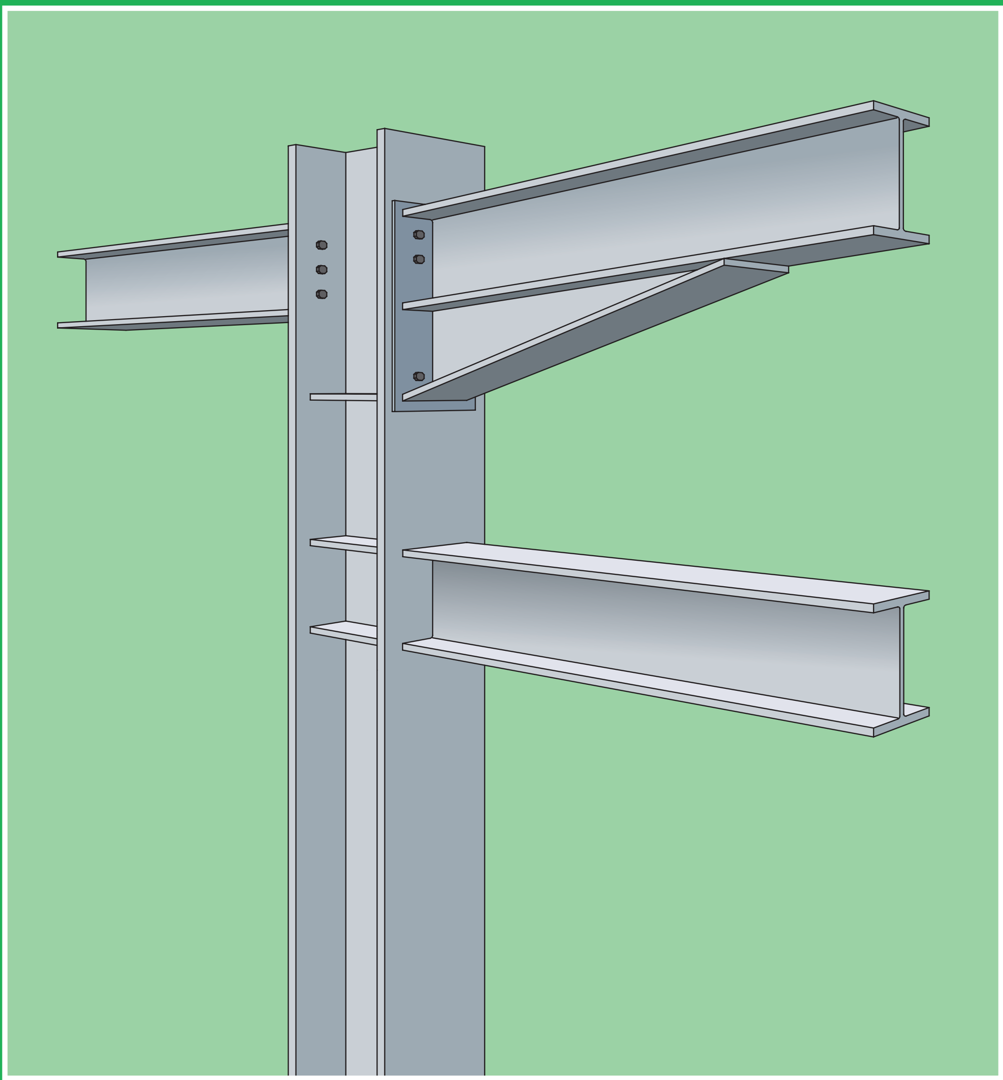
JOINTS IN STEEL CONSTRUCTION:

MOMENT-RESISTING 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 moment-resisting joints, designed in accordance with Eurocode 3 Design of steel structures, as implemented by its UK National Annexes. A companion publication, *Joints in Steel Construction: Simple Joints to Eurocode 3* (P358), covers design of nominally pinned joints.

This publication is the successor to *Joints in steel construction – Moment connections* (P207/95), which covers connections designed in accordance with BS 5950.

The major changes in scope compared to P207/95 are:

- The adoption of the published design rules in BS EN 1993-1-8 and its UK National Annex. Although most checks are almost identical, some differences will be observed, such as the modest revisions to the yield line patterns and the allowance for the effect of shear in the column web panel.
- Indicative resistances of connections are given, instead of comprehensive standardised details, recognising that software is most often used for the design of moment-resisting joints.
- The ‘hybrid’ connections, comprising welded parts and parts connected using pre-tensioned bolts, have been omitted, since they have little use in the UK.

The primary drafters of this guide were David Brown and David Iles, with assistance from Mary Brettle and Abdul Malik (all of SCI). Special thanks are due to Alan Rathbone and Robert Weeden for their comprehensive checking of the draft publication.

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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1 INTRODUCTION

1.1 ABOUT THIS DESIGN GUIDE

This publication provides guidance for designing moment-resisting joints in accordance with Eurocode 3. The publication covers:

- Bolted end plate connections between beams and columns in multi-storey frames and portal frames.
- Welded beam to column connections in multi-storey frames.
- Splices in columns and beams, including apex connections in portal frames.
- Column bases.

All connections described in procedures, examples and appendices are between I section and H section members bending about their major axes. Nevertheless, the general principles presented here can be applied to connections between other member types.

Design procedures

Design procedures are included for all the components in the above types of connection. Generally, the procedure is to calculate the design resistances for a given connection configuration, for the lowest mode of failure, and to ensure these are at least equal to the design moments and forces.

