge distance	End distance	Pitch	Gauge	Thickness in mm of ply, t										
d	e_2	e_1	p_1	p_2											
mm	mm	mm	mm	mm	5	6	7	8	9	10	12	15	20	25	30
12	20	25	35	40	26.4	31.7	*37*	*42.2*	*47.5*	*52.8*	*63.4*	*79.2*	*106*	*132*	*158*
16	25	35	50	50	37.2	44.7	52.1	59.6	*67*	*74.5*	*89.3*	*112*	*149*	*186*	*223*
20	30	40	60	60	**42.1**	50.5	58.9	67.4	75.8	84.2	*101*	*126*	*168*	*211*	*253*
24	35	50	70	70	**52.2**	**62.6**	73.1	83.5	94	104	125	*157*	*209*	*261*	*313*
30	45	60	85	90	**63.2**	**75.8**	**88.4**	**101**	114	126	152	189	*253*	*316*	*379*

See clause 3.7(1) of BS EN 1993-1-8: 2005 for calculation of the design resistance of a group of fasteners.
Values of bearing resistance in **bold** are less than the single shear resistance of the bolt.
Values of bearing resistance in *italic* are greater than the double shear resistance of the bolt.
Bearing values assume standard clearance holes.
If oversize or short slotted holes are used, bearing values should be multiplied by 0.8.
If long slotted or kidney shaped holes are used, bearing values should be multiplied by 0.6.
In single lap joints with only one bolt row, the design bearing resistance for each bolt should be limited to $1.5 f_u d\ t/\gamma_{M2}$.

BS EN 1993-1-8:2005
BS EN ISO 4014
BS EN ISO 4017

Table G.41 Non preloaded hexagon head bolts, property class 8.8, in S275

Diameter of Bolt	Tensile Stress Area	Tension Resistance	Shear Resistance		Bolts in tension
			Single Shear	Double Shear	Min thickness for punching shear
d	A_s	$F_{t,Rd}$	$F_{v,Rd}$	$2 \times F_{v,Rd}$	t_{min}
mm	mm²	kN	kN	kN	mm
12	84.3	48.6	27.5	55.0	4.3
16	157	90.4	60.3	121	6.3
20	245	141	94.1	188	7.8
24	353	203	136	271	9.4
30	561	323	215	431	11.6

See notes below

Diameter of Bolt	Minimum Edge distance	Minimum End distance	Minimum Pitch	Minimum Gauge	Bearing Resistance (kN), Thickness in mm of ply, t										
d	e_2	e_1	p_1	p_2											
mm	mm	mm	mm	mm	5	6	7	8	9	10	12	15	20	25	30
12	20	25	35	40	**26.4**	31.7	37	42.2	47.5	52.8	*63.4*	*79.2*	*106*	*132*	*158*
16	25	35	50	50	**37.2**	**44.7**	**52.1**	**59.6**	67	74.5	89.3	112	*149*	*186*	*223*
20	30	40	60	60	**42.1**	**50.5**	**58.9**	**67.4**	**75.8**	**84.2**	101	126	168	*211*	*253*
24	35	50	70	70	**52.2**	**62.6**	**73.1**	**83.5**	**94**	**104**	**125**	157	209	261	*313*
30	45	60	85	90	**63.2**	**75.8**	**88.4**	**101**	**114**	**126**	**152**	**189**	253	316	379

See notes below

Diameter of Bolt	Minimum Edge distance	Minimum End distance	Minimum Pitch	Minimum Gauge	Bearing Resistance (kN), Thickness in mm of ply, t										
d	e_2	e_1	p_1	p_2											
mm	mm	mm	mm	mm	5	6	7	8	9	10	12	15	20	25	30
12	25	40	50	45	46.3	*55.5*	*64.8*	*74*	*83.3*	*92.5*	*111*	*139*	*185*	*231*	*278*
16	30	50	65	55	60.7	72.9	85	97.2	109	121	*146*	*182*	*243*	*304*	*364*
20	35	60	80	70	**74.5**	**89.5**	104	119	134	149	179	*224*	*298*	*373*	*447*
24	40	75	95	80	**94.6**	**114**	**132**	151	170	189	227	*284*	*378*	*473*	*568*
30	50	90	115	100	**112**	**134**	**157**	**179**	**201**	224	268	335	*447*	*559*	*671*

For M12 bolts the design shear resistance $F_{v,Rd}$ has been calculated as 0.85 times the value given in BS EN 1993-1-8, Table 3.4 (§3.6.1(5)).
See clause 3.7(1) of BS EN 1993-1-8: 2005 for calculation of the design resistance of a group of fasteners.
Values of bearing resistance in **bold** are less than the single shear resistance of the bolt.
Values of bearing resistance in *italic* are greater than the double shear resistance of the bolt.
Bearing values assume standard clearance holes.
If oversize or short slotted holes are used, bearing values should be multiplied by 0.8.
If long slotted or kidney shaped holes are used, bearing values should be multiplied by 0.6.
In single lap joints with only one bolt row, the design bearing resistance for each bolt should be limited to $1.5 f_u d\ t / \gamma_{M2}$.

BS EN 1993-1-8:2005
BS EN ISO 4014
BS EN ISO 4017

Table G.42 Non preloaded hexagon head bolts, property class 10.9, in S275

Diameter of Bolt	Tensile Stress Area	Tension Resistance	Shear Resistance		Bolts in tension
			Single Shear	Double Shear	Min thickness for punching shear
d	A_s	$F_{t,Rd}$	$F_{v,Rd}$	$2 \times F_{v,Rd}$	t_{min}
mm	mm^2	kN	kN	kN	mm
12	84.3	60.7	28.7	57.3	5.3
16	157	113	62.8	126	7.9
20	245	176	98.0	196	9.8
24	353	254	141	282	11.7
30	561	404	224	449	14.5

See notes below

Diameter of Bolt	Minimum				Bearing Resistance (kN)										
	Edge distance	End distance	Pitch	Gauge	Thickness in mm of ply, t										
d	e_2	e_1	p_1	p_2											
mm	mm	mm	mm	mm	5	6	7	8	9	10	12	15	20	25	30
12	20	25	35	40	**26.4**	31.7	37	42.2	47.5	52.8	*63.4*	*79.2*	*106*	*132*	*158*
16	25	35	50	50	**37.2**	**44.7**	**52.1**	**59.6**	67	74.5	89.3	112	*149*	*186*	*223*
20	30	40	60	60	**42.1**	**50.5**	**58.9**	**67.4**	**75.8**	**84.2**	101	126	168	*211*	*253*
24	35	50	70	70	**52.2**	**62.6**	**73.1**	**83.5**	**94**	**104**	**125**	157	209	261	*313*
30	45	60	85	90	**63.2**	**75.8**	**88.4**	**101**	**114**	**126**	**152**	**189**	253	316	379

See notes below

Diameter of Bolt	Minimum				Bearing Resistance (kN)										
	Edge distance	End distance	Pitch	Gauge	Thickness in mm of ply, t										
d	e_2	e_1	p_1	p_2											
mm	mm	mm	mm	mm	5	6	7	8	9	10	12	15	20	25	30
12	25	40	50	45	46.3	55.5	*64.8*	*74*	*83.3*	*92.5*	*111*	*139*	*185*	*231*	*278*
16	30	50	65	55	**60.7**	72.9	85	97.2	109	121	*146*	*182*	*243*	*304*	*364*
20	35	60	80	70	**74.5**	**89.5**	104	119	134	149	179	*224*	*298*	*373*	*447*
24	40	75	95	80	**94.6**	**114**	**132**	151	170	189	227	*284*	*378*	*473*	*568*
30	50	90	115	100	**112**	**134**	**157**	**179**	**201**	224	268	335	447	*559*	*671*

For M12 bolts the design shear resistance $F_{v,Rd}$ has been calculated as 0.85 times the value given in BS EN 1993-1-8, Table 3.4 (§3.6.1(5)).

See clause 3.7(1) of BS EN 1993-1-8: 2005 for calculation of the design resistance of a group of fasteners.

Values of bearing resistance in **bold** are less than the single shear resistance of the bolt.

Values of bearing resistance in *italic* are greater than the double shear resistance of the bolt.

Bearing values assume standard clearance holes.

If oversize or short slotted holes are used, bearing values should be multiplied by 0.8.

If long slotted or kidney shaped holes are used, bearing values should be multiplied by 0.6.

In single lap joints with only one bolt row, the design bearing resistance for each bolt should be limited to $1.5 f_u d\ t / \gamma_{M2}$.

BS EN 1993-1-8:2005
BS EN 14399:2005
EN 1090:2008

S275

Table G.43 Preloaded hexagon head bolts in category B shear connections, property class 8.8, in S275

Diameter of Bolt	Tensile Stress Area	Shear Resistance		Slip resistance, $F_{s,Rd,ser}$							
		Single Shear	Double Shear	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
d	A_s	$F_{v,Rd}$	2 x $F_{v,Rd}$								
mm	mm^2	kN	kN	kN	kN	kN	kN	kN	kN	kN	kN
12	84.3	27.5	55.0	8.6	17.2	12.9	25.7	17.2	34.3	21.5	42.9
16	157	60.3	121	16.0	32.0	24.0	48.0	32.0	63.9	40.0	79.9
20	245	94.1	188	24.9	49.9	37.4	74.8	49.9	100	62.4	125
24	353	136	271	35.9	71.9	53.9	108	71.9	144	89.9	180
30	561	215	431	57.1	114	85.7	171	114	228	143	286

Bearing resitances may be taken from the tables for non-preloaded bolts.
For M12 bolts the design shear resistance $F_{v,Rd}$ has been calculated as 0.85 times the value given in BS EN 1993-1-8, Table 3.4 (§3.6.1(5)).
See clause 3.7(1) of BS EN 1993-1-8: 2005 for calculation of the design resistance of a group of fasteners.
The shear resistances are ULS values.
The slip resistances are SLS values.
Values have been calculated assuming k_s=1. See BS EN 1993-1-8, section 3.9 for other values of k_s.

Table G.44 Preloaded hexagon head bolts in category E tension connections, property class 8.8, in S275

Diameter of Bolt	Tensile Stress Area	Tension Resistance	Min thickness for punching shear
d	A_s	$F_{t,Rd}$	t_{min}
mm	mm^2	kN	mm
12	84.3	48.6	3.7
16	157	90.4	5.4
20	245	141	7.1
24	353	203	8.0
30	561	323	10.5

The minimum thickness is such that the design punching shear resistance $B_{p,Rd}$ is equal to the design tension resistance, $F_{t,Rd}$.

Table G.45 Preloaded hexagon head bolts in category B shear connections, property class 10.9, in S275

Diameter of Bolt	Tensile Stress Area	Shear Resistance		Slip resistance, $F_{s,Rd,ser}$							
				$\mu = 0.2$		$\mu = 0.3$		$\mu = 0.4$		$\mu = 0.5$	
		Single Shear	Double Shear	Single Shear	Double Shear	Single Shear	Double Shear	Single Shear	Double Shear	Single Shear	Double Shear
d	A_s	$F_{v,Rd}$	2 x $F_{v,Rd}$								
mm	mm^2	kN	kN	kN	kN	kN	kN	kN	kN	kN	kN
16	157	62.8	126	20.0	40.0	30.0	59.9	40.0	79.9	50.0	99.9
20	245	98.0	196	31.2	62.4	46.8	93.5	62.4	125	78.0	156
24	353	141	282	44.9	89.9	67.4	135	89.9	180	112	225
30	561	224	449	71.4	143	107	214	143	286	179	357

Bearing resitances should be taken from the tables for non-preloaded bolts.

See clause 3.7(1) of BS EN 1993-1-8: 2005 for calculation of the design resistance of a group of fasteners.

The shear resistances are ULS values.

The slip resistances are SLS values.

Values have been calculated assuming k_s=1. See BS EN 1993-1-8, section 3.9 for other values of k_s.

Table G.46 Preloaded hexagon head bolts in category E tension connections, property class 10.9, in S275

Diameter of Bolt	Tensile Stress Area	Tension Resistance	Min thickness for punching shear
d	A_s	$F_{t,Rd}$	t_{min}
mm	mm^2	kN	mm
16	157	113.0	6.8
20	245	176	8.9
24	353	254	10.0
30	561	404	13.1

The minimum thickness is such that the design punching shear resistance $B_{p,Rd}$ is equal to the design tension resistance, $F_{t,Rd}$.

BS EN 1993-1-8:2005
BS EN 14399:2005
EN 1090:2008

S275

Table G.47 Preloaded hexagon head bolts in category C shear connections, property class 8.8, in S275

Diameter of Bolt	Tensile Stress Area	Slip resistance, $F_{s,Rd}$							
		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
d	A_s								
mm	mm^2	kN	kN	kN	kN	kN	kN	kN	kN
12	84.3	7.55	15.1	11.3	22.7	15.1	30.2	18.9	37.8
16	157	14.1	28.1	21.1	42.2	28.1	56.3	35.2	70.3
20	245	22.0	43.9	32.9	65.9	43.9	87.8	54.9	110
24	353	31.6	63.3	47.4	94.9	63.3	127	79.1	158
30	561	50.3	101	75.4	151	101	201	126	251

Bearing resistances may be taken from the tables for non-preloaded bolts.

BS EN 1993-1-8:2005
BS EN 14399:2005
EN 1090:2008

Table G.48 Preloaded hexagon head bolts in category C shear connections, property class 10.9, in S275

Diameter of Bolt	Tensile Stress Area	Slip resistance, $F_{s,Rd}$							
		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
d	A_s								
mm	mm^2	kN	kN	kN	kN	kN	kN	kN	kN
16	157	17.6	35.2	26.4	52.8	35.2	70.3	44.0	87.9
20	245	27.4	54.9	41.2	82.3	54.9	110	68.6	137
24	353	39.5	79.1	59.3	119	79.1	158	98.8	198
30	561	62.8	126	94.2	188	126	251	157	314

Bearing resistances may be taken from the tables for non-preloaded bolts.

BS EN 1993-1-8:2005
BS EN ISO 4016
BS EN ISO 4018

S355

Table G.49 Non Preloaded hexagon head bolts, property class 4.6, in S355

Diameter of Bolt	Tensile Stress Area	Tension Resistance	Shear Resistance		Bolts in tension
			Single Shear	Double Shear	Min thickness for punching shear
d	A_s	$F_{t,Rd}$	$F_{v,Rd}$	$2 \times F_{v,Rd}$	t_{min}
mm	mm^2	kN	kN	kN	mm
12	84.3	24.3	16.2	32.4	1.8
16	157	45.2	30.1	60.3	2.7
20	245	70.6	47.0	94.1	3.4
24	353	102	67.8	136	4.1
30	561	162	108	215	5.1

See notes below

Diameter of Bolt	Minimum				Bearing Resistance (kN)										
	Edge distance	End distance	Pitch	Gauge	Thickness in mm of ply, t										
d	e_2	e_1	p_1	p_2											
mm	mm	mm	mm	mm	5	6	7	8	9	10	12	15	20	25	30
12	20	25	35	40	30.3	*36.3*	*42.4*	*48.4*	*54.5*	*60.5*	*72.6*	*90.8*	*121*	*151*	*182*
16	25	35	50	50	42.7	51.2	59.7	*68.3*	*76.8*	*85.4*	*102*	*128*	*171*	*213*	*256*
20	30	40	60	60	48.3	57.9	67.6	77.2	86.9	*96.5*	*116*	*145*	*193*	*241*	*290*
24	35	50	70	70	**59.8**	71.8	83.8	95.8	108	120	*144*	*180*	*239*	*299*	*359*
30	45	60	85	90	**72.4**	**86.9**	**101**	116	130	145	174	*217*	*290*	*362*	*434*

See clause 3.7(1) of BS EN 1993-1-8: 2005 for calculation of the design resistance of a group of fasteners.
Values of bearing resistance in **bold** are less than the single shear resistance of the bolt.
Values of bearing resistance in *italic* are greater than the double shear resistance of the bolt.
Bearing values assume standard clearance holes.
If oversize or short slotted holes are used, bearing values should be multiplied by 0.8.
If long slotted or kidney shaped holes are used, bearing values should be multiplied by 0.6.
In single lap joints with only one bolt row, the design bearing resistance for each bolt should be limited to $1.5 f_u d\ t/\gamma_{M2}$.