The design of moment-resisting joints can be a laborious process if undertaken by hand, especially as a number of iterations may be required to obtain the optimum connection configuration. In most cases, the connection design will be carried out using software. The procedures in this publication will serve as guidance for developing customised software and for manual checks on a completed design.

Design examples

Worked examples illustrating the design procedures are included for all the above types of moment-resisting joints

Standardisation

Although there are no standard moment-resisting joints, the principles of standardisation remain important for structural efficiency, cost-effective construction and safety. The following are generally recommended, at least for initial design:

- M20 or M24 property class 8.8 bolts, fully threaded.
- Bolts at 90 or 100 mm cross-centres (‘gauge’).
- Bolts at 90 mm vertical centres (‘pitch’).
- S275 or S355 fittings (end plates, splice plates and stiffeners).
- 20 mm end plates with M20 bolts; 25 mm end plates with M24 bolts.

Examples of typical configurations are given in Appendix A.

Steel grades

The connections described in this guide are suitable for members in steel grades up to S460.

Indicative connection resistances

To facilitate, at an early stage in the design, an assessment of whether the calculated design moment at a joint can be transferred by a reasonably sized connection, indicative connection resistances are provided in Appendix B.

1.2 EUROCODE 3

Design of connections in steel structures in the UK is covered by BS EN 1993-1-8[1] and its National Annex[2].

The following partial factors are defined in the UK National Annex (UK NA). The worked examples and indicative resistances in Appendix B have used these values.

Table 1.1 Partial factors in NA to BS EN 1993-1-8

Partial Factor	Value	Comment
γ_{M2}	1.25	Used for the resistance of bolts and welds
γ_{M2}	1.25	Used for the resistance of plates in bearing*
γ_{M3}	1.25	Used for slip resistance at ULS
$\gamma_{M3,ser}$	1.1	Used for slip resistance at SLS

*γ_{M2} = 1.5 should be used if deformation control is important but this is not generally necessary for connections covered in this publication.

The resistance of members and sections, and the local buckling resistance of components such as splice cover plates, is given by BS EN 1993-1-1. The UK NA defines the following partial factors:

Table 1.2 Partial factors in NA to BS EN 1993-1-1

Partial Factor	Value	Comment
γ_{M0}	1.0	Used for the resistance of sections
γ_{M1}	1.0	Used for buckling resistance
γ_{M2}	1.1	Used for the resistance of net sections in tension

1.3 JOINT CLASSIFICATION

BS EN 1993-1-8 requires that joints are classified by stiffness (as rigid, semi-rigid or nominally pinned) and by strength (as full strength, partial strength or nominally pinned). The stiffness classification is relevant for elastic analysis of frames; the strength classification is for frames analysed plastically. The Standard defines joint models as simple, semi-continuous or continuous, depending on stiffness and strength. Moment-resisting joints will usually be rigid and either full or partial strength and thus the joints are either continuous or semi-continuous.

In most situations, the design intent would be that moment-resisting joints are rigid, and modelled as such in the frame analysis*. If the joints were in fact semi-rigid, the behaviour of the joint would need to be taken into account in the frame analysis but the UK NA discourages this approach until experience is gained with the numerical method of calculating rotational stiffness.

Clause 5.2.2.1(2) of the Standard notes that a joint may be classified on the basis of experimental evidence, experience of previous satisfactory performance in similar cases or by calculations based on test evidence.

The UK NA offers further clarification, and in NA.2.6 comments that connections designed in accordance with the previous version of this publication[3] may be classified in accordance with the recommendations in that publication. It is expected that the reference will in due course be updated to refer to the present publication, which provides equivalent guidance on classification below.

Rigid joint classification

Well-proportioned connections that follow the recommendations for standardisation given in this guide and designed for strength alone can generally be assumed to be rigid for joints in braced frames and single-storey portal frames. For multi-storey unbraced frames, joint rotational stiffness is fundamental to the determination of frame stability. The designer must therefore either evaluate joint stiffness (in accordance with BS EN 1993-1-8) and account for this in the frame design and assessment of frame stability or, if rigid joints have been assumed in the frame analysis, ensure that the joint design matches this assumption.

For an end plate connection, it may be assumed that the connection is rigid if both the following requirements are satisfied:

- Mode 3 (see Section 2.5, STEP 1) is the critical mode for the top row of bolts. This will mean adopting relatively thick end plates and may mean that the column flange has to be stiffened.
- The column web panel shear force does not exceed 80% of the design shear resistance. If this is not possible, a stronger column should be used, or suitable strengthening should be provided.

Semi-rigid joint classification

Where a rigid joint cannot be assumed, the joint should be assumed to be semi-rigid.

1.4 COSTS

Moment-resisting joints are invariably more expensive to fabricate than simple (shear only) connections. Although the material cost of the components in the connection (the plates, the bolts etc.) may not be significant, moment-resisting joints generally have much more welding than other connections. Welding is an expensive operation and also involves inspection after completing the welds.

Local strengthening adds further expense: increasing the resistance of the main members should always be considered as a cost-effective alternative. Local strengthening often makes the connections to the minor axis more difficult to achieve, adding further cost.

Haunches involve a large amount of welding and are therefore expensive. When used to increase the resistance of the member, such as in a portal frame rafter, their use is justified, but haunches can be an expensive option if provided only to make a bolted connection feasible.

The indicative connection resistances provided in Appendix B allow designers to make a rapid assessment of the resistance of connections without haunches.