BS EN 1993-1-8:2005
BS EN ISO 4014
BS EN ISO 4017

Table G.50 Non Preloaded hexagon head bolts, property class 8.8, in S355

Diameter of Bolt	Tensile Stress Area	Tension Resistance	Shear Resistance		Bolts in tension
			Single Shear	Double Shear	Min thickness for punching shear
d	A_s	$F_{t,Rd}$	$F_{v,Rd}$	$2 \times F_{v,Rd}$	t_{min}
mm	mm^2	kN	kN	kN	mm
12	84.3	48.6	27.5	55.0	3.7
16	157	90.4	60.3	121	5.5
20	245	141	94.1	188	6.8
24	353	203	136	271	8.2
30	561	323	215	431	10.1

See notes below

Diameter of Bolt	Minimum				Bearing Resistance (kN)										
	Edge distance	End distance	Pitch	Gauge											
d	e_2	e_1	p_1	p_2	Thickness in mm of ply, t										
mm	mm	mm	mm	mm	5	6	7	8	9	10	12	15	20	25	30
12	20	25	35	40	30.3	36.3	42.4	48.4	54.5	*60.5*	*72.6*	*90.8*	*121*	*151*	*182*
16	25	35	50	50	**42.7**	**51.2**	**59.7**	68.3	76.8	85.4	102	*128*	*171*	*213*	*256*
20	30	40	60	60	**48.3**	**57.9**	**67.6**	**77.2**	**86.9**	96.5	116	145	*193*	*241*	*290*
24	35	50	70	70	**59.8**	**71.8**	**83.8**	**95.8**	**108**	**120**	144	180	239	*299*	*359*
30	45	60	85	90	**72.4**	**86.9**	**101**	**116**	**130**	**145**	**174**	217	290	362	*434*

See notes below

Diameter of Bolt	Minimum				Bearing Resistance (kN)										
	Edge distance	End distance	Pitch	Gauge											
d	e_2	e_1	p_1	p_2	Thickness in mm of ply, t										
mm	mm	mm	mm	mm	5	6	7	8	9	10	12	15	20	25	30
12	25	40	50	45	53.0	*63.7*	*74.3*	*84.9*	*95.5*	*106*	*127*	*159*	*212*	*265*	*318*
16	30	50	65	55	69.6	83.6	97.5	111	*125*	*139*	*167*	*209*	*279*	*348*	*418*
20	35	60	80	70	**85.5**	103	120	137	154	171	*205*	*256*	*342*	*427*	*513*
24	40	75	95	80	**108**	**130**	152	174	195	217	260	*325*	*434*	*542*	*651*
30	50	90	115	100	**128**	**154**	**179**	**205**	231	256	308	385	*513*	*641*	*769*

For M12 bolts the design shear resistance $F_{v,Rd}$ has been calculated as 0.85 times the value given in BS EN 1993-1-8, Table 3.4 (§3.6.1(5)).

See clause 3.7(1) of BS EN 1993-1-8: 2005 for calculation of the design resistance of a group of fasteners.

Values of bearing resistance in **bold** are less than the single shear resistance of the bolt.

Values of bearing resistance in *italic* are greater than the double shear resistance of the bolt.

Bearing values assume standard clearance holes.

If oversize or short slotted holes are used, bearing values should be multiplied by 0.8.

If long slotted or kidney shaped holes are used, bearing values should be multiplied by 0.6.

In single lap joints with only one bolt row, the design bearing resistance for each bolt should be limited to $1.5 f_u d\ t/\gamma_{M2}$.

BS EN 1993-1-8:2005
BS EN ISO 4014
BS EN ISO 4017

S355

Table G.51 Non Preloaded hexagon head bolts, property class 10.9, in S355

Diameter of Bolt	Tensile Stress Area	Tension Resistance	Shear Resistance		Bolts in tension
			Single Shear	Double Shear	Min thickness for punching shear
d	A_s	$F_{t,Rd}$	$F_{v,Rd}$	$2 \times F_{v,Rd}$	t_{min}
mm	mm^2	kN	kN	kN	mm
12	84.3	60.7	28.7	57.3	4.6
16	157	113	62.8	126	6.9
20	245	176	98.0	196	8.5
24	353	254	141	282	10.2
30	561	404	224	449	12.6

See notes below

Diameter of Bolt	Minimum Edge distance	Minimum End distance	Minimum Pitch	Minimum Gauge	Bearing Resistance (kN)										
d	e_2	e_1	p_1	p_2	Thickness in mm of ply, t										
mm	mm	mm	mm	mm	5	6	7	8	9	10	12	15	20	25	30
12	20	25	35	40	30.3	36.3	42.4	48.4	54.5	*60.5*	*72.6*	*90.8*	*121*	*151*	*182*
16	25	35	50	50	**42.7**	**51.2**	**59.7**	68.3	76.8	85.4	102	*128*	*171*	*213*	*256*
20	30	40	60	60	**48.3**	**57.9**	**67.6**	**77.2**	**86.9**	**96.5**	116	145	193	*241*	*290*
24	35	50	70	70	**59.8**	**71.8**	**83.8**	**95.8**	**108**	**120**	144	180	239	*299*	*359*
30	45	60	85	90	**72.4**	**86.9**	**101**	**116**	**130**	**145**	**174**	**217**	290	362	434

See notes below

Diameter of Bolt	Minimum Edge distance	Minimum End distance	Minimum Pitch	Minimum Gauge	Bearing Resistance (kN)										
d	e_2	e_1	p_1	p_2	Thickness in mm of ply, t										
mm	mm	mm	mm	mm	5	6	7	8	9	10	12	15	20	25	30
12	25	40	50	45	53	*63.7*	*74.3*	*84.9*	*95.5*	*106*	*127*	*159*	*212*	*265*	*318*
16	30	50	65	55	69.6	83.6	97.5	111	125	*139*	*167*	*209*	*279*	*348*	*418*
20	35	60	80	70	**85.5**	103	120	137	154	171	*205*	*256*	*342*	*427*	*513*
24	40	75	95	80	**108**	**130**	152	174	195	217	260	*325*	*434*	*542*	*651*
30	50	90	115	100	**128**	**154**	**179**	**205**	231	256	308	385	*513*	*641*	*769*

For M12 bolts the design shear resistance $F_{v,Rd}$ has been calculated as 0.85 times the value given in BS EN 1993-1-8, Table 3.4 (§3.6.1(5)).
See clause 3.7(1) of BS EN 1993-1-8: 2005 for calculation of the design resistance of a group of fasteners.
Values of bearing resistance in **bold** are less than the single shear resistance of the bolt.
Values of bearing resistance in *italic* are greater than the double shear resistance of the bolt.
Bearing values assume standard clearance holes.
If oversize or short slotted holes are used, bearing values should be multiplied by 0.8.
If long slotted or kidney shaped holes are used, bearing values should be multiplied by 0.6.
In single lap joints with only one bolt row, the design bearing resistance for each bolt should be limited to $1.5 f_u d\ t/\gamma_{M2}$.

BS EN 1993-1-8:2005
BS EN 14399:2005
EN 1090:2008

S355

Table G.52 Preloaded hexagon head bolts in category B shear connections, property class 8.8, in S355

Diameter of Bolt	Tensile Stress Area	Shear Resistance		Slip resistance, $F_{s,Rd,ser}$							
		Single Shear	Double Shear	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
d	A_s	$F_{v,Rd}$	2 x $F_{v,Rd}$								
mm	mm^2	kN	kN	kN	kN	kN	kN	kN	kN	kN	kN
12	84.3	27.5	55.0	8.6	17.2	12.9	25.7	17.2	34.3	21.5	42.9
16	157	60.3	121	16.0	32.0	24.0	48.0	32.0	63.9	40.0	79.9
20	245	94.1	188	24.9	49.9	37.4	74.8	49.9	100	62.4	125
24	353	136	271	35.9	71.9	53.9	108	71.9	144	89.9	180
30	561	215	431	57.1	114	85.7	171	114	228	143	286

Bearing resitances may be taken from the tables for non-preloaded bolts.

For M12 bolts the design shear resistance $F_{v,Rd}$ has been calculated as 0.85 times the value given in BS EN 1993-1-8, Table 3.4 (§3.6.1(5)).

See clause 3.7(1) of BS EN 1993-1-8: 2005 for calculation of the design resistance of a group of fasteners.

The shear resistances are ULS values.

The slip resistances are SLS values.

Values have been calculated assuming k_s=1. See BS EN 1993-1-8, section 3.9 for other values of k_s.

Table G.53 Preloaded hexagon head bolts in category E tension connections, property class 8.8, in S355

Diameter of Bolt	Tensile Stress Area	Tension Resistance	Min thickness for punching shear
d	A_s	$F_{t,Rd}$	t_{min}
mm	mm^2	kN	mm
12	84.3	48.6	3.2
16	157	90.4	4.7
20	245	141	6.2
24	353	203	7.0
30	561	323	9.1

The minimum thickness is such that the design punching shear resistance $B_{p,Rd}$ is equal to the design tension resistance, $F_{t,Rd}$

BS EN 1993-1-8:2005
BS EN 14399:2005
EN 1090:2008

S355

Table G.54 Preloaded hexagon head bolts in category B shear connections, property class 10.9, in S355

Diameter of Bolt	Tensile Stress Area	Shear Resistance		Slip resistance, $F_{s,Rd,ser}$							
		Single Shear	Double Shear	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
d	A_s	$F_{v,Rd}$	2 x $F_{v,Rd}$								
mm	mm^2	kN	kN	kN	kN	kN	kN	kN	kN	kN	kN
16	157	62.8	126	20.0	40.0	30.0	59.9	40.0	79.9	50.0	99.9
20	245	98.0	196	31.2	62.4	46.8	93.5	62.4	125	78.0	156
24	353	141	282	44.9	89.9	67.4	135	89.9	180	112	225
30	561	224	449	71.4	143	107	214	143	286	179	357

Bearing resitances should be taken from the tables for non-preloaded bolts.
For M12 bolts the design shear resistance $F_{v,Rd}$ has been calculated as 0.85 times the value given in BS EN 1993-1-8, Table 3.4 (§3.6.1(5)).
See clause 3.7(1) of BS EN 1993-1-8: 2005 for calculation of the design resistance of a group of fasteners.
The shear resistances are ULS values.
The slip resistances are SLS values.
Values have been calculated assuming k_s=1. See BS EN 1993-1-8, section 3.9 for other values of k_s.

Table G.55 Preloaded hexagon head bolts in category E tension connections, property class 10.9, in S355

Diameter of Bolt	Tensile Stress Area	Tension Resistance	Min thickness for punching shear
d	A_s	$F_{t,Rd}$	t_{min}
mm	mm^2	kN	mm
16	157	113	5.9
20	245	176	7.8
24	353	254	8.7
30	561	404	11.4

BS EN 1993-1-8:2005
BS EN 14399:2005
EN 1090:2008

Table G.56 Preloaded hexagon head bolts in category C shear connections, property class 8.8, in S355

Diameter of Bolt d mm	Tensile Stress Area A_s mm^2	Slip resistance, $F_{s,Rd}$							
		μ = 0.2		μ = 0.3		μ = 0.4		μ = 0.5	
		Single Shear kN	Double Shear kN	Single Shear kN	Double Shear kN	Single Shear kN	Double Shear kN	Single Shear kN	Double Shear kN
12	84.3	7.55	15.1	11.3	22.7	15.1	30.2	18.9	37.8
16	157	14.1	28.1	21.1	42.2	28.1	56.3	35.2	70.3
20	245	22.0	43.9	32.9	65.9	43.9	87.8	54.9	110
24	353	31.6	63.3	47.4	94.9	63.3	127	79.1	158
30	561	50.3	101	75.4	151	101	201	126	251

Bearing resistances may be taken from the tables for non-preloaded bolts.

BS EN 1993-1-8:2005
BS EN 14399:2005
EN 1090:2008

S355

Table G.57 Preloaded hexagon head bolts in category C shear connections, property class 10.9, in S355

Diameter of Bolt	Tensile Stress Area	Slip resistance, $F_{s,Rd}$							
		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
d	A_s								
mm	mm^2	kN	kN	kN	kN	kN	kN	kN	kN
16	157	17.6	35.2	26.4	52.8	35.2	70.3	44.0	87.9
20	245	27.4	54.9	41.2	82.3	54.9	110	68.6	137
24	353	39.5	79.1	59.3	119	79.1	158	98.8	198
30	561	62.8	126	94.2	188	126	251	157	314

Bearing resistances may be taken from the tables for non-preloaded bolts.

FLOWDRILL

Tension resistance – normal design

The tension resistances of Property class 8.8 bolts for normal design are shown in Table G.58 and take account of the hollow section wall thickness and the bolt strength.

Table G.58

FLOWDRILL CONNECTIONS Normal tension resistance ($F_{t,Rd}$) kN								
Bolt Diameter mm	Hollow section wall thickness (mm)							
	S275					S355		
	5	6.3	8	10	12.5	5	6.3	8 to 12.5
M16	46	60	70.3			59	70.3	
M20	70	85	95	97	110	102	110	
M24	80	101	122	134	158	103	130	158

Tension resistance – structural integrity

The resistances in Table G.59 for structural integrity are the normal resistance values, multiplied by $\frac{\gamma_{M2}}{\lambda_{Mu}} = \frac{1.25}{1.1}$. The local resistance of the member must also be checked (see Checks 14 and 15 for end plates).

Table G.59

FLOWDRILL CONNECTIONS Structural integrity tension resistance ($F_{t,Rd,u}$) kN								
Bolt Diameter mm	Hollow section wall thickness (mm)							
	S275					S355		
	5	6.3	8	10	12.5	5	6.3	8 to 12.5
M16	52	68	80			67	80	
M20	80	97	108	110	125	116	125	
M24	91	115	139	152	180	117	148	180

Shear and bearing resistance

For shear and bearing resistances, refer to resistance tables for ordinary bolts (Tables G.40 and G.49).

Note: Additional information on Flowdrill is given in Appendix D.

HOLLO-BOLT RESISTANCES

Shear resistance

Hollo-Bolts have a shear resistance higher than that for ordinary bolting, since the body of the fastener provides resistance as well as the bolt. Values are given in Table G.60 for carbon steel Hollo-Bolts and in Table G.61 for stainless steel Hollo-Bolts.

Tension resistance – normal design

The tension resistances of Hollo-Bolts for normal design are shown in Table G.60 for carbon steel Hollo-Bolts and in Table G.61 for stainless steel Hollo-Bolts.

Tension resistance – structural integrity

The pull-out resistances for structural integrity are higher than those for normal design because large deformations are allowed and a lower partial factor is applied. Values are given in Table G.60 for carbon steel Hollo-Bolts and in Table G.61 for stainless steel Hollo-Bolts. The local resistance of the member must also be checked (see Checks 14 and 15 for end plates).

Table G.60

HOLLO-BOLT DESIGN RESISTANCES FOR CARBON STEEL			
Bolt Diameter (mm)	**Shear Resistance, $F_{v,Rd}$ (kN)**	**Normal Tension Resistance, $F_{t,Rd}$ (kN)**	**Structural Integrity Tension Resistance, $F_{t,Rd,u}$ (kN)**
M8	26.3	18.5	21.0
M10	43.4	31.7	36.0
M12	56.8	36.6	41.6
M16	111	67.4	76.6
M20	169	99.2	113

Table G.61

HOLLO-BOLT DESIGN RESISTANCES FOR STAINLESS STEEL			
Bolt Diameter (mm)	**Shear Resistance, $F_{v,Rd}$ (kN)**	**Normal Tension Resistance, $F_{t,Rd}$ (kN)**	**Structural Integrity Tension Resistance, $F_{t,Rd,u}$ (kN)**
M8	24.6	21.4	24.4
M10	40.8	36.8	41.8
M12	52.0	42.6	48.5
M16	102	78.4	89.1
M20	164	123	140

Note: The resistance values given in this table have been provided by the manufacturer. Contact the manufacturer for up-to-date information.

Bearing resistance

For bearing resistances refer to resistance tables for ordinary bolts (Tables G.40 to G49).

Note: Additional information on Hollo-Bolts is given in Appendix E.

BLIND BOLT RESISTANCES

Shear resistance

The shear resistances of Blind Bolts are given in Table G.62. The resistance depends if the shear plane is within the length of the slot, or the shank of the bolt, when it is assumed that the shear plane falls in the threaded length. The values in the threaded length account for the grooves machined along the length of the shank.

Table G.62

ZINC AND YELLOW BLIND BOLT DESIGN RESISTANCES		
Diameter	**Shear resistance (threaded zone)** $F_{v,Rd,thread}$ **(kN)**	**Shear resistance (over slot)** $F_{v,Rd,slot}$ **(kN)**
M8	14.6	11.1
M10	23.2	19.0
M12	33.7	26.3
M16	62.7	51.5
M20	97.9	76.1
M24	141	105

Tension resistance

Tensile resistances for Blind Bolts are given in Table G.63. The resistance values for structural integrity checks are the design values, multiplied by $\frac{\gamma_{M2}}{\lambda_{Mu}} = \frac{1.25}{1.1}$. Note that the resistances in Table G.63 for use in the structural integrity checks are for the Blind Bolt alone. The local resistance of the member must also be checked (see Checks 14 and 15 for end plates).

Table G.63

ZINC AND YELLOW BLIND BOLT DESIGN RESISTANCES		
Diameter	**Normal Tension Resistance** $F_{t,Rd}$ **(kN)**	**Structural Integrity Tension Resistance** $F_{t,Rd,u}$ **(kN)**
M8	6.9	7.8
M10	12.9	14.7
M12	18.8	21.4
M16	40.1	45.6
M20	57.8	65.7
M24	82.3	93.5

Note: The resistance values given in these tables have been provided by the manufacturer. Contact the manufacturer for up-to-date information.

Bearing resistance

For bearing resistances refer to resistance tables for ordinary bolts (Tables G.40 to G49).

Combined shear and tension

Blind Bolts should satisfy the requirements of BS EN 1993-1-8 Table 3.4

FILLET WELDS

Table G.64 Design resistances - S275

Leg Length *s* mm	Throat Thickness *a* mm	Longitudinal resistance $F_{w,L,Rd}$ kN/mm	Transverse resistance $F_{w,T,Rd}$ kN/mm
6.0	4.2	0.94	1.15
8.0	5.6	1.25	1.53
10.0	7.0	1.56	1.91
12.0	8.4	1.87	2.29
15.0	10.5	2.34	2.87
18.0	12.6	2.81	3.44
20.0	14.0	3.12	3.82

Table G.65 Design resistances - S355

Leg Length *s* mm	Throat Thickness *a* mm	Longitudinal resistance $F_{w,L,Rd}$ kN/mm	Transverse resistance $F_{w,T,Rd}$ kN/mm
6.0	4.2	1.01	1.24
8.0	5.6	1.35	1.65
10.0	7.0	1.69	2.07
12.0	8.4	2.03	2.48
15.0	10.5	2.53	3.10
18.0	12.6	3.04	3.72
20.0	14.0	3.38	4.14

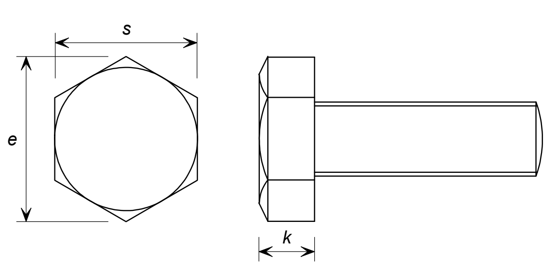
Table G.66

DIMENSIONS OF ORDINARY BOLT ASSEMBLIES
(All dimensions in millimetres)

Nuts, bolts and washers are covered by a number of British, European and International Standards. Specification of these components should be in accordance with the product Standards referenced in the latest version of the National Structural Steelwork Specification[10]. The details and dimensions in this table are intended only to provide sufficient information to enable the connection design to be completed.

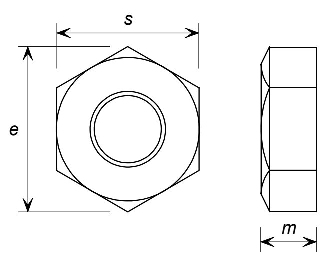
Approximate dimensions are shown in the following tables; precise dimensions, if required, should be obtained from the relevant product Standard.

Ordinary Bolts



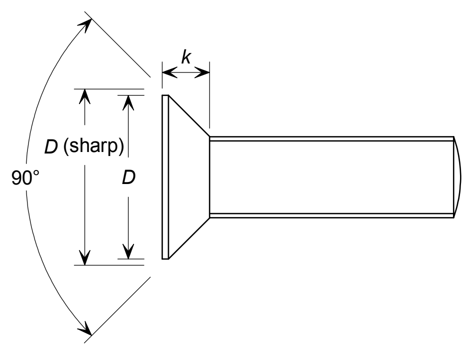
	M12	M16	**M20**	M24
s	18	24	**30**	36
e	20	26	**33**	40
k	8	10	**13**	15

Nuts



	M12	M16	**M20**	M24
s	18	24	**30**	36
e	20	26	**33**	40
m	12	16	**19**	22

90° Countersunk Round Bolts



	M12	M16	**M20**	M24
D (sharp)	24	32	**40**	48
D	20	27	**34**	41
k	6	8	**10**	12

Washers

Bolt Size	M12	M16	**M20**	M24
Outside dia	24	30	**37**	44
Thickness	2.5	3	**3**	4

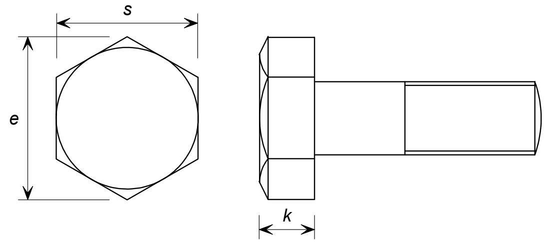
Table G.67

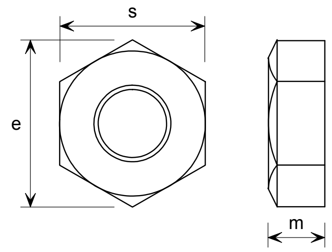
DIMENSIONS OF PRELOADED BOLT ASSEMBLIES
(All dimensions in millimetres)

Nuts, bolts and washers are covered by a number of British, European and International Standards. Specification of these components should be in accordance with the product Standards referenced in the latest version of the National Structural Steelwork Specification[10]. The details and dimensions in this table are intended only to provide sufficient information to enable the connection design to be completed.

Approximate dimensions are shown in the following tables; precise dimensions, if required, should be obtained from the relevant product Standard.

Preloaded Bolts and Nuts

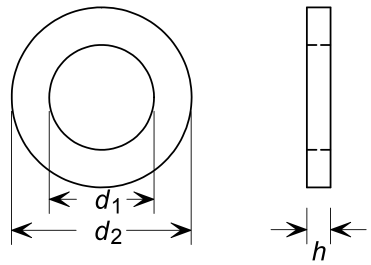




	M16	**M20**	M24	M30
s	27	**32**	41	50
e	30	**35**	45	55
k	10	**13**	15	19

	M16	**M20**	M24	M30
s	27	**32**	41	50
e	30	**35**	45	55
m	15	**18**	21	25

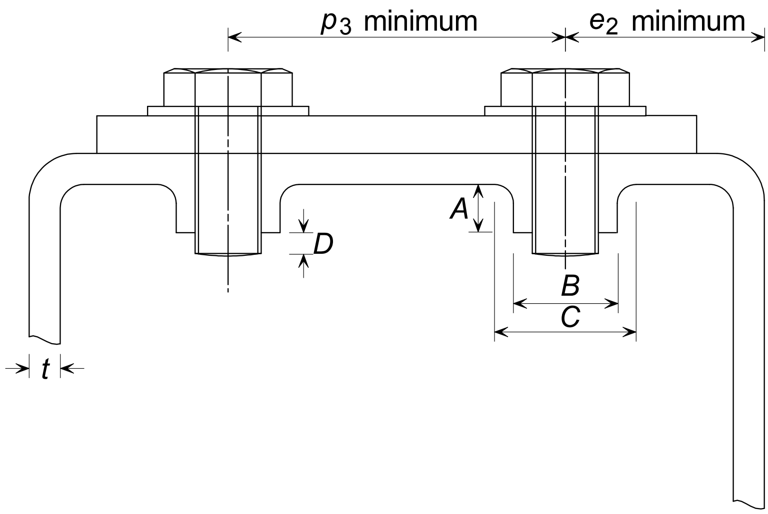
Plain and chamfered Washers



	M16	**M20**	M24	M30
d_1	17	**21**	25	31
d_2	30	**37**	44	56
h	4	**4**	4	5

Table G.68

DETAILING OF THERMAL DRILLING BOLT ASSEMBLIES



	Dimensions (mm) for bolt size			
	M12	**M16**	**M20**	**M24**
A	7	10	12	15
B	13	17	22	25
C	18	20	26	29
D	Varies with overall bolt length			
e_2 minimum	$C/2 + t$ (for connections made to a single face or opposite faces)			
	$B/2 + A + D + t$ (for connections made to adjacent faces)			
p_3 minimum	30	40	50	60

Notes:

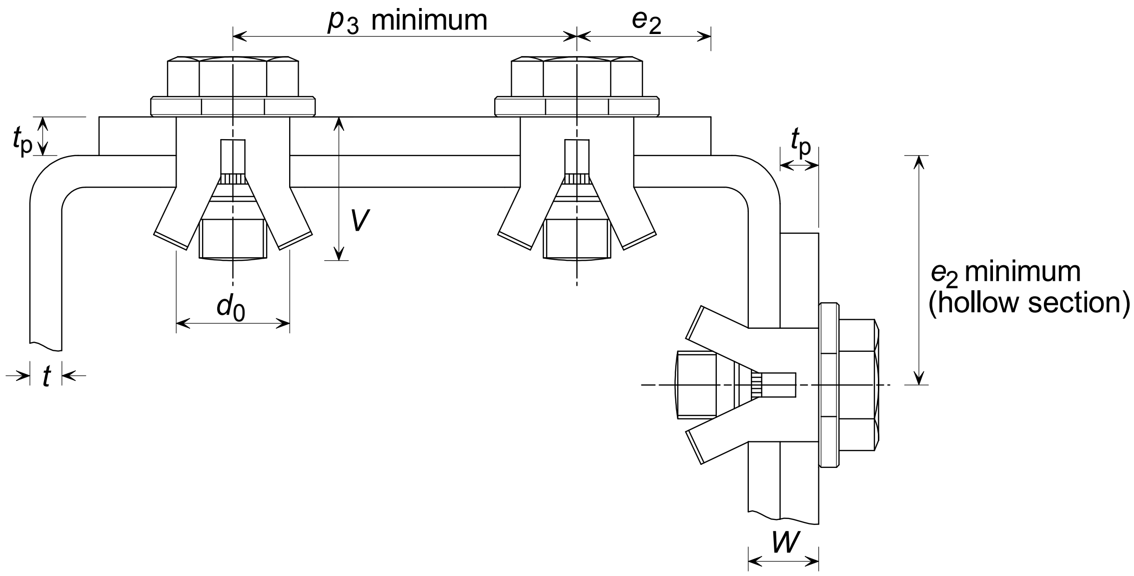
1. The thermal drilling process is limited to hollow sections thicknesses up to and including 12.5 mm. For thicknesses of 16 mm and over, conventional drill and tap methods are recommended, although, because the hollow section material strength is lower than that of property class 8.8 bolts, pull out strengths may be below the bolt tension resistance.

2. Detailing must comply with the requirements given in Check 1 of the appropriate design procedure.

3. Additional information on thermal drilling is given in Appendix D.

Table G.69

DETAILING OF HOLLO-BOLT ASSEMBLIES

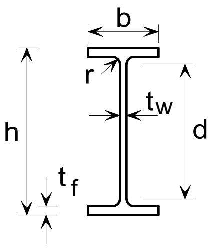


Bolt Size	Bolt length V (mm)	Fixing thickness W (mm)		p_3 (min.) (mm)	e_2 (min. on fitting) (mm)	e_2 (min. on section) (mm)	Hole dia. d_0 (mm)	Dimension across flats of collar (mm)	Nominal bolt dia. (mm)	Tightening torque (Nm)
		min	max							
M8 (Size 1)	50	3	22	35	13	$50 - t_p$	13.8 -15	19	8	23
M8 (Size 2)	70	22	41							
M8 (Size 3)	90	41	60							
M10 (Size 1)	55	3	22	40	15	$55 - t_p$	17.8 -19	24	10	45
M10 (Size 2)	70	22	41							
M10 (Size 3)	90	41	60							
M12 (Size 1)	60	3	25	50	18	$60 - t_p$	19.8 -21	30	12	80
M12 (Size 2)	80	25	47							
M12 (Size 3)	100	47	69							
M16 (Size 1)	75	12	29	55	20	$75 - t_p$	25.8 -28	36	16	190
M16 (Size 2)	100	29	50							
M16 (Size 3)	120	50	71							
M20 (Size 1)	90	12	34	70	25	$90 - t_p$	32.8 - 35	46	20	300
M20 (Size 2)	120	34	60							
M20 (Size 3)	150	60	86							

Notes:

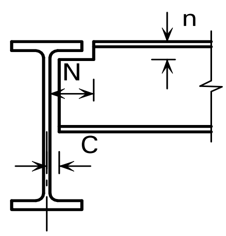
1. To maximise shear resistance, the fitting thickness (t_p) must be at least 8 mm when M16 or M20 Hollo-Bolts are used. Where the outer ply is less than 8 mm, washers should be used to make the thickness up to 8 mm.
2. Detailing must comply with the requirements given in Check 1 of the appropriate design procedure.
3. Additional information on Hollo-Bolts is given in Appendix E.