* In multi-storey unbraced frames, the sensitivity to second order effects depends not only on the stiffness of the beams and columns but also on the stiffness of the joints. If the joints are considered to be rigid when calculating sensitivity to second order effects (measured by α_{cr}), this assumption must be realised in the joint details.

1.5 MAJOR SYMBOLS

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

A_s tensile stress area of a bolt

a effective throat thickness of a fillet weld (subscript c refers to column web/flange weld, b refers to beam to column weld and p refers to beam to end plate weld)

b section breadth (subscript c or b refers to column or beam)

d_0 hole diameter

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

e distance from the centre of a fastener to the nearest edge (subscripts are defined for the particular use)

f_y yield strength of an element (subscript fb,fc or p refers to beam flange, column flange or end plate)

f_u ultimate strength of an element (subscript fb,fc or p refers to beam flange, column flange or end plate)

f_{ub} ultimate tensile strength of a bolt

$F_{b,Rd}$ design bearing resistance of a bolt

$F_{v,Rd}$ design shear resistance of a bolt

$F_{t,Rd}$ design tension resistance of a bolt

$F_{t,Ed}$ design tensile force per bolt at ULS

$F_{v,Ed}$ design shear force per bolt at ULS

h section height (subscript c or b refers to column or beam)

m distance from the centre of a fastener to a fillet weld or to the radius of a rolled section fillet (in both cases, measured to a distance into the fillet equal to 20% of its size)

p spacing between centres of fasteners ('pitch' - subscripts are defined for the particular use))

w horizontal spacing between lines of bolts in an end plate connection ('gauge')

$M_{j,Rd}$ design moment resistance of a joint

r root radius of a rolled section

s leg length of a fillet weld ($s = \sqrt{2}a$ for symmetric weld between two parts at right angles); stiff bearing length (or part thereof)

t_f thickness of flange (subscript c or b refers to column or beam)

t_w thickness of web (subscript c or b refers to column or beam)

t_p thickness of plate, or packing

W elastic modulus (subscript b refers to bolt group modulus)

Lengths and thicknesses stated without units are in millimetres.

2 BOLTED BEAM TO COLUMN CONNECTIONS

2.1 SCOPE

This Section covers the design of bolted end plate connections between I section or H section beams and columns such as those shown in Figure 2.1. The design approach follows that described in BS EN 1993-1-8. Bolted end plate splices and apex connections, which use similar design procedures, are covered in Section 4.3.

2.2 DESIGN BASIS

The resistance of a bolted end plate connection is provided by a combination of tension forces in the bolts adjacent to one flange and compression forces in bearing at the other flange. Unless there is axial force in the beam, the total tension and compression forces are equal and opposite. Vertical shear is resisted by bolts in bearing and shear; the force is usually assumed to be resisted mainly by bolts adjacent to the compression flange. These forces are illustrated diagrammatically in Figure 2.2.

At the ultimate limit state, the centre of rotation is at, or near, the compression flange and, for simplicity in design, it may be assumed that the compression resistance is concentrated at the level of the centre of the flange.

The bolt row furthest from the compression flange will tend to attract the greatest tension force and design practice in the past has been to assume a ‘triangular’ distribution of forces, pro rata to the distance from the bottom flange. However, where either the column flange or the end plate is sufficiently flexible (as defined by NA.2.7 of the UK NA) that a ductile failure mode is achieved, the full resistances of the lower rows may be used (this is sometimes referred to as a plastic distribution of bolt row forces).

2.3 DESIGN METHOD

The full design method for an end plate connection is necessarily an iterative procedure: a configuration of bolts and, if necessary, stiffeners is selected; the resistance of that configuration is evaluated; the configuration is modified for greater resistance or greater economy, as appropriate; the revised configuration is re-evaluated, until a satisfactory solution is achieved.