Table G.70 **BS 4-1:2005**



UNIVERSAL BEAMS

Advance UKB



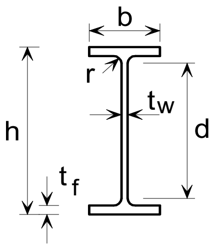
Dimensions

Section Designation	Mass per Metre	Depth of Section	Width of Section	Thickness		Root Radius	Depth between Fillets	Ratios for Local Buckling		Dimensions for Detailing			Surface Area		Area of Section
				Web	Flange			Flange	Web	End Clearance	Notch		Per Metre	Per Tonne	
		h	*b*	t_w	t_f	*r*	*d*	c_f / t_f	c_w / t_w	*C*	*N*	*n*			*A*
	kg/m	mm	mm	mm	mm	mm	mm			mm	mm	mm	m^2	m^2	cm^2
1016x305x487+	486.7	1036.3	308.5	30.0	54.1	30.0	868.1	2.02	28.9	17	150	86	3.20	6.58	620
1016x305x437 +	437.0	1026.1	305.4	26.9	49.0	30.0	868.1	2.23	32.3	15	150	80	3.17	7.25	557
1016x305x393 +	392.7	1015.9	303.0	24.4	43.9	30.0	868.1	2.49	35.6	14	150	74	3.14	8.00	500
1016x305x349+	349.4	1008.1	302.0	21.1	40.0	30.0	868.1	2.76	41.1	13	152	70	3.13	8.96	445
1016x305x314 +	314.3	999.9	300.0	19.1	35.9	30.0	868.1	3.08	45.5	12	152	66	3.11	9.89	400
1016x305x272 +	272.3	990.1	300.0	16.5	31.0	30.0	868.1	3.60	52.6	10	152	62	3.10	11.4	347
1016x305x249 +	248.7	980.1	300.0	16.5	26.0	30.0	868.1	4.30	52.6	10	152	56	3.08	12.4	317
1016x305x222 +	222.0	970.3	300.0	16.0	21.1	30.0	868.1	5.31	54.3	10	152	52	3.06	13.8	283
914x419x388	388.0	921.0	420.5	21.4	36.6	24.1	799.6	4.79	37.4	13	210	62	3.44	8.87	494
914x419x343	343.3	911.8	418.5	19.4	32.0	24.1	799.6	5.48	41.2	12	210	58	3.42	9.96	437
914x305x289	289.1	926.6	307.7	19.5	32.0	19.1	824.4	3.91	42.3	12	156	52	3.01	10.4	368
914x305x253	253.4	918.4	305.5	17.3	27.9	19.1	824.4	4.48	47.7	11	156	48	2.99	11.8	323
914x305x224	224.2	910.4	304.1	15.9	23.9	19.1	824.4	5.23	51.8	10	156	44	2.97	13.2	286
914x305x201	200.9	903.0	303.3	15.1	20.2	19.1	824.4	6.19	54.6	10	156	40	2.96	14.7	256
838x292x226	226.5	850.9	293.8	16.1	26.8	17.8	761.7	4.52	47.3	10	150	46	2.81	12.4	289
838x292x194	193.8	840.7	292.4	14.7	21.7	17.8	761.7	5.58	51.8	9	150	40	2.79	14.4	247
838x292x176	175.9	834.9	291.7	14.0	18.8	17.8	761.7	6.44	54.4	9	150	38	2.78	15.8	224
762x267x197	196.8	769.8	268.0	15.6	25.4	16.5	686.0	4.32	44.0	10	138	42	2.55	13.0	251
762x267x173	173.0	762.2	266.7	14.3	21.6	16.5	686.0	5.08	48.0	9	138	40	2.53	14.6	220
762x267x147	146.9	754.0	265.2	12.8	17.5	16.5	686.0	6.27	53.6	8	138	34	2.51	17.1	187
762x267x134	133.9	750.0	264.4	12.0	15.5	16.5	686.0	7.08	57.2	8	138	32	2.51	18.7	171
686x254x170	170.2	692.9	255.8	14.5	23.7	15.2	615.1	4.45	42.4	9	132	40	2.35	13.8	217
686x254x152	152.4	687.5	254.5	13.2	21.0	15.2	615.1	5.02	46.6	9	132	38	2.34	15.4	194
686x254x140	140.1	683.5	253.7	12.4	19.0	15.2	615.1	5.55	49.6	8	132	36	2.33	16.6	178
686x254x125	125.2	677.9	253.0	11.7	16.2	15.2	615.1	6.51	52.6	8	132	32	2.32	18.5	159
610x305x238	238.1	635.8	311.4	18.4	31.4	16.5	540.0	4.14	29.3	11	158	48	2.45	10.3	303
610x305x179	179.0	620.2	307.1	14.1	23.6	16.5	540.0	5.51	38.3	9	158	42	2.41	13.5	228
610x305x149	149.2	612.4	304.8	11.8	19.7	16.5	540.0	6.60	45.8	8	158	38	2.39	16.0	190
610x229x140	139.9	617.2	230.2	13.1	22.1	12.7	547.6	4.34	41.8	9	120	36	2.11	15.1	178
610x229x125	125.1	612.2	229.0	11.9	19.6	12.7	547.6	4.89	46.0	8	120	34	2.09	16.7	159
610x229x113	113.0	607.6	228.2	11.1	17.3	12.7	547.6	5.54	49.3	8	120	30	2.08	18.4	144
610x229x101	101.2	602.6	227.6	10.5	14.8	12.7	547.6	6.48	52.2	7	120	28	2.07	20.5	129
610x178x100 +	100.3	607.4	179.2	11.3	17.2	12.7	547.6	4.14	48.5	8	94	30	1.89	18.8	128
610x178x92 +	92.2	603.0	178.8	10.9	15.0	12.7	547.6	4.75	50.2	7	94	28	1.88	20.4	117
610x178x82 +	81.8	598.6	177.9	10.0	12.8	12.7	547.6	5.57	54.8	7	94	26	1.87	22.9	104
533x312x273 +	273.3	577.1	320.2	21.1	37.6	12.7	476.5	3.64	22.6	13	160	52	2.37	8.67	348
533x312x219 +	218.8	560.3	317.4	18.3	29.2	12.7	476.5	4.69	26.0	11	160	42	2.33	10.7	279
533x312x182 +	181.5	550.7	314.5	15.2	24.4	12.7	476.5	5.61	31.3	10	160	38	2.31	12.7	231
533x312x151 +	150.6	542.5	312.0	12.7	20.3	12.7	476.5	6.75	37.5	8	160	34	2.29	15.2	192

Advance and UKB are registered trademarks of Tata Steel. A fuller description of the relationship between Universal Beams (UB) and the Advance range of sections manufactured by Tata Steel is given in Appendix H.

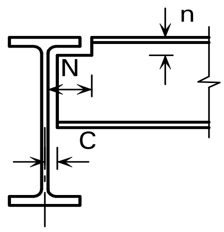
+ These sections are in addition to the range of BS 4 sections.

Table G.70 *Continued* **BS 4-1:2005**



UNIVERSAL BEAMS

Advance UKB



Dimensions

Section Designation	Mass per Metre	Depth of Section	Width of Section	Thickness		Root Radius	Depth between Fillets	Ratios for Local Buckling		Dimensions for Detailing			Surface Area		Area of Section
				Web	Flange			Flange	Web	End Clearance	Notch		Per Metre	Per Tonne	
		h	*b*	t_w	t_f	*r*	*d*	c_f / t_f	c_w / t_w	*C*	*N*	*n*			*A*
	kg/m	mm	mm	mm	mm	mm	mm			mm	mm	mm	m^2	m^2	cm^2
533x210x138 +	138.3	549.1	213.9	14.7	23.6	12.7	476.5	3.68	32.4	9	110	38	1.90	13.7	176
533x210x122	122.0	544.5	211.9	12.7	21.3	12.7	476.5	4.08	37.5	8	110	34	1.89	15.5	155
533x210x109	109.0	539.5	210.8	11.6	18.8	12.7	476.5	4.62	41.1	8	110	32	1.88	17.2	139
533x210x101	101.0	536.7	210.0	10.8	17.4	12.7	476.5	4.99	44.1	7	110	32	1.87	18.5	129
533x210x92	92.1	533.1	209.3	10.1	15.6	12.7	476.5	5.57	47.2	7	110	30	1.86	20.2	117
533x210x82	82.2	528.3	208.8	9.6	13.2	12.7	476.5	6.58	49.6	7	110	26	1.85	22.5	105
533x165x85 +	84.8	534.9	166.5	10.3	16.5	12.7	476.5	3.96	46.3	7	90	30	1.69	19.9	108
533x165x75 +	74.7	529.1	165.9	9.7	13.6	12.7	476.5	4.81	49.1	7	90	28	1.68	22.5	95.2
533x165x66 +	65.7	524.7	165.1	8.9	11.4	12.7	476.5	5.74	53.5	6	90	26	1.67	25.4	83.7
457x191x161 +	161.4	492.0	199.4	18.0	32.0	10.2	407.6	2.52	22.6	11	102	44	1.73	10.7	206
457x191x133 +	133.3	480.6	196.7	15.3	26.3	10.2	407.6	3.06	26.6	10	102	38	1.70	12.8	170
457x191x106 +	105.8	469.2	194.0	12.6	20.6	10.2	407.6	3.91	32.3	8	102	32	1.67	15.8	135
457x191x98	98.3	467.2	192.8	11.4	19.6	10.2	407.6	4.11	35.8	8	102	30	1.67	17.0	125
457x191x89	89.3	463.4	191.9	10.5	17.7	10.2	407.6	4.55	38.8	7	102	28	1.66	18.6	114
457x191x82	82.0	460.0	191.3	9.9	16.0	10.2	407.6	5.03	41.2	7	102	28	1.65	20.1	104
457x191x74	74.3	457.0	190.4	9.0	14.5	10.2	407.6	5.55	45.3	7	102	26	1.64	22.1	94.6
457x191x67	67.1	453.4	189.9	8.5	12.7	10.2	407.6	6.34	48.0	6	102	24	1.63	24.3	85.5
457x152x82	82.1	465.8	155.3	10.5	18.9	10.2	407.6	3.29	38.8	7	84	30	1.51	18.4	105
457x152x74	74.2	462.0	154.4	9.6	17.0	10.2	407.6	3.66	42.5	7	84	28	1.50	20.2	94.5
457x152x67	67.2	458.0	153.8	9.0	15.0	10.2	407.6	4.15	45.3	7	84	26	1.50	22.3	85.6
457x152x60	59.8	454.6	152.9	8.1	13.3	10.2	407.6	4.68	50.3	6	84	24	1.49	24.9	76.2
457x152x52	52.3	449.8	152.4	7.6	10.9	10.2	407.6	5.71	53.6	6	84	22	1.48	28.3	66.6
406x178x85 +	85.3	417.2	181.9	10.9	18.2	10.2	360.4	4.14	33.1	7	96	30	1.52	17.8	109
406x178x74	74.2	412.8	179.5	9.5	16.0	10.2	360.4	4.68	37.9	7	96	28	1.51	20.4	94.5
406x178x67	67.1	409.4	178.8	8.8	14.3	10.2	360.4	5.23	41.0	6	96	26	1.50	22.3	85.5
406x178x60	60.1	406.4	177.9	7.9	12.8	10.2	360.4	5.84	45.6	6	96	24	1.49	24.8	76.5
406x178x54	54.1	402.6	177.7	7.7	10.9	10.2	360.4	6.86	46.8	6	96	22	1.48	27.3	69.0
406x140x53 +	53.3	406.6	143.3	7.9	12.9	10.2	360.4	4.46	45.6	6	78	24	1.35	25.3	67.9
406x140x46	46.0	403.2	142.2	6.8	11.2	10.2	360.4	5.13	53.0	5	78	22	1.34	29.1	58.6
406x140x39	39.0	398.0	141.8	6.4	8.6	10.2	360.4	6.69	56.3	5	78	20	1.33	34.1	49.7
356x171x67	67.1	363.4	173.2	9.1	15.7	10.2	311.6	4.58	34.2	7	94	26	1.38	20.6	85.5
356x171x57	57.0	358.0	172.2	8.1	13.0	10.2	311.6	5.53	38.5	6	94	24	1.37	24.1	72.6
356x171x51	51.0	355.0	171.5	7.4	11.5	10.2	311.6	6.25	42.1	6	94	22	1.36	26.7	64.9
356x171x45	45.0	351.4	171.1	7.0	9.7	10.2	311.6	7.41	44.5	6	94	20	1.36	30.2	57.3
356x127x39	39.1	353.4	126.0	6.6	10.7	10.2	311.6	4.63	47.2	5	70	22	1.18	30.2	49.8
356x127x33	33.1	349.0	125.4	6.0	8.5	10.2	311.6	5.82	51.9	5	70	20	1.17	35.4	42.1
305x165x54	54.0	310.4	166.9	7.9	13.7	8.9	265.2	5.15	33.6	6	90	24	1.26	23.3	68.8
305x165x46	46.1	306.6	165.7	6.7	11.8	8.9	265.2	5.98	39.6	5	90	22	1.25	27.1	58.7
305x165x40	40.3	303.4	165.0	6.0	10.2	8.9	265.2	6.92	44.2	5	90	20	1.24	30.8	51.3

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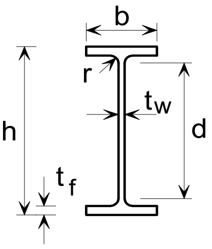
+ These sections are in addition to the range of BS 4 sections.

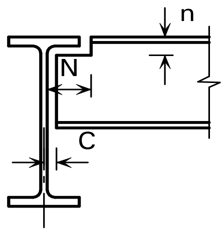
Table G.70 *Continued*

BS 4-1:2005



UNIVERSAL BEAMS

Advance UKB



Dimensions

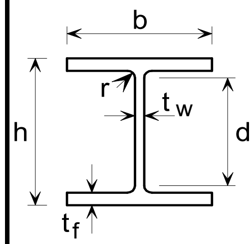
Section Designation	Mass per Metre	Depth of Section	Width of Section	Thickness		Root Radius	Depth between Fillets	Ratios for Local Buckling		Dimensions for Detailing			Surface Area		Area of Section
				Web	Flange			Flange	Web	End Clearance	Notch		Per Metre	Per Tonne	
		h	*b*	t_w	t_f	*r*	*d*	c_f / t_f	c_w / t_w	*C*	*N*	*n*			*A*
	kg/m	mm	mm	mm	mm	mm	mm			mm	mm	mm	m^2	m^2	cm^2
305x127x48	48.1	311.0	125.3	9.0	14.0	8.9	265.2	3.52	29.5	7	70	24	1.09	22.7	61.2
305x127x42	41.9	307.2	124.3	8.0	12.1	8.9	265.2	4.07	33.2	6	70	22	1.08	25.8	53.4
305x127x37	37.0	304.4	123.4	7.1	10.7	8.9	265.2	4.60	37.4	6	70	20	1.07	28.9	47.2
305x102x33	32.8	312.7	102.4	6.6	10.8	7.6	275.9	3.73	41.8	5	58	20	1.01	30.8	41.8
305x102x28	28.2	308.7	101.8	6.0	8.8	7.6	275.9	4.58	46.0	5	58	18	1.00	35.5	35.9
305x102x25	24.8	305.1	101.6	5.8	7.0	7.6	275.9	5.76	47.6	5	58	16	0.992	40.0	31.6
254x146x43	43.0	259.6	147.3	7.2	12.7	7.6	219.0	4.92	30.4	6	82	22	1.08	25.1	54.8
254x146x37	37.0	256.0	146.4	6.3	10.9	7.6	219.0	5.73	34.8	5	82	20	1.07	28.9	47.2
254x146x31	31.1	251.4	146.1	6.0	8.6	7.6	219.0	7.26	36.5	5	82	18	1.06	34.0	39.7
254x102x28	28.3	260.4	102.2	6.3	10.0	7.6	225.2	4.04	35.7	5	58	18	0.904	31.9	36.1
254x102x25	25.2	257.2	101.9	6.0	8.4	7.6	225.2	4.80	37.5	5	58	16	0.897	35.7	32.0
254x102x22	22.0	254.0	101.6	5.7	6.8	7.6	225.2	5.93	39.5	5	58	16	0.890	40.5	28.0
203x133x30	30.0	206.8	133.9	6.4	9.6	7.6	172.4	5.85	26.9	5	74	18	0.923	30.8	38.2
203x133x25	25.1	203.2	133.2	5.7	7.8	7.6	172.4	7.20	30.2	5	74	16	0.915	36.5	32.0
203x102x23	23.1	203.2	101.8	5.4	9.3	7.6	169.4	4.37	31.4	5	60	18	0.790	34.2	29.4
178x102x19	19.0	177.8	101.2	4.8	7.9	7.6	146.8	5.14	30.6	4	60	16	0.738	38.7	24.3
152x89x16	16.0	152.4	88.7	4.5	7.7	7.6	121.8	4.48	27.1	4	54	16	0.638	40.0	20.3
127x76x13	13.0	127.0	76.0	4.0	7.6	7.6	96.6	3.74	24.2	4	46	16	0.537	41.4	16.5

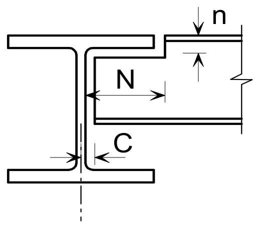
Advance and UKB are registered trademarks of Tata Steel. A fuller description of the relationship between Universal Beams (UB) and the Advance range of sections manufactured by Tata Steel is given in Appendix H.

Table G.71 BS 4-1:2005

UNIVERSAL COLUMNS

Advance UKC





Dimensions

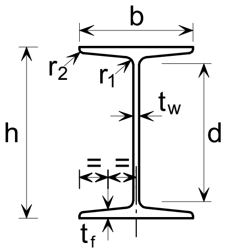
Section Designation	Mass per Metre	Depth of Section	Width of Section	Thickness		Root Radius	Depth between Fillets	Ratios for Local Buckling		Dimensions for Detailing			Surface Area		Area of Section
				Web	Flange			Flange	Web	End Clearance	Notch		Per Metre	Per Tonne	
		h	b	t_w	t_f	r	d	c_f / t_f	c_w / t_w	C	N	n			A
	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	2.25	6.10	26	200	94	2.52	3.98	808
356x406x551	551.0	455.6	418.5	42.1	67.5	15.2	290.2	2.56	6.89	23	200	84	2.47	4.48	702
356x406x467	467.0	436.6	412.2	35.8	58.0	15.2	290.2	2.98	8.11	20	200	74	2.42	5.18	595
356x406x393	393.0	419.0	407.0	30.6	49.2	15.2	290.2	3.52	9.48	17	200	66	2.38	6.06	501
356x406x340	339.9	406.4	403.0	26.6	42.9	15.2	290.2	4.03	10.9	15	200	60	2.35	6.91	433
356x406x287	287.1	393.6	399.0	22.6	36.5	15.2	290.2	4.74	12.8	13	200	52	2.31	8.05	366
356x406x235	235.1	381.0	394.8	18.4	30.2	15.2	290.2	5.73	15.8	11	200	46	2.28	9.70	299
356x368x202	201.9	374.6	374.7	16.5	27.0	15.2	290.2	6.07	17.6	10	190	44	2.19	10.8	257
356x368x177	177.0	368.2	372.6	14.4	23.8	15.2	290.2	6.89	20.2	9	190	40	2.17	12.3	226
356x368x153	152.9	362.0	370.5	12.3	20.7	15.2	290.2	7.92	23.6	8	190	36	2.16	14.1	195
356x368x129	129.0	355.6	368.6	10.4	17.5	15.2	290.2	9.4	27.9	7	190	34	2.14	16.6	164
305x305x283	282.9	365.3	322.2	26.8	44.1	15.2	246.7	3.00	9.21	15	158	60	1.94	6.86	360
305x305x240	240.0	352.5	318.4	23.0	37.7	15.2	246.7	3.51	10.7	14	158	54	1.91	7.96	306
305x305x198	198.1	339.9	314.5	19.1	31.4	15.2	246.7	4.22	12.9	12	158	48	1.87	9.44	252
305x305x158	158.1	327.1	311.2	15.8	25.0	15.2	246.7	5.30	15.6	10	158	42	1.84	11.6	201
305x305x137	136.9	320.5	309.2	13.8	21.7	15.2	246.7	6.11	17.90	9	158	38	1.82	13.3	174
305x305x118	117.9	314.5	307.4	12.0	18.7	15.2	246.7	7.09	20.6	8	158	34	1.81	15.4	150
305x305x97	96.9	307.9	305.3	9.9	15.4	15.2	246.7	8.60	24.9	7	158	32	1.79	18.5	123
254x254x167	167.1	289.1	265.2	19.2	31.7	12.7	200.3	3.48	10.4	12	134	46	1.58	9.46	213
254x254x132	132.0	276.3	261.3	15.3	25.3	12.7	200.3	4.36	13.1	10	134	38	1.55	11.7	168
254x254x107	107.1	266.7	258.8	12.8	20.5	12.7	200.3	5.38	15.6	8	134	34	1.52	14.2	136
254x254x89	88.9	260.3	256.3	10.3	17.3	12.7	200.3	6.38	19.4	7	134	30	1.50	16.9	113
254x254x73	73.1	254.1	254.6	8.6	14.2	12.7	200.3	7.77	23.3	6	134	28	1.49	20.4	93.1
203x203x127+	127.5	241.4	213.9	18.1	30.1	10.2	160.8	2.91	8.88	11	108	42	1.28	10.0	162
203x203x113+	113.5	235.0	212.1	16.3	26.9	10.2	160.8	3.26	9.87	10	108	38	1.27	11.2	145
203x203x100+	99.6	228.6	210.3	14.5	23.7	10.2	160.8	3.70	11.1	9	108	34	1.25	12.6	127
203x203x86	86.1	222.2	209.1	12.7	20.5	10.2	160.8	4.29	12.7	8	110	32	1.24	14.4	110
203x203x71	71.0	215.8	206.4	10.0	17.3	10.2	160.8	5.09	16.1	7	110	28	1.22	17.2	90.4
203x203x60	60.0	209.6	205.8	9.4	14.2	10.2	160.8	6.20	17.1	7	110	26	1.21	20.2	76.4
203x203x52	52.0	206.2	204.3	7.9	12.5	10.2	160.8	7.04	20.4	6	110	24	1.20	23.1	66.3
203x203x46	46.1	203.2	203.6	7.2	11.0	10.2	160.8	8.00	22.3	6	110	22	1.19	25.8	58.7
152x152x51+	51.2	170.2	157.4	11.0	15.7	7.6	123.6	4.18	11.2	8	84	24	0.935	18.3	65.2
152x152x44+	44.0	166.0	155.9	9.5	13.6	7.6	123.6	4.82	13.0	7	84	22	0.924	21.0	56.1
152x152x37	37.0	161.8	154.4	8.0	11.5	7.6	123.6	5.70	15.5	6	84	20	0.912	24.7	47.1
152x152x30	30.0	157.6	152.9	6.5	9.4	7.6	123.6	6.98	19.0	5	84	18	0.901	30.0	38.3
152x152x23	23.0	152.4	152.2	5.8	6.8	7.6	123.6	9.65	21.3	5	84	16	0.889	38.7	29.2

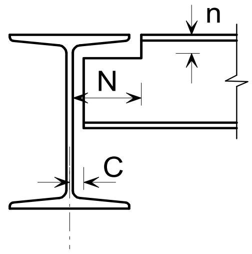
Advance and UKC are registered trademarks of Tata Steel. A fuller description of the relationship between Universal Columns (UC) and the Advance range of sections manufactured by Tata Steel is given in Appendix H.

+ These sections are in addition to the range of BS 4 sections.

Table G.72 BS 4-1:2005

JOISTS





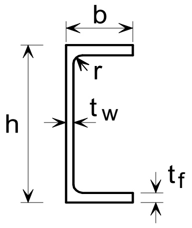
Dimensions

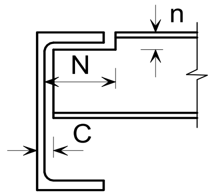
Section Designation	Mass per Metre	Depth of Section	Width of Section	Thickness		Radii		Depth between Fillets	Ratios for Local Buckling		Dimensions for Detailing			Surface Area		Area of Section
				Web	Flange	Root	Toe		Flange	Web	End Clearance	Notch		Per Metre	Per Tonne	
		h	b	t_w	t_f	r_1	r_2	d	c_f / t_f	c_w / t_w	C	N	n			A
	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	3.86	16.3	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	3.20	26.2	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	3.41	15.0	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	3.39	9.07	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	3.67	7.79	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	3.82	10.7	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	2.70	15.4	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	3.57	6.40	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	3.39	5.81	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	2.16	17.3	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	2.89	4.65	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	3.11	4.28	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	3.11	7.47	5	42	20	0.411	32.1	16.2

Table G.73 BS 4-1:2005

PARALLEL FLANGE CHANNELS

Advance UKPFC



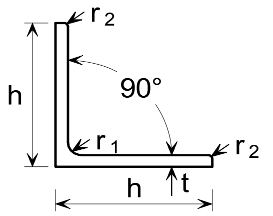


Dimensions

Section Designation	Mass per Metre	Depth of Section	Width of Section	Thickness		Root Radius	Depth between Fillets	Ratios for Local Buckling		Distance	Dimensions for Detailing			Surface Area		Area of Section
				Web	Flange			Flange	Web		End Clearance	Notch		Per Metre	Per Tonne	
		h	b	t_w	t_f	r	d	c_f / t_f	c_w / t_w	e_o	C	N	n			A
	kg/m	mm	mm	mm	mm	mm	mm			cm	mm	mm	mm	m^2	m^2	cm^2
430x100x64	64.4	430	100	11.0	19.0	15	362	3.89	32.9	3.27	13	96	36	1.23	19.0	82.1
380x100x54	54.0	380	100	9.5	17.5	15	315	4.31	33.2	3.48	12	98	34	1.13	20.9	68.7
300x100x46	45.5	300	100	9.0	16.5	15	237	4.61	26.3	3.68	11	98	32	0.969	21.3	58.0
300x90x41	41.4	300	90	9.0	15.5	12	245	4.45	27.2	3.18	11	88	28	0.932	22.5	52.7
260x90x35	34.8	260	90	8.0	14.0	12	208	5.00	26.0	3.32	10	88	28	0.854	24.5	44.4
260x75x28	27.6	260	75	7.0	12.0	12	212	4.67	30.3	2.62	9	74	26	0.796	28.8	35.1
230x90x32	32.2	230	90	7.5	14.0	12	178	5.04	23.7	3.46	10	90	28	0.795	24.7	41.0
230x75x26	25.7	230	75	6.5	12.5	12	181	4.52	27.8	2.78	9	76	26	0.737	28.7	32.7
200x90x30	29.7	200	90	7.0	14.0	12	148	5.07	21.1	3.60	9	90	28	0.736	24.8	37.9
200x75x23	23.4	200	75	6.0	12.5	12	151	4.56	25.2	2.91	8	76	26	0.678	28.9	29.9
180x90x26	26.1	180	90	6.5	12.5	12	131	5.72	20.2	3.64	9	90	26	0.697	26.7	33.2
180x75x20	20.3	180	75	6.0	10.5	12	135	5.43	22.5	2.87	8	76	24	0.638	31.4	25.9
150x90x24	23.9	150	90	6.5	12.0	12	102	5.96	15.7	3.71	9	90	26	0.637	26.7	30.4
150x75x18	17.9	150	75	5.5	10.0	12	106	5.75	19.3	2.99	8	76	24	0.579	32.4	22.8
125x65x15	14.8	125	65	5.5	9.5	12	82.0	5.00	14.9	2.56	8	66	22	0.489	33.1	18.8
100x50x10	10.2	100	50	5.0	8.5	9	65.0	4.24	13.0	1.94	7	52	18	0.382	37.5	13.0

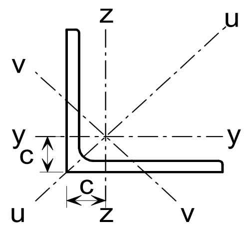
Advance and UKPFC are registered trademarks of Tata Steel. A fuller description of the relationship between Parallel Flange Channels (PFC) and the Advance range of sections manufactured by Tata Steel is given in Appendix F.
e_0 is the distance from the centre of the web to the shear centre.

Table G.74 **BS EN 10056-1:1999**



EQUAL ANGLES

Advance UKA - Equal Angles



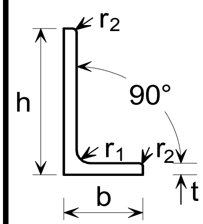
Dimensions and properties

Section Designation		Mass per	Radius		Area of	Distance to	Second Moment of Area			Radius of Gyration			Elastic Modulus	Torsional Constant	Equivalent Slenderness
Size	Thickness	Metre	Root	Toe	Section	centroid	Axis y-y, z-z	Axis u-u	Axis v-v	Axis y-y, z-z	Axis u-u	Axis v-v	Axis y-y, z-z		Coefficient
$h \times h$ mm	t mm	kg/m	r_1 mm	r_2 mm	cm^2	c cm	cm^4	cm^4	cm^4	cm	cm	cm	cm^3	I_T cm^4	ϕ_a
200x200	24	71.1	18.0	9.00	90.6	5.84	3330	5280	1380	6.06	7.64	3.90	235	182	2.50
	20	59.9	18.0	9.00	76.3	5.68	2850	4530	1170	6.11	7.70	3.92	199	107	3.05
	18	54.3	18.0	9.00	69.1	5.60	2600	4150	1050	6.13	7.75	3.90	181	78.9	3.43
	16	48.5	18.0	9.00	61.8	5.52	2340	3720	960	6.16	7.76	3.94	162	56.1	3.85
150x150	18+	40.1	16.0	8.00	51.2	4.38	1060	1680	440	4.55	5.73	2.93	99.8	58.6	2.48
	15	33.8	16.0	8.00	43.0	4.25	898	1430	370	4.57	5.76	2.93	83.5	34.6	3.01
	12	27.3	16.0	8.00	34.8	4.12	737	1170	303	4.60	5.80	2.95	67.7	18.2	3.77
	10	23.0	16.0	8.00	29.3	4.03	624	990	258	4.62	5.82	2.97	56.9	10.8	4.51
120x120	15+	26.6	13.0	6.50	34.0	3.52	448	710	186	3.63	4.57	2.34	52.8	27.0	2.37
	12	21.6	13.0	6.50	27.5	3.40	368	584	152	3.65	4.60	2.35	42.7	14.2	2.99
	10	18.2	13.0	6.50	23.2	3.31	313	497	129	3.67	4.63	2.36	36.0	8.41	3.61
	8+	14.7	13.0	6.50	18.8	3.24	259	411	107	3.71	4.67	2.38	29.5	4.44	4.56
100x100	15+	21.9	12.0	6.00	28.0	3.02	250	395	105	2.99	3.76	1.94	35.8	22.3	1.92
	12	17.8	12.0	6.00	22.7	2.90	207	328	85.7	3.02	3.80	1.94	29.1	11.8	2.44
	10	15.0	12.0	6.00	19.2	2.82	177	280	73.0	3.04	3.83	1.95	24.6	6.97	2.94
	8	12.2	12.0	6.00	15.5	2.74	145	230	59.9	3.06	3.85	1.96	19.9	3.68	3.70
90x90	12+	15.9	11.0	5.50	20.3	2.66	149	235	62.0	2.71	3.40	1.75	23.5	10.5	2.17
	10	13.4	11.0	5.50	17.1	2.58	127	201	52.6	2.72	3.42	1.75	19.8	6.20	2.64
	8	10.9	11.0	5.50	13.9	2.50	104	166	43.1	2.74	3.45	1.76	16.1	3.28	3.33
	7	9.61	11.0	5.50	12.2	2.45	92.6	147	38.3	2.75	3.46	1.77	14.1	2.24	3.80
80x80	10	11.9	10.0	5.00	15.1	2.34	87.5	139	36.4	2.41	3.03	1.55	15.4	5.45	2.33
	8	9.63	10.0	5.00	12.3	2.26	72.2	115	29.9	2.43	3.06	1.56	12.6	2.88	2.94
75x75	8	8.99	9.00	4.50	11.4	2.14	59.1	93.8	24.5	2.27	2.86	1.46	11.0	2.65	2.76
	6	6.85	9.00	4.50	8.73	2.05	45.8	72.7	18.9	2.29	2.89	1.47	8.41	1.17	3.70
70x70	7	7.38	9.00	4.50	9.40	1.97	42.3	67.1	17.5	2.12	2.67	1.36	8.41	1.69	2.92
	6	6.38	9.00	4.50	8.13	1.93	36.9	58.5	15.3	2.13	2.68	1.37	7.27	1.09	3.41
65x65	7	6.83	9.00	4.50	8.73	2.05	33.4	53.0	13.8	1.96	2.47	1.26	7.18	1.58	2.67
60x60	8	7.09	8.00	4.00	9.03	1.77	29.2	46.1	12.2	1.80	2.26	1.16	6.89	2.09	2.14
	6	5.42	8.00	4.00	6.91	1.69	22.8	36.1	9.44	1.82	2.29	1.17	5.29	0.922	2.90
	5	4.57	8.00	4.00	5.82	1.64	19.4	30.7	8.03	1.82	2.30	1.17	4.45	0.550	3.48
50x50	6	4.47	7.00	3.50	5.69	1.45	12.8	20.3	5.34	1.50	1.89	0.968	3.61	0.755	2.38
	5	3.77	7.00	3.50	4.80	1.40	11.0	17.4	4.55	1.51	1.90	0.973	3.05	0.450	2.88
	4	3.06	7.00	3.50	3.89	1.36	8.97	14.2	3.73	1.52	1.91	0.979	2.46	0.240	3.57
45x45	5	3.06	7.00	3.50	3.90	1.25	7.14	11.4	2.94	1.35	1.71	0.870	2.20	0.304	2.84
40x40	5	2.97	6.00	3.00	3.79	1.16	5.43	8.60	2.26	1.20	1.51	0.773	1.91	0.352	2.26
	4	2.42	6.00	3.00	3.08	1.12	4.47	7.09	1.86	1.21	1.52	0.777	1.55	0.188	2.83
35x35	4	2.09	5.00	2.50	2.67	1.00	2.95	4.68	1.23	1.05	1.32	0.678	1.18	0.158	2.50
30x30	4	1.78	5.00	2.50	2.27	0.878	1.80	2.85	0.754	0.892	1.12	0.577	0.850	0.137	2.07
	3	1.36	5.00	2.50	1.74	0.835	1.40	2.22	0.585	0.899	1.13	0.581	0.649	0.0613	2.75
25x25	4	1.45	3.50	1.75	1.85	0.762	1.02	1.61	0.430	0.741	0.931	0.482	0.586	0.1070	1.75
	3	1.12	3.50	1.75	1.42	0.723	0.803	1.27	0.334	0.751	0.945	0.484	0.452	0.0472	2.38
20x20	3	0.882	3.50	1.75	1.12	0.598	0.392	0.618	0.165	0.590	0.742	0.383	0.279	0.0382	1.81