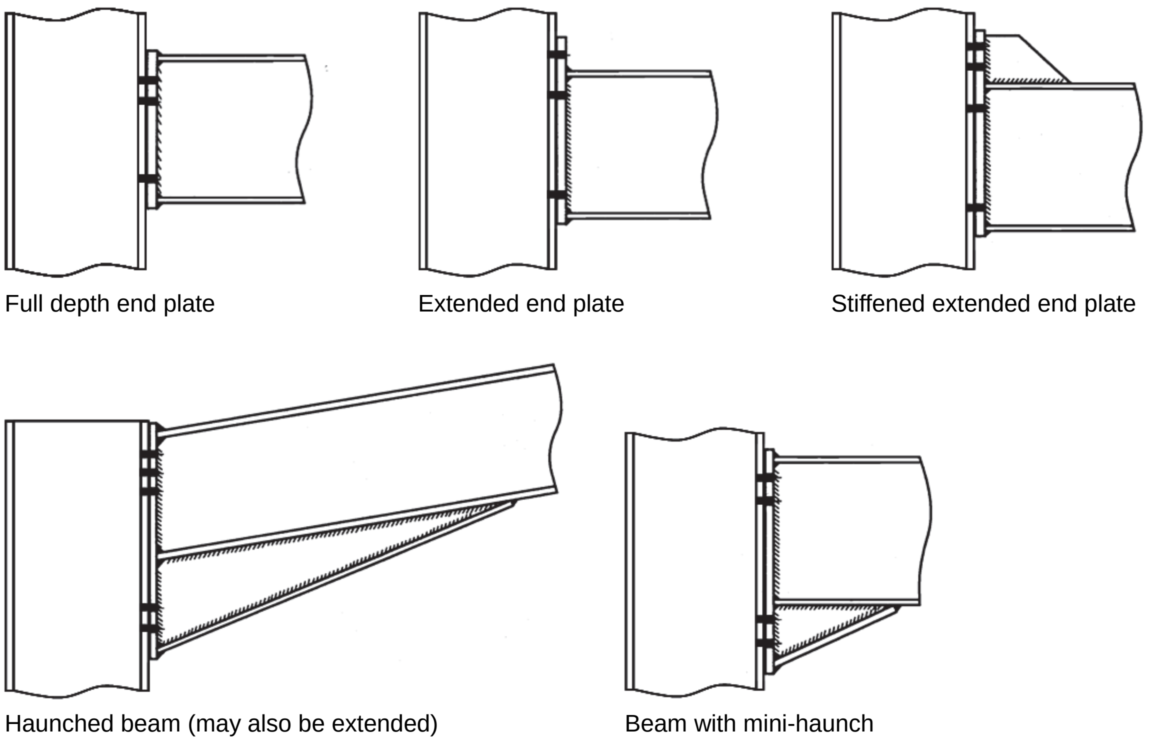


Figure 2.1 Typical bolted end plate beam to column connections

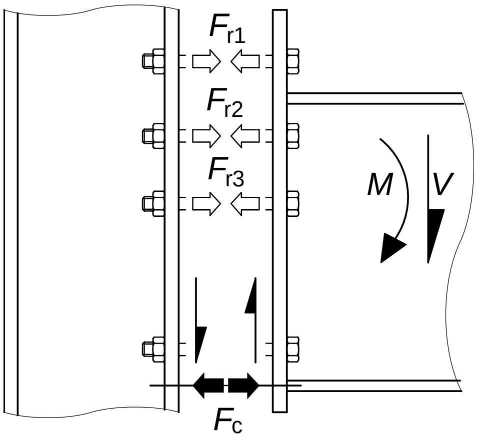


Figure 2.2 Forces in an end plate connection

The verification of the resistance of an end plate connection is summarised in the seven STEPS outlined below.

STEP 1

Calculate the effective tension resistances of the bolt rows. This involves calculating the resistance of the bolts, the end plate, the column flange, the beam web and the column web. The effective resistance for any row may be that for the row in isolation, or as part of a group of rows, or may be limited by a 'triangular' distribution from compression flange level.

The conclusion of this stage is a set of effective tension resistances, one value for each bolt row, and the summation of all bolt rows to give the total resistance of the tension zone. (These resistances may need to be reduced in STEP 4.)

STEP 2

Calculate the resistances of the compression zone of the joint, considering the column web and the beam flange.

STEP 3

Calculate the shear resistance of the column web. (Note: the influence of the shear force in the column web on the resistances of the tension and compression zones will already have been taken into account in STEPS 1 and 2.)

STEP 4

Calculate the 'final' set of tension resistances for the bolt rows, reducing the effective resistances (calculated in STEP 1) where necessary in order to ensure equilibrium (if the total effective tension resistance exceeds the compression resistance calculated in STEP 2) or to match the limiting column web panel shear resistance calculated in STEP 3.

Calculate the moment resistance. This is the summation of the products of bolt row force multiplied by its respective lever arm, calculated from the centre of compression.

STEP 5

Calculate the shear resistance of the bolt rows. The resistance is taken as the sum of the full shear resistance of the bottom row (or rows) of bolts (which are not assumed to resist tension) and 28% of the shear resistance of the bolts in the tension zone (assuming, conservatively, that they are fully utilised in tension).

STEP 6

Verify the adequacy of any stiffeners in the configuration. See Section 2.4 for types of strengthening covered in this guide.

STEP 7

Verify the adequacy of the welds in the connection. (Note that welds sizes are not critical in the preceding STEPS but they do affect the values of m and if the assumed weld sizes need to be modified, the values calculated in previous STEPS will need to be re-evaluated).

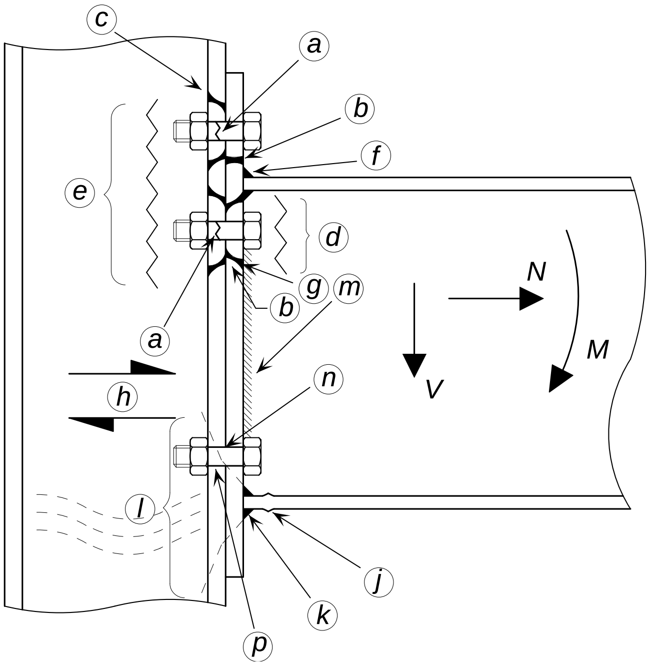
Components in compression in direct bearing need only a nominal weld, unless moment reversal must be considered.

The above STEPS involve the determination of resistance values of 14 distinct components of an end plate connection. These components are illustrated in Figure 2.3.

For haunched beams, an additional STEP is required:

STEP 8

Verify the adequacy of the welds connecting the haunched portion to the beam and the adequacy of the beam web to resist the transverse force at the end of the haunch.



ZONE	REF	COMPONENT	Procedure
TENSION	a	Bolt tension	STEP lA
	b	End plate bending	STEP lA
	c	Column flange bending	STEP lA
	d	Beam web tension	STEP IB
	e	Column web tension	STEP IB
	f	Flange to end plate weld	STEP 7
	g	Web to end plate weld	STEP 7
HORIZONTAL SHEAR	h	Column web panel shear	STEP 3
COMPRESSION	j	Beam flange compression	STEP 2
	k	Beam flange weld	STEP 7
	l	Column web	STEP 2
VERTICAL SHEAR	m	Web to end plate weld	STEP 7
	n	Bolt shear	STEP 5
	p	Bolt bearing (plate or flange)	STEP 5

Figure 2.3 Joint components to be evaluated

2.4 METHODS OF STRENGTHENING

Careful selection of the members during design will often avoid the need for strengthening of the joint and will lead to a more cost-efficient structure. Sometimes, there is no alternative to strengthening one or more of the connection zones. The range of stiffeners which can be employed is indicated in Figure 2.4.

The type of strengthening must be chosen such that it does not clash with other components at the connection. This is often a problem with conventional stiffeners when secondary beams connect into the column web.

There are usually several ways of strengthening each zone and many of them can contribute to overcoming a deficiency in more than one area, as shown in Table 2.1.

Table 2.1 Methods of strengthening columns

TYPE OF COLUMN STIFFENER	DEFICIENCY			
	Web in tension	Flange in bending	Web in compression	Web in shear
Horizontal stiffeners				
Full depth	•	•	•	
Partial depth	•	•	•	
Supplementary web plates	•		•	•
Diagonal stiffeners (N & K)	•	•		•
Morris stiffeners	•	•		•
Flange backing plates		•		

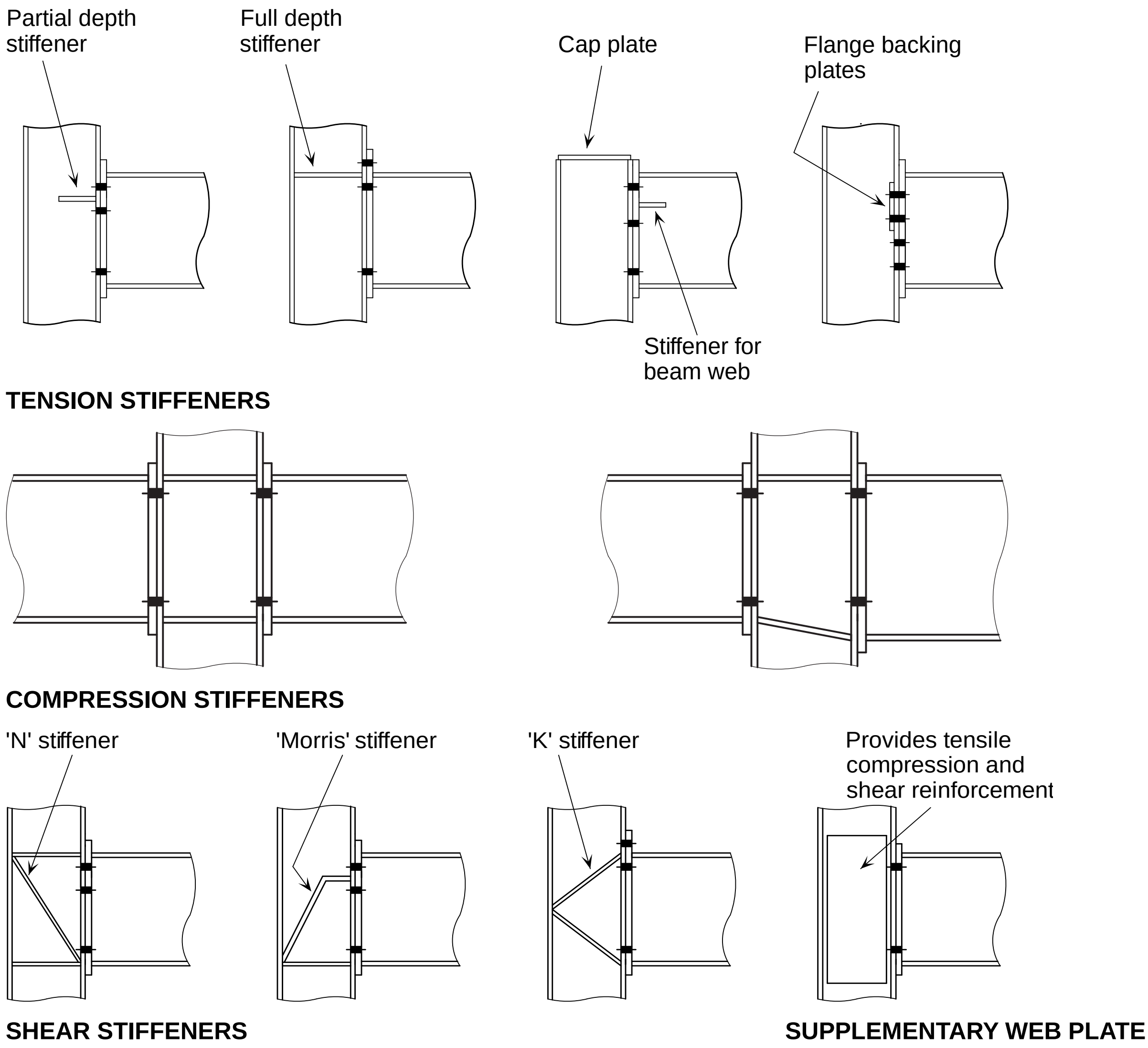
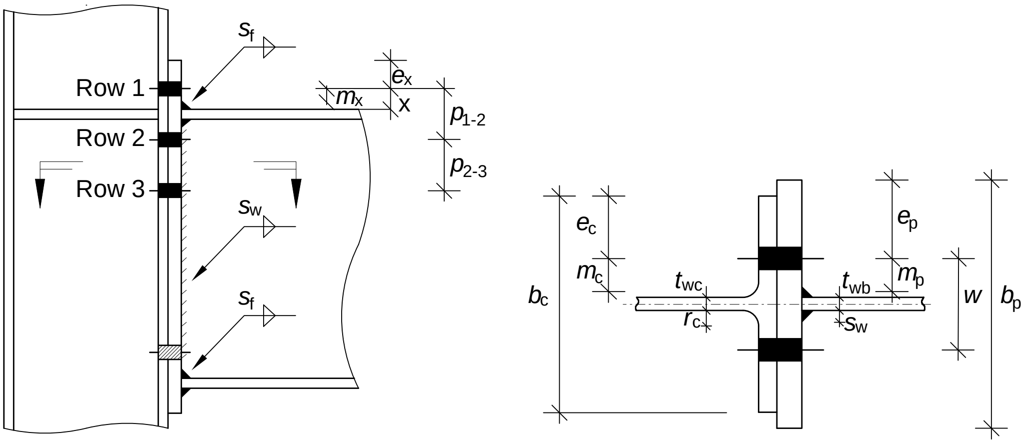