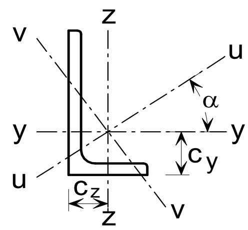
Advance and UKA are registered trademarks of Tata Steel. A fuller description of the relationship between Angles and the Advance range of sections manufactured by Tata Steel is given in Appendix H.
+ These sections are in addition to the range of BS EN 10056-1 sections.
c is the distance from the back of the leg to the centre of gravity.

Table G.75 **BS EN 10056-1:1999**

UNEQUAL ANGLES



Advance UKA - Unequal Angles



Dimensions and properties

Section Designation		Mass per	Radius		Dimension		Second Moment of Area				Radius of Gyration				Area of
Size	Thickness	Metre	Root	Toe			Axis y-y	Axis z-z	Axis u-u	Axis v-v	Axis y-y	Axis z-z	Axis u-u	Axis v-v	Section
h x *b*	*t*		r_1	r_2	c_y	c_z									
mm	mm	kg/m	mm	mm	cm	cm	cm^4	cm^4	cm^4	cm^4	cm	cm	cm	cm	cm^2
200x150	18+	47.1	15.0	7.50	6.33	3.85	2380	1150	2920	623	6.29	4.37	6.97	3.22	60.0
	15	39.6	15.0	7.50	6.21	3.73	2020	979	2480	526	6.33	4.40	7.00	3.23	50.5
	12	32.0	15.0	7.50	6.08	3.61	1650	803	2030	430	6.36	4.44	7.04	3.25	40.8
200x100	15	33.8	15.0	7.50	7.16	2.22	1760	299	1860	193	6.40	2.64	6.59	2.12	43.0
	12	27.3	15.0	7.50	7.03	2.10	1440	247	1530	159	6.43	2.67	6.63	2.14	34.8
	10	23.0	15.0	7.50	6.93	2.01	1220	210	1290	135	6.46	2.68	6.65	2.15	29.2
150x90	15	33.9	12.0	6.00	5.21	2.23	761	205	841	126	4.74	2.46	4.98	1.93	33.9
	12	21.6	12.0	6.00	5.08	2.12	627	171	694	104	4.77	2.49	5.02	1.94	27.5
	10	18.2	12.0	6.00	5.00	2.04	533	146	591	88.3	4.80	2.51	5.05	1.95	23.2
150x75	15	24.8	12.0	6.00	5.52	1.81	713	119	753	78.6	4.75	1.94	4.88	1.58	31.7
	12	20.2	12.0	6.00	5.40	1.69	588	99.6	623	64.7	4.78	1.97	4.92	1.59	25.7
	10	17.0	12.0	6.00	5.31	1.61	501	85.6	531	55.1	4.81	1.99	4.95	1.60	21.7
125x75	12	17.8	11.0	5.50	4.31	1.84	354	95.5	391	58.5	3.95	2.05	4.15	1.61	22.7
	10	15.0	11.0	5.50	4.23	1.76	302	82.1	334	49.9	3.97	2.07	4.18	1.61	19.1
	8	12.2	11.0	5.50	4.14	1.68	247	67.6	274	40.9	4.00	2.09	4.21	1.63	15.5
100x75	12	15.4	10.0	5.00	3.27	2.03	189	90.2	230	49.5	3.10	2.14	3.42	1.59	19.7
	10	13.0	10.0	5.00	3.19	1.95	162	77.6	197	42.2	3.12	2.16	3.45	1.59	16.6
	8	10.6	10.0	5.00	3.10	1.87	133	64.1	162	34.6	3.14	2.18	3.47	1.60	13.5
100x65	10+	12.3	10.0	5.00	3.36	1.63	154	51.0	175	30.1	3.14	1.81	3.35	1.39	15.6
	8+	9.94	10.0	5.00	3.27	1.55	127	42.2	144	24.8	3.16	1.83	3.37	1.40	12.7
	7+	8.77	10.0	5.00	3.23	1.51	113	37.6	128	22.0	3.17	1.83	3.39	1.40	11.2
100x50	8	8.97	8.00	4.00	3.60	1.13	116	19.7	123	12.8	3.19	1.31	3.28	1.06	11.4
	6	6.84	8.00	4.00	3.51	1.05	89.9	15.4	95.4	9.92	3.21	1.33	3.31	1.07	8.71
80x60	7	7.36	8.00	4.00	2.51	1.52	59.0	28.4	72.0	15.4	2.51	1.74	2.77	1.28	9.38
80x40	8	7.07	7.00	3.50	2.94	0.963	57.6	9.61	60.9	6.34	2.53	1.03	2.60	0.838	9.01
	6	5.41	7.00	3.50	2.85	0.884	44.9	7.59	47.6	4.93	2.55	1.05	2.63	0.845	6.89
75x50	8	7.39	7.00	3.50	2.52	1.29	52.0	18.4	59.6	10.8	2.35	1.40	2.52	1.07	9.41
	6	5.65	7.00	3.50	2.44	1.21	40.5	14.4	46.6	8.36	2.37	1.42	2.55	1.08	7.19
70x50	6	5.41	7.00	3.50	2.23	1.25	33.4	14.2	39.7	7.92	2.20	1.43	2.40	1.07	6.89
65x50	5	4.35	6.00	3.00	1.99	1.25	23.2	11.9	28.8	6.32	2.05	1.47	2.28	1.07	5.54
60x40	6	4.46	6.00	3.00	2.00	1.01	20.1	7.12	23.1	4.16	1.88	1.12	2.02	0.855	5.68
	5	3.76	6.00	3.00	1.96	0.972	17.2	6.11	19.7	3.54	1.89	1.13	2.03	0.860	4.79
60x30	5	3.36	5.00	2.50	2.17	0.684	15.6	2.63	16.5	1.71	1.91	0.784	1.97	0.633	4.28
50x30	5	2.96	5.00	2.50	1.73	0.741	9.36	2.51	10.3	1.54	1.57	0.816	1.65	0.639	3.78
45x30	4	2.25	4.50	2.25	1.48	0.740	5.78	2.05	6.65	1.18	1.42	0.850	1.52	0.640	2.87
40x25	4	1.93	4.00	2.00	1.36	0.623	3.89	1.16	4.35	0.700	1.26	0.687	1.33	0.534	2.46
40x20	4	1.77	4.00	2.00	1.47	0.480	3.59	0.600	3.80	0.393	1.26	0.514	1.30	0.417	2.26
30x20	4	1.46	4.00	2.00	1.03	0.541	1.59	0.553	1.81	0.330	0.925	0.546	0.988	0.421	1.86
	3	1.12	4.00	2.00	0.990	0.502	1.25	0.437	1.43	0.256	0.935	0.553	1.00	0.424	1.43

Advance and UKA are registered trademarks of Tata Steel. A fuller description of the relationship between Angles and the Advance range of sections manufactured by Tata Steel is given in Appendix H.

+ These sections are in addition to the range of BS EN 10056-1 sections.

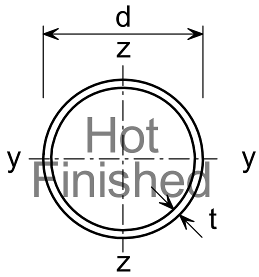
c_x is the distance from the back of the short leg to the centre of gravity.

c_y is the distance from the back of the long leg to the centre of gravity.

Table G.76 **BS EN 10056-1:1999**

HOT FINISHED CIRCULAR HOLLOW SECTIONS

Celsius CHS



Dimensions

Section Designation: Outside Diameter d mm	Section Designation: Thickness t mm	Mass per Metre kg/m	Area of Section A cm²	Surface Area: Per Metre m²	Surface Area: Per Tonne m²
21.3	2.6	1.2	1.53	0.067	834
	2.9	1.32	1.68	0.067	760
	3.2	1.43	1.82	0.067	700
26.9	2.6	1.56	1.98	0.085	642
	2.9	1.72	2.19	0.085	583
	3.2	1.87	2.38	0.085	535
	3.6	2.07	2.64	0.085	483
33.7	2.6	1.99	2.54	0.106	501
	2.9	2.2	2.81	0.106	454
	3.2	2.41	3.07	0.106	415
	3.6	2.67	3.4	0.106	374
	4.0	2.93	3.73	0.106	341
	4.5	3.24	4.13	0.106	309
	5.0	3.54	4.51	0.106	283
42.4	2.6	2.55	3.25	0.133	392
	2.9	2.82	3.6	0.133	354
	3.2	3.09	3.94	0.133	323
	3.6	3.44	4.39	0.133	290
	4.0	3.79	4.83	0.133	264
	4.5	4.21	5.36	0.133	238
	5.0	4.61	5.87	0.133	217
48.3	2.6	2.93	3.73	0.152	341
	2.9	3.25	4.14	0.152	308
	3.2	3.56	4.53	0.152	281
	3.6	3.97	5.06	0.152	252
	4.0	4.37	5.57	0.152	229
	4.5	4.86	6.19	0.152	206
	5.0	5.34	6.8	0.152	187
	5.6	5.9	7.51	0.152	170
	6.3	6.53	8.31	0.152	153
60.3	2.6	3.7	4.71	0.189	270
	2.9	4.11	5.23	0.189	244
	3.2	4.51	5.74	0.189	222
	3.6	5.03	6.41	0.189	199
	4.0	5.55	7.07	0.189	180
	4.5	6.19	7.89	0.189	161
	5.0	6.82	8.69	0.189	147
	5.6	7.55	9.62	0.189	132
	6.3	8.39	10.7	0.189	119
	8.0	10.3	13.1	0.189	96.9

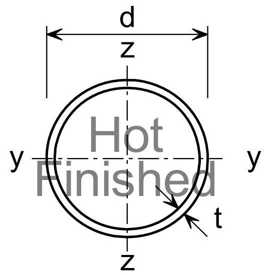
Section Designation: Outside Diameter d mm	Section Designation: Thickness t mm	Mass per Metre kg/m	Area of Section A cm²	Surface Area: Per Metre m²	Surface Area: Per Tonne m²
76.1	2.9	5.24	6.67	0.239	191
	3.2	5.75	7.33	0.239	174
	3.6	6.44	8.2	0.239	155
	4.0	7.11	9.06	0.239	141
	4.5	7.95	10.1	0.239	126
	5.0	8.77	11.2	0.239	114
	5.6	9.74	12.4	0.239	103
	6.3	10.8	13.8	0.239	92.2
	8.0	13.4	17.1	0.239	74.4
88.9	2.9	6.15	7.84	0.279	163
	3.2	6.76	8.62	0.279	148
	3.6	7.57	9.65	0.279	132
	4.0	8.38	10.7	0.279	119
	4.5	9.37	11.9	0.279	107
	5.0	10.3	13.2	0.279	96.7
	5.6	11.5	14.7	0.279	86.9
	6.3	12.8	16.3	0.279	77.9
	8.0	16	20.3	0.279	62.7
	10.0	19.5	24.8	0.279	51.4
101.6	3.2	7.77	9.89	0.319	129
	3.6	8.7	11.1	0.319	115
	4.0	9.63	12.3	0.319	104
	4.5	10.8	13.7	0.319	92.8
	5.0	11.9	15.2	0.319	84
	5.6	13.3	16.9	0.319	75.4
	6.3	14.8	18.9	0.319	67.5
	8.0	18.5	23.5	0.319	54.2
	10.0	22.6	28.8	0.319	44.3
114.3	3.2	8.77	11.2	0.359	114
	3.6	9.83	12.5	0.359	102
	4.0	10.9	13.9	0.359	91.9
	4.5	12.2	15.5	0.359	82.1
	5.0	13.5	17.2	0.359	74.2
	5.6	15	19.1	0.359	66.6
	6.3	16.8	21.4	0.359	59.6
	8.0	21	26.7	0.359	47.7
	10.0	25.7	32.8	0.359	38.9

Celsius® is a registered trademark of Tata Steel. A fuller description of the relationship between Hot Finished Circular Hollow Sections (HFCHS) and the Celsius® range of sections manufactured by Tata Steel is given in Appendix H.

Table G.76 *Continued* **BS EN 10056-1:1999**

HOT FINISHED CIRCULAR HOLLOW SECTIONS

Celsius CHS



Dimensions

Section Designation Outside Diameter *d* mm	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
139.7	3.2	10.8	13.7	0.439	92.8
	3.6	12.1	15.4	0.439	82.8
	4.0	13.4	17.1	0.439	74.7
	4.5	15	19.1	0.439	66.6
	5.0	16.6	21.2	0.439	60.2
	5.6	18.5	23.6	0.439	54
	6.3	20.7	26.4	0.439	48.2
	8.0	26	33.1	0.439	38.5
	10.0	32	40.7	0.439	31.3
	12.5	39.2	50	0.439	25.5
168.3	5.0	20.1	25.7	0.529	49.7
	5.6	22.5	28.6	0.529	44.5
	6.3	25.2	32.1	0.529	39.7
	8.0	31.6	40.3	0.529	31.6
	10.0	39	49.7	0.529	25.6
	12.5	48	61.2	0.529	20.8
193.7	5.0	23.3	29.6	0.609	43
	5.6	26	33.1	0.609	38.5
	6.3	29.1	37.1	0.609	34.3
	8.0	36.6	46.7	0.609	27.3
	10.0	45.3	57.7	0.609	22.1
	12.5	55.9	71.2	0.609	17.9
	16.0	70.1	89.3	0.609	14.3
219.1	4.5	23.8	30.3	0.688	42
	5.0	26.4	33.6	0.688	37.9
	5.6	29.5	37.6	0.688	33.9
	6.3	33.1	42.1	0.688	30.2
	8.0	41.6	53.1	0.688	24
	10.0	51.6	65.7	0.688	19.4
	12.5	63.7	81.1	0.688	15.7
	14.2	71.8	91.4	0.688	13.9
	16.0	80.1	102	0.688	12.5
244.5	5.0	29.5	37.6	0.768	33.9
	5.6	33	42	0.768	30.3
	6.3	37	47.1	0.768	27
	8.0	46.7	59.4	0.768	21.4
	10.0	57.8	73.7	0.768	17.3
	12.5	71.5	91.1	0.768	14
	14.2	80.6	103	0.768	12.4
	16.0	90.2	115	0.768	11.1

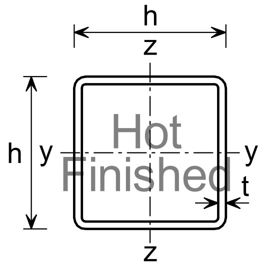
Section Designation Outside Diameter *d* mm	Thickness *t* mm	Mass per Metre kg/m	Area of Section *A* cm^2	Surface Area Per Metre m^2	Surface Area Per Tonne m^2
273.0	5.0	33	42.1	0.858	30.3
	5.6	36.9	47	0.858	27.1
	6.3	41.4	52.8	0.858	24.1
	8.0	52.3	66.6	0.858	19.1
	10.0	64.9	82.6	0.858	15.4
	12.5	80.3	102	0.858	12.5
	14.2	90.6	115	0.858	11
	16.0	101	129	0.858	9.86
323.9	5.0	39.3	50.1	1.02	25.4
	5.6	44	56	1.02	22.7
	6.3	49.3	62.9	1.02	20.3
	8.0	62.3	79.4	1.02	16
	10.0	77.4	98.6	1.02	12.9
	12.5	96	122	1.02	10.4
	14.2	108	138	1.02	9.22
	16.0	121	155	1.02	8.23
355.6	6.3	54.3	69.1	1.12	18.4
	8.0	68.6	87.4	1.12	14.6
	10.0	85.2	109	1.12	11.7
	12.5	106	135	1.12	9.45
	14.2	120	152	1.12	8.36
	16.0	134	171	1.12	7.46
406.4	6.3	62.2	79.2	1.28	16.1
	8.0	78.6	100	1.28	12.7
	10.0	97.8	125	1.28	10.2
	12.5	121	155	1.28	8.24
	14.2	137	175	1.28	7.28
	16.0	154	196	1.28	6.49
457.0	6.3	70	89.2	1.44	14.3
	8.0	88.6	113	1.44	11.3
	10.0	110	140	1.44	9.07
	12.5	137	175	1.44	7.3
	14.2	155	198	1.44	6.45
	16.0	174	222	1.44	5.75
508.0	6.3	77.9	99.3	1.6	12.8
	8.0	98.6	126	1.6	10.1
	10.0	123	156	1.6	8.14
	12.5	153	195	1.6	6.55
	14.2	173	220	1.6	5.78
	16.0	194	247	1.6	5.15

Celsius® is a registered trademark of Tata Steel. A fuller description of the relationship between Hot Finished Circular Hollow Sections (HFCHS) and the Celsius® range of sections manufactured by Tata Steel is given in Appendix H.

Table G.77 **BS EN 10056-1:1999**

HOT FINISHED SQUARE HOLLOW SECTIONS

Celsius SHS



Dimensions

Section Designation Outside Diameter *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
40 x 40	3.0	3.41	4.34	0.152	293
	3.2	3.61	4.60	0.152	277
	3.6	4.01	5.10	0.151	250
	4.0	4.39	5.59	0.150	228
	5.0	5.28	6.73	0.147	189
50 x 50	3.0	4.35	5.54	0.192	230
	3.2	4.62	5.88	0.192	217
	3.6	5.14	6.54	0.191	195
	4.0	5.64	7.19	0.190	177
	5.0	6.85	8.73	0.187	146
	6.3	8.31	10.6	0.184	120
	7.1	9.14	11.6	0.182	109
	8.0	10.0	12.8	0.179	99.9
60 x 60	3.0	5.29	6.74	0.232	189
	3.2	5.62	7.16	0.232	178
	3.6	6.27	7.98	0.231	160
	4.0	6.90	8.79	0.230	145
	5.0	8.42	10.7	0.227	119
	6.3	10.3	13.1	0.224	97.2
	7.1	11.4	14.5	0.222	88.0
	8.0	12.5	16.0	0.219	79.9
70 x 70	3.0	6.24	7.94	0.272	160
	3.2	6.63	8.44	0.272	151
	3.6	7.40	9.42	0.271	135
	4.0	8.15	10.4	0.270	123
	5.0	9.99	12.7	0.267	100
	6.3	12.3	15.6	0.264	81.5
	7.1	13.6	17.3	0.262	73.5
	8.0	15.0	19.2	0.259	66.5
	8.8	16.3	20.7	0.257	61.5

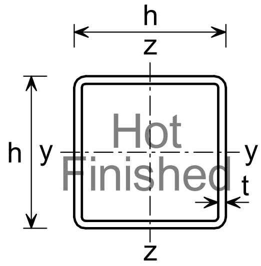
Section Designation Outside Diameter *d* mm	Section Designation Thickness *t* mm	Mass per Metre kg/m	Area of Section *A* cm^2	Surface Area Per Metre m^2	Surface Area Per Tonne m^2
80 x 80	3.0	7.18	9.14	0.312	139
	3.2	7.63	9.72	0.312	131
	3.6	8.53	10.9	0.311	117
	4.0	9.41	12.0	0.310	106
	5.0	11.6	14.7	0.307	86.5
	6.3	14.2	18.1	0.304	70.2
	7.1	15.8	20.2	0.302	63.2
	8.0	17.5	22.4	0.299	57.0
	8.8	19.0	24.2	0.297	52.6
	10.0	21.1	26.9	0.294	47.3
	12.5	25.2	32.1	0.288	39.7
90 x 90	3.6	9.66	12.3	0.351	104
	4.0	10.7	13.6	0.350	93.7
	5.0	13.1	16.7	0.347	76.1
	6.3	16.2	20.7	0.344	61.6
	7.1	18.1	23.0	0.342	55.4
	8.0	20.1	25.6	0.339	49.9
	8.8	21.8	27.8	0.337	45.9
	10.0	24.3	30.9	0.334	41.2
	12.5	29.1	37.1	0.328	34.4
100 x 100	3.6	10.8	13.7	0.391	92.7
	4.0	11.9	15.2	0.390	83.9
	5.0	14.7	18.7	0.387	68.0
	6.3	18.2	23.2	0.384	54.9
	7.1	20.3	25.8	0.382	49.3
	8.0	22.6	28.8	0.379	44.3
	8.8	24.5	31.3	0.377	40.7
	10.0	27.4	34.9	0.374	36.5
	12.5	33.0	42.1	0.368	30.3
120 x 120	4.0	14.4	18.4	0.470	69.3
	5.0	17.8	22.7	0.467	56.0
	6.3	22.2	28.2	0.464	45.1
	7.1	24.7	31.5	0.462	40.4
	8.0	27.6	35.2	0.459	36.2
	8.8	30.1	38.3	0.457	33.3
	10.0	33.7	42.9	0.454	29.7
	12.5	40.9	52.1	0.448	24.5

Celsius® is a registered trademark of Tata Steel. A fuller description of the relationship between Hot Finished Square Hollow Sections (HFSHS) and the Celsius® range of sections manufactured by Tata Steel is given in Appendix H.

Table G.77 *Continued* **BS EN 10056-1:1999**

HOT FINISHED SQUARE HOLLOW SECTIONS

Celsius SHS



Dimensions

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
140 x 140	5.0	21.0	26.7	0.547	47.7
	6.3	26.1	33.3	0.544	38.3
	7.1	29.2	37.2	0.542	34.2
	8.0	32.6	41.6	0.539	30.7
	8.8	35.6	45.4	0.537	28.1
	10.0	40.0	50.9	0.534	25.0
	12.5	48.7	62.1	0.528	20.5
150 x 150	5.0	22.6	28.7	0.587	44.3
	6.3	28.1	35.8	0.584	35.6
	7.1	31.4	40.0	0.582	31.8
	8.0	35.1	44.8	0.579	28.5
	8.8	38.4	48.9	0.577	26.1
	10.0	43.1	54.9	0.574	23.2
	12.5	52.7	67.1	0.568	19.0
	14.2	58.9	75.0	0.563	17.0
	16.0	65.2	83.0	0.559	15.3
160 x 160	5.0	24.1	30.7	0.627	41.5
	6.3	30.1	38.3	0.624	33.3
	7.1	33.7	42.9	0.622	29.7
	8.0	37.6	48.0	0.619	26.6
	8.8	41.1	52.4	0.617	24.3
	10.0	46.3	58.9	0.614	21.6
	12.5	56.6	72.1	0.608	17.7
	14.2	63.3	80.7	0.603	15.8
	16.0	70.2	89.4	0.599	14.2
180 x 180	5.0	27.3	34.7	0.707	36.7
	6.3	34.0	43.3	0.704	29.4
	7.1	38.1	48.6	0.702	26.2
	8.0	42.7	54.4	0.699	23.4
	8.8	46.7	59.4	0.697	21.4
	10.0	52.5	66.9	0.694	19.0
	12.5	64.4	82.1	0.688	15.5
	14.2	72.2	92.0	0.683	13.8
	16.0	80.2	102	0.679	12.5
200 x 200	5.0	30.4	38.7	0.787	32.9
	6.3	38.0	48.4	0.784	26.3
	7.1	42.6	54.2	0.782	23.5
	8.0	47.7	60.8	0.779	21.0
	8.8	52.2	66.5	0.777	19.2
	10.0	58.8	74.9	0.774	17.0
	12.5	72.3	92.1	0.768	13.8
	14.2	81.1	103	0.763	12.3
	16.0	90.3	115	0.759	11.1

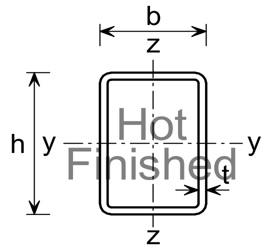
Section Designation		Mass per Metre	Area of Section	Surface Area	
Outside Diameter *d* mm	Thickness *t* mm	kg/m	*A* cm^2	Per Metre m^2	Per Tonne m^2
250 x 250	5.0	38.3	48.7	0.987	26.1
	6.3	47.9	61.0	0.984	20.9
	7.1	53.7	68.4	0.982	18.6
	8.0	60.3	76.8	0.979	16.6
	8.8	66.0	84.1	0.977	15.2
	10.0	74.5	94.9	0.974	13.4
	12.5	91.9	117	0.968	10.9
	14.2	103	132	0.963	9.67
	16.0	115	147	0.959	8.67
260 x 260	6.3	49.9	63.5	1.02	20.1
	7.1	56.0	71.3	1.02	17.9
	8.0	62.8	80.0	1.02	15.9
	8.8	68.8	87.6	1.02	14.5
	10.0	77.7	98.9	1.01	12.9
	12.5	95.8	122	1.01	10.4
	14.2	108	137	1.00	9.27
	16.0	120	153	0.999	8.30
300 x 300	6.3	57.8	73.6	1.18	17.3
	7.1	64.9	82.6	1.18	15.4
	8.0	72.8	92.8	1.18	13.7
	8.8	79.8	102	1.18	12.5
	10.0	90.2	115	1.17	11.1
	12.5	112	142	1.17	8.97
	14.2	126	160	1.16	7.95
	16.0	141	179	1.16	7.12
350 x 350	8.0	85.4	109	1.38	11.7
	8.8	93.6	119	1.38	10.7
	10.0	106	135	1.37	9.44
	12.5	131	167	1.37	7.62
	14.2	148	189	1.36	6.76
	16.0	166	211	1.36	6.04
400 x 400	8.0	97.9	125	1.58	10.2
	8.8	107	137	1.58	9.31
	10.0	122	155	1.57	8.22
	12.5	151	192	1.57	6.63
	14.2	170	217	1.56	5.87
	16.0	191	243	1.56	5.24
	20.0	235	300	1.55	4.25

Celsius® is a registered trademark of Tata Steel. A fuller description of the relationship between Hot Finished Square Hollow Sections (HFSHS) and the Celsius® range of sections manufactured by Tata Steel is given in Appendix H.

Table G.78 **BS EN 10056-1:1999**

HOT FINISHED RECTANGULAR HOLLOW SECTIONS

Celsius RHS



Dimensions

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
50 x 30	3.0	3.41	4.34	0.152	293
	3.2	3.61	4.60	0.152	277
	3.6	4.01	5.10	0.151	250
	4.0	4.39	5.59	0.150	228
	5.0	5.28	6.73	0.147	189
60 x 40	3.0	4.35	5.54	0.192	230
	3.2	4.62	5.88	0.192	217
	3.6	5.14	6.54	0.191	195
	4.0	5.64	7.19	0.190	177
	5.0	6.85	8.73	0.187	146
	6.3	8.31	10.6	0.184	120
80 x 40	3.0	5.29	6.74	0.232	189
	3.2	5.62	7.16	0.232	178
	3.6	6.27	7.98	0.231	160
	4.0	6.90	8.79	0.230	145
	5.0	8.42	10.7	0.227	119
	6.3	10.3	13.1	0.224	97.2
	7.1	11.4	14.5	0.222	88.0
	8.0	12.5	16.0	0.219	79.9
90 x 50	3.0	6.24	7.94	0.272	160
	3.2	6.63	8.44	0.272	151
	3.6	7.40	9.42	0.271	135
	4.0	8.15	10.4	0.270	123
	5.0	9.99	12.7	0.267	100
	6.3	12.3	15.6	0.264	81.5
	7.1	13.6	17.3	0.262	73.5
	8.0	15.0	19.2	0.259	66.5
100 x 50	3.0	6.71	8.54	0.292	149
	3.2	7.13	9.08	0.292	140
	3.6	7.96	10.1	0.291	126
	4.0	8.78	11.2	0.290	114
	5.0	10.8	13.7	0.287	92.8
	6.3	13.3	16.9	0.284	75.4
	7.1	14.7	18.7	0.282	68.0
	8.0	16.3	20.8	0.279	61.4
	8.8	17.6	22.5	0.277	56.7
	10.0	19.6	24.9	0.274	51.1

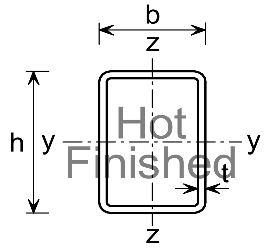
Section Designation		Mass per Metre	Area of Section	Surface Area	
Outside Diameter d mm	Thickness t mm	kg/m	A cm^2	Per Metre m^2	Per Tonne m^2
100 x 60	3.0	7.18	9.14	0.312	139
	3.2	7.63	9.72	0.312	131
	3.6	8.53	10.9	0.311	117
	4.0	9.41	12.0	0.310	106
	5.0	11.6	14.7	0.307	86.5
	6.3	14.2	18.1	0.304	70.2
	7.1	15.8	20.2	0.302	63.2
	8.0	17.5	22.4	0.299	57.0
	8.8	19.0	24.2	0.297	52.6
	10.0	21.1	26.9	0.294	47.3
120 x 60	3.0	8.12	10.3	0.352	123
	3.2	8.64	11.0	0.352	116
	3.6	9.66	12.3	0.351	104
	4.0	10.7	13.6	0.350	93.7
	5.0	13.1	16.7	0.347	76.1
	6.3	16.2	20.7	0.344	61.6
	7.1	18.1	23.0	0.342	55.4
	8.0	20.1	25.6	0.339	49.9
	8.8	21.8	27.8	0.337	45.9
	10.0	24.3	30.9	0.334	41.2
	12.5	29.1	37.1	0.328	34.4
120 x 80	3.6	10.8	13.7	0.391	92.7
	4.0	11.9	15.2	0.390	83.9
	5.0	14.7	18.7	0.387	68.0
	6.3	18.2	23.2	0.384	54.9
	7.1	20.3	25.8	0.382	49.3
	8.0	22.6	28.8	0.379	44.3
	8.8	24.5	31.3	0.377	40.7
	10.0	27.4	34.9	0.374	36.5
	12.5	33.0	42.1	0.368	30.3
150 x 100	4.0	15.1	19.2	0.490	66.4
	5.0	18.6	23.7	0.487	53.7
	6.3	23.1	29.5	0.484	43.2
	7.1	25.9	32.9	0.482	38.7
	8.0	28.9	36.8	0.479	34.7
	8.8	31.5	40.1	0.477	31.8
	10.0	35.3	44.9	0.474	28.4
	12.5	42.8	54.6	0.468	23.3

Celsius® is a registered trademark of Tata Steel. A fuller description of the relationship between Hot Finished Rectangular Hollow Sections (HFRHS) and the Celsius® range of sections manufactured by Tata Steel is given in Appendix H.