Figure 2.4 Methods of strengthening

2.5 DESIGN STEPS

The following pages set out the details of the eight Design STEPS described above. Worked examples illustrating the procedures are given in Appendix C.

The connection geometry for an end plate connection with three rows of bolts in the tension zone is shown in Figure 2.5. The geometry for a haunched connection in a portal frame would be similar, although the beam would usually be at a slope and there would be more bolt rows.



For the end plate:

$$m_p = \frac{w}{2} - \frac{t_{wb}}{2} - 0.8s$$

$$e_p = \frac{b_p}{2} - \frac{w}{2}$$

For the column flange:

$$m_c = \frac{w}{2} - \frac{t_{wc}}{2} - 0.8r_c$$

$$e_c = \frac{b_c}{2} - \frac{w}{2}$$

where:

w	is the horizontal distance between bolt centrelines (gauge)
b_p	is the end plate width
b_c	is the column flange width
t_{wb}	is the beam web thickness
t_{wc}	is the column web thickness
s	is the weld leg length ($s = \sqrt{2}a$, where a is the weld throat) (subscripts f and w refer to the flange and the web welds respectively)
r_c	is the fillet radius of the rolled section (for a welded column section use s, the weld leg length)

For the end plate extension only:

$m_x = x - 0.8s_f$

Adjacent to a flange or stiffener:

m_2 is calculated in a similar way to m_x, above. m_2 is the distance to the face of the flange or stiffener, less 0.8 of the weld leg length.

Note: dimensions m and e, used without subscripts, will commonly differ between column and beam sides

Figure 2.5 Connection geometry

STEP 1 RESISTANCES OF BOLT ROWS IN THE TENSION ZONE

GENERAL

The effective design tension resistance for each row of bolts in the tension zone is limited by the least resistance of the following:

- Bending in the end plate.
- Bending in the column flange.
- Tension in the beam web.
- Tension in the column web.

Additionally, the resistance of a group of several rows may be less than the sum of the resistances of the individual rows because different failure modes apply.

Resistances of Individual Rows

The procedure is first to calculate the resistance for each individual row, F_{ri}. For a typical connection with three bolt rows, the values F_{r1}, $F_{r2,}$ F_{r3} etc. are calculated in turn, starting at the top (row 1) and working down. At this stage, the presence of all the other rows is ignored.

The detailed procedure for each element is given in:

- Column flange bending/bolt failure......... STEP 1A
- End plate bending/bolt failure................. STEP 1A
- Column web in tension .... STEP 1B
- Beam web in tension .... STEP 1B

For each bolt row, an effective length of equivalent T-stub is determined, for each of the possible yield line patterns shown in Table 2.2 that are relevant to the location of the fastener, and the design resistance of each element is calculated. The effective design resistance of the row is the lowest of the resistances calculated for the beam and column sides of the connection.

Resistances of Groups of Rows

As well as determining the effective resistance of individual rows, the resistances of groups of bolt rows are evaluated, using the same procedures. The effective lengths of equivalent T-stubs for groups of rows are given by the yield line patterns in Table 2.3. The effective design resistance of the group of rows is the lowest of the resistances calculated for the beam and column sides of the connection. If rows are separated by a flange or stiffener, no behaviour as a combined group is possible and the resistance of the group is not evaluated.

Effective Resistances of Rows

For rows not separated by a flange or stiffener, the bolt rows are usually sufficiently close together that the resistance of a group of rows will be limited by a group failure mode. In such cases, to maximise the bending resistance provided by the rows in tension, it is assumed that the highest row provides the resistance that it would as an individual row and that lower rows in the group provide only the additional resistance that each row contributes as it is added to the group.

The procedure for determining these reduced effective design resistances of the rows may be summarised as follows:

$F_{t1,Rd}$ = [resistance of row 1 alone]

$F_{t2,Rd}$ = lesser of:

$$\begin{bmatrix} \text{resistance of row 2 alone} \\ (\text{resistance of rows } 2+1) - F_{t1,Rd} \end{bmatrix}$$

$F_{t3,Rd}$ = least of:

$$\begin{bmatrix} \text{resistance of row 3 alone} \\ (\text{resistance of rows } 3+2) - F_{t2,Rd} \\ (\text{resistance of rows } 3+2+1) - F_{t2,Rd} - F_{t1,Rd} \end{bmatrix}$$

and in a similar manner for subsequent rows.

The process is therefore to establish the resistance of a row, individually or as part of a group, before considering the next (lower) row.

Limitation to Triangular Distribution

Additionally, if the failure mode for any row is not ductile, the effective design resistances of lower rows will need to be limited to a ‘triangular’ distribution – see STEP 1C.

STEP 1 RESISTANCES OF BOLT ROWS IN THE TENSION ZONE

RESISTANCES OF T-STUBS

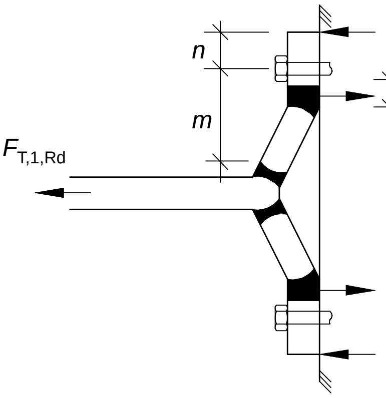
The resistances of the equivalent T-stubs are evaluated separately for the end plate and the column flange. The resistances are calculated for three possible modes of failure. The resistance is taken as the minimum of the values for the three modes.

The design resistance of the T-stub flange, for each of the modes, is given below.

Mode 1 Complete Flange Yielding

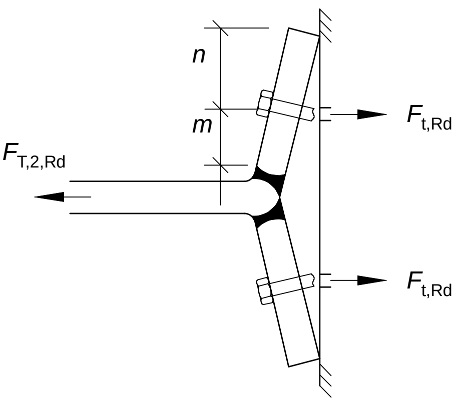
Using 'Method 2' in Table 6.2 of BS EN 1993-1-8:

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



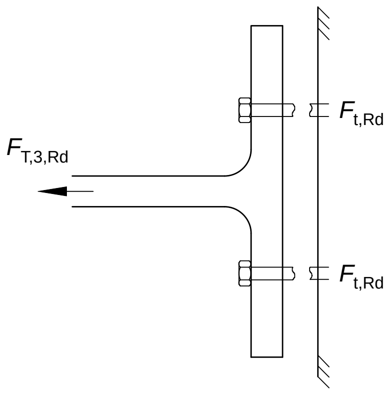
Mode 2 Bolt Failure with Flange Yielding

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



Mode 3 Bolt Failure

$$F_{\mathrm{T,3,Rd}} = \Sigma F_{\mathrm{t,Rd}}$$



where:

$M_{\mathrm{pl,1,Rd}}$ and $M_{\mathrm{pl,2,Rd}}$ are the plastic resistance moments of the equivalent T-stubs for Modes 1 and 2, given by:

$$M_{\mathrm{pl,1,Rd}} = 0.25\,\Sigma \ell_{\mathrm{eff,1}} t_{\mathrm{f}}^2 f_{\mathrm{y}} / \gamma_{\mathrm{M0}}$$

$$M_{\mathrm{pl,2,Rd}} = 0.25\,\Sigma \ell_{\mathrm{eff,2}} t_{\mathrm{f}}^2 f_{\mathrm{y}} / \gamma_{\mathrm{M0}}$$

$\ell_{\mathrm{eff,1}}$ is the effective length of the equivalent T-stub for Mode 1, taken as the lesser of $\ell_{\mathrm{eff,cp}}$ and $\ell_{\mathrm{eff,nc}}$ (see Table 2.2 for effective lengths for individual rows and Table 2.3 for groups of rows)

$\ell_{\mathrm{eff,2}}$ is the effective length of the equivalent T-stub for Mode 2, taken as $\ell_{\mathrm{eff,nc}}$ (see Table 2.2 for effective lengths for individual rows and Table 2.3 for groups of rows)

t_{f} is the thickness of the T-stub flange (= t_{p} or t_{fc})

f_{y} is the yield strength of the T-stub flange (i.e. of the column or end plate)

$\Sigma F_{\mathrm{t,Rd}}$ is the total tension resistance for the bolts in the T-stub (= $2F_{\mathrm{t,Rd}}$ for a single row)

e_{w} = d_{w} / 4

d_{w} is the diameter of the washer or the width across the points of the bolt head, as relevant

m is as defined in Figure 2.5

n is the minimum of:

e_{c} (edge distance of the column flange)

e_{p} (edge distance of the end plate)

1.25m (for end plate or column flange, as appropriate)

STEP 1 RESISTANCES OF BOLT ROWS IN THE TENSION ZONE

Backing Plates

For small section columns with thin flanges, loose backing plates can increase the Mode 1 resistance of the column flange. Design procedures for backing plates are given in STEP 6E.

Stiffeners

The presence of web stiffeners on the column web and the position of the beam flange on the end plate will influence the effective lengths of the equivalent T-stubs. Stiffeners (or the beam flange) will prevent a group mode of failure from extending across the line of attachment on that side of the connection. To influence the effective lengths, the width of stiffener or cap plate should be wider than the gauge and comply with:

$b \geq 1.33w$

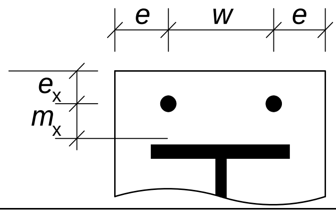
where b is the width of beam, cap plate or overall width of a pair of stiffeners.

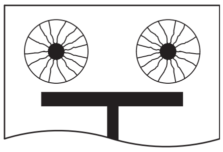
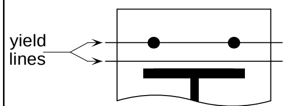
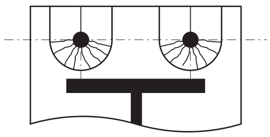
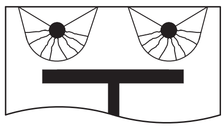
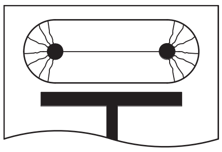
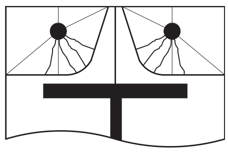
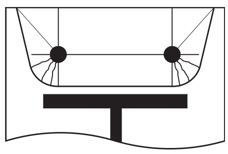
STEP 1A T-STUB FLANGE IN BENDING

Table 2.2 Effective lengths ℓ_{eff} for equivalent T-stubs for bolt row acting alone

(a) Pair of bolts in an unstiffened end plate extension

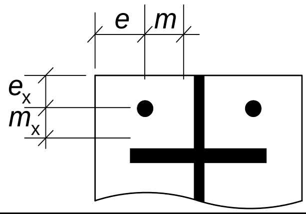
Note: Use m_x in place of m and e_x in place of n in the expressions for $F_{T,1,Rd}$ and $F_{T,2,Rd}$.

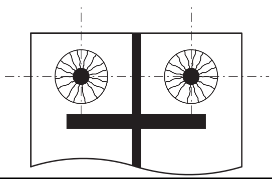
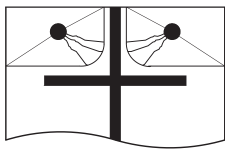
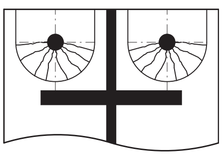
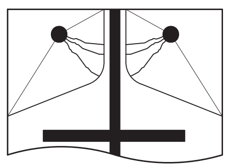


Circular patterns		Non-circular patterns	
	Circular yielding $\ell_{eff,cp} = 2\pi m_x$		Double curvature $\ell_{eff,nc} = \frac{b_p}{2}$
	Individual end yielding $\ell_{eff,cp} = \pi m_x + 2e_x$		Individual end yielding $\ell_{eff,nc} = 4m_x + 1.25e_x$
	Circular group yielding $\ell_{eff,cp} = \pi m_x + w$		Corner yielding $\ell_{eff,nc} = 2m_x + 0.625e_x + e$
			Group end yielding $\ell_{eff,nc} = 2m_x + 0.625e_x + \frac{w}{2}$

(b) Pair of bolts at end of column or on a stiffened end plate extension

Note: The expressions below may also be used for a column without a stiffener except that the corner yielding pattern is not applicable.



Circular patterns		Non-circular patterns	
	Circular yielding $\ell_{eff,cp} = 2\pi m$		Corner yielding $\ell_{eff,nc} = \alpha m - (2m + 0.625e) + e_x$
	Individual end yielding, $\ell_{eff,cp} = \pi m + 2e_x$		Corner yielding away from the stiffener/flange (m_x large) $\ell_{eff,nc} = 2m + 0.625e + e_x$

See Notes on Page 14

STEP 1A T-STUB FLANGE IN BENDING

Table 2.2 (continued)

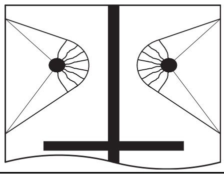
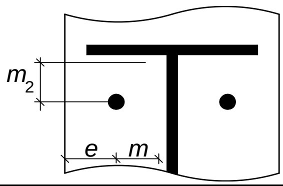
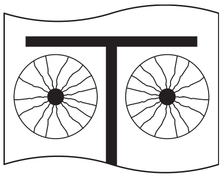
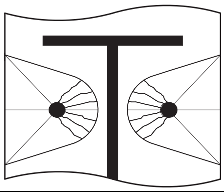
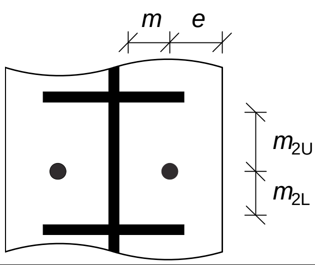