Table G.78 *Continued* **BS EN 10056-1:1999**

HOT FINISHED RECTANGULAR HOLLOW SECTIONS

Celsius RHS



Dimensions

Section Designation		Mass per	Area of	Surface Area	
Outside Diameter	Thickness	Metre	Section	Per Metre	Per Tonne
d	*t*		*A*		
mm	mm	kg/m	cm^2	m^2	m^2
160 x 80	4.0	14.4	18.4	0.470	69.3
	5.0	17.8	22.7	0.467	56.0
	6.3	22.2	28.2	0.464	45.1
	7.1	24.7	31.5	0.462	40.4
	8.0	27.6	35.2	0.459	36.2
	8.8	30.1	38.3	0.457	33.3
	10.0	33.7	42.9	0.454	29.7
	12.5	40.9	52.1	0.448	24.5
180 x 60	4.0	14.4	18.4	0.470	69.3
	5.0	17.8	22.7	0.467	56.0
	6.3	22.2	28.2	0.464	45.1
	7.1	24.7	31.5	0.462	40.4
	8.0	27.6	35.2	0.459	36.2
	8.8	30.1	38.3	0.457	33.3
	10.0	33.7	42.9	0.454	29.7
	12.5	40.9	52.1	0.448	24.5
180 x 100	4.0	16.9	21.6	0.550	59.0
	5.0	21.0	26.7	0.547	47.7
	6.3	26.1	33.3	0.544	38.3
	7.1	29.2	37.2	0.542	34.2
	8.0	32.6	41.6	0.539	30.7
	8.8	35.6	45.4	0.537	28.1
	10.0	40.0	50.9	0.534	25.0
	12.5	48.7	62.1	0.528	20.5
200 x 100	4.0	18.2	23.2	0.590	54.9
	5.0	22.6	28.7	0.587	44.3
	6.3	28.1	35.8	0.584	35.6
	7.1	31.4	40.0	0.582	31.8
	8.0	35.1	44.8	0.579	28.5
	8.8	38.4	48.9	0.577	26.1
	10.0	43.1	54.9	0.574	23.2
	12.5	52.7	67.1	0.568	19.0
	14.2	58.9	75.0	0.563	17.0
	16.0	65.2	83.0	0.559	15.3
200 x 120	5.0	24.1	30.7	0.627	41.5
	6.3	30.1	38.3	0.624	33.3
	7.1	33.7	42.9	0.622	29.7
	8.0	37.6	48.0	0.619	26.6
	8.8	41.1	52.4	0.617	24.3
	10.0	46.3	58.9	0.614	21.6
	12.5	56.6	72.1	0.608	17.7
	14.2	63.3	80.7	0.603	15.8
	16.0	70.2	89.4	0.599	14.2

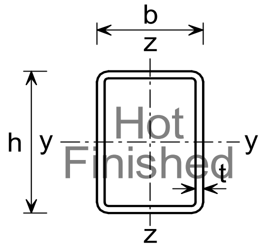
Section Designation		Mass per	Area of	Surface Area	
Outside Diameter	Thickness	Metre	Section	Per Metre	Per Tonne
d	*t*		*A*		
mm	mm	kg/m	cm^2	m^2	m^2
200 x 150	5.0	26.5	33.7	0.687	37.8
	6.3	33.0	42.1	0.684	30.3
	7.1	37.0	47.1	0.682	27.0
	8.0	41.4	52.8	0.679	24.1
	8.8	45.3	57.7	0.677	22.1
	10.0	51.0	64.9	0.674	19.6
	12.5	62.5	79.6	0.668	16.0
	14.2	70.0	89.2	0.663	14.3
	16.0	77.7	99.0	0.659	12.9
220 x 120	5.0	25.7	32.7	0.667	38.9
	6.3	32.0	40.8	0.664	31.2
	7.1	35.9	45.7	0.662	27.9
	8.0	40.2	51.2	0.659	24.9
	8.8	43.9	55.9	0.657	22.8
	10.0	49.4	62.9	0.654	20.2
	12.5	60.5	77.1	0.648	16.5
	14.2	67.8	86.3	0.643	14.8
	16.0	75.2	95.8	0.639	13.3
250 x 100	5.0	26.5	33.7	0.687	37.8
	6.3	33.0	42.1	0.684	30.3
	7.1	37.0	47.1	0.682	27.0
	8.0	41.4	52.8	0.679	24.1
	8.8	45.3	57.7	0.677	22.1
	10.0	51.0	64.9	0.674	19.6
	12.5	62.5	79.6	0.668	16.0
	14.2	70.0	89.2	0.663	14.3
	16.0	77.7	99.0	0.659	12.9
250 x 150	5.0	30.4	38.7	0.787	32.9
	6.3	38.0	48.4	0.784	26.3
	7.1	42.6	54.2	0.782	23.5
	8.0	47.7	60.8	0.779	21.0
	8.8	52.2	66.5	0.777	19.2
	10.0	58.8	74.9	0.774	17.0
	12.5	72.3	92.1	0.768	13.8
	14.2	81.1	103	0.763	12.3
	16.0	90.3	115	0.759	11.1

Celsius® is a registered trademark of Tata Steel. A fuller description of the relationship between Hot Finished Rectangular Hollow Sections (HFRHS) and the Celsius® range of sections manufactured by Tata Steel is given in Appendix H.

Table G.78 *Continued* **BS EN 10056-1:1999**

HOT FINISHED RECTANGULAR HOLLOW SECTIONS

Celsius RHS



Dimensions

Section Designation		Mass per	Area of	Surface Area	
Outside Diameter d mm	Thickness t mm	Metre kg/m	Section A cm^2	Per Metre m^2	Per Tonne m^2
260 x 140	5.0	30.4	38.7	0.787	32.9
	6.3	38.0	48.4	0.784	26.3
	7.1	42.6	54.2	0.782	23.5
	8.0	47.7	60.8	0.779	21.0
	8.8	52.2	66.5	0.777	19.2
	10.0	58.8	74.9	0.774	17.0
	12.5	72.3	92.1	0.768	13.8
	14.2	81.1	103	0.763	12.3
	16.0	90.3	115	0.759	11.1
300 x 100	5.0	30.4	38.7	0.787	32.9
	6.3	38.0	48.4	0.784	26.3
	7.1	42.6	54.2	0.782	23.5
	8.0	47.7	60.8	0.779	21.0
	8.8	52.2	66.5	0.777	19.2
	10.0	58.8	74.9	0.774	17.0
	12.5	72.3	92.1	0.768	13.8
	14.2	81.1	103	0.763	12.3
	16.0	90.3	115	0.759	11.1
300 x 150	8.0	54.0	68.8	0.879	18.5
	8.8	59.1	75.3	0.877	16.9
	10.0	66.7	84.9	0.874	15.0
	12.5	82.1	105	0.868	12.2
	14.2	92.3	118	0.863	10.8
	16.0	103	131	0.859	9.72
300 x 200	5.0	38.3	48.7	0.987	26.1
	6.3	47.9	61.0	0.984	20.9
	7.1	53.7	68.4	0.982	18.6
	8.0	60.3	76.8	0.979	16.6
	8.8	66.0	84.1	0.977	15.2
	10.0	74.5	94.9	0.974	13.4
	12.5	91.9	117	0.968	10.9
	14.2	103	132	0.963	9.67
	16.0	115	147	0.959	8.67

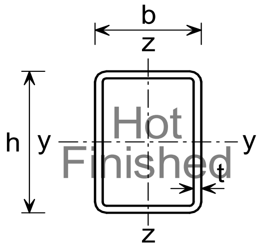
Section Designation		Mass per	Area of	Surface Area	
Outside Diameter d mm	Thickness t mm	Metre kg/m	Section A cm^2	Per Metre m^2	Per Tonne m^2
300 x 250	5.0	42.2	53.7	1.09	23.7
	6.3	52.8	67.3	1.08	18.9
	7.1	59.3	75.5	1.08	16.9
	8.0	66.5	84.8	1.08	15.0
	8.8	72.9	92.9	1.08	13.7
	10.0	82.4	105	1.07	12.1
	12.5	102	130	1.07	9.83
	14.2	115	146	1.06	8.73
	16.0	128	163	1.06	7.81
340 x 100	10.0	65.1	82.9	0.854	15.4
350 x 150	5.0	38.3	48.7	0.987	26.1
	6.3	47.9	61.0	0.984	20.9
	7.1	53.7	68.4	0.982	18.6
	8.0	60.3	76.8	0.979	16.6
	8.8	66.0	84.1	0.977	15.2
	10.0	74.5	94.9	0.974	13.4
	12.5	91.9	117	0.968	10.9
	14.2	103	132	0.963	9.67
	16.0	115	147	0.959	8.67
350 x 250	6.3	57.8	73.6	1.18	17.3
	7.1	64.9	82.6	1.18	15.4
	8.0	72.8	92.8	1.18	13.7
	8.8	79.8	102	1.18	12.5
	10.0	90.2	115	1.17	11.1
	12.5	112	142	1.17	8.97
	14.2	126	160	1.16	7.95
	16.0	141	179	1.16	7.12
400 x 150	6.3	52.8	67.3	1.08	18.9
	7.1	59.3	75.5	1.08	16.9
	8.0	66.5	84.8	1.08	15.0
	8.8	72.9	92.9	1.08	13.7
	10.0	82.4	105	1.07	12.1
	12.5	102	130	1.07	9.83
	14.2	115	146	1.06	8.73
	16.0	128	163	1.06	7.81

Celsius® is a registered trademark of Tata Steel. A fuller description of the relationship between Hot Finished Rectangular Hollow Sections (HFRHS) and the Celsius® range of sections manufactured by Tata Steel is given in Appendix H.

Table G.78 *Continued* **BS EN 10056-1:1999**

HOT FINISHED RECTANGULAR HOLLOW SECTIONS

Celsius RHS



Dimensions

Section Designation		Mass per Metre	Area of Section	Surface Area	
Outside Diameter	Thickness			Per Metre	Per Tonne
d	*t*		*A*		
mm	mm	kg/m	cm^2	m^2	m^2
400 x 200	6.3	57.8	73.6	1.18	17.3
	7.1	64.9	82.6	1.18	15.4
	8.0	72.8	92.8	1.18	13.7
	8.8	79.8	102	1.18	12.5
	10.0	90.2	115	1.17	11.1
	12.5	112	142	1.17	8.97
	14.2	126	160	1.16	7.95
	16.0	141	179	1.16	7.12
400 x 300	8.0	85.4	109	1.38	11.7
	8.8	93.6	119	1.38	10.7
	10.0	106	135	1.37	9.44
	12.5	131	167	1.37	7.62
	14.2	148	189	1.36	6.76
	16.0	166	211	1.36	6.04
450 x 250	8.0	85.4	109	1.38	11.7
	8.8	93.6	119	1.38	10.7
	10.0	106	135	1.37	9.44
	12.5	131	167	1.37	7.62
	14.2	148	189	1.36	6.76
	16.0	166	211	1.36	6.04
500 x 200	8.0	85.4	109	1.38	11.7
	8.8	93.6	119	1.38	10.7
	10.0	106	135	1.37	9.44
	12.5	131	167	1.37	7.62
	14.2	148	189	1.36	6.76
	16.0	166	211	1.36	6.04
500 x 300	8.0	97.9	125	1.58	10.2
	8.8	107	137	1.58	9.31
	10.0	122	155	1.57	8.22
	12.5	151	192	1.57	6.63
	14.2	170	217	1.56	5.87
	16.0	191	243	1.56	5.24
	20.0	235	300	1.55	4.25

Celsius® is a registered trademark of Tata Steel. A fuller description of the relationship between Hot Finished Rectangular Hollow Sections (HFRHS) and the Celsius® range of sections manufactured by Tata Steel is given in Appendix H.

APPENDIX H SECTION DESIGNATIONS

H.1 INTRODUCTION

In this publication sections are generally referred to as UKB, UKC, etc. rather than the traditional designations of UB, UC, etc. The tables of section dimensions and properties refer the “Advance” designation of open sections and “Celsius” designation for hot finished hollow sections.

H.2 OPEN SECTIONS

The Advance range of sections encompasses all the UB, UC, Tee and PFC sections in BS 4-1 and most of the angle sections in BS EN 10056-1. The dimensions and properties of the Advance sections are the same as those of the corresponding British Standard sections and the same standards for dimensional tolerance apply. The Advance range also includes additional beam and column sections that are not in BS 4 1 and angle sections not in BS EN 10056-1; these are designated by ‘+’ in the tables. These sections are manufactured to the same tolerances as those in the British Standards and the nominal dimensions may be taken as characteristic values and used in design.

The difference between Advance sections and BS sections is that the Advance sections are always CE Marked.

The table below shows the relationship between the BS 4-1 section designation and the section designation for the Advance sections.

BS designation		Tata Steel Advance designation	
Universal beam	UB*	UK Beam	UKB
Universal column	UC*	UK Column	UKC
Parallel flange channel	PFC*	UK Parallel flange channel	UKPFC
Tee		UK Tee	UKT
Equal leg angle Unequal leg angle	L	UK Angle	UKA

* These abbreviations are commonly used but are not a BS designation

Tables are included for joist sections to BS 4-1. These are not part of the Advance range.

The Advance designation is a simplified designation that encompasses the specification to BS EN 10025 and the additional quality control procedures to ensure CE Marking. It also enables a shorter form of designating the grade when ordering.

H.3 HOLLOW SECTIONS

The dimension and member resistance tables given in this publication are dual titled.

The only difference between a section to BS EN 10210-2 and its equivalent Celsius® sections is that the Tata Steel section will always be CE Marked.

The table below shows the relationship between section designations in BS EN 10210: 2006 and those for Celsius® sections produced by Tata Steel.

BS EN 10210: 2006	Tata Steel designation
Hot finished circular hollow section	Celsius® CHS
Hot finished square hollow section	Celsius® SHS
Hot finished rectangular hollow section	Celsius® RHS

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This publication covers nominally pinned joints – the most common joint type in steel building structures. Resistance tables are provided for the commonly used connection types, including partial depth end plates, fin plates, splices and column bases. Full depth end plates are also covered, which provide significant resistance to tying forces. Detailed design checks are included to cover non-standard joints and facilitate the development of design software.

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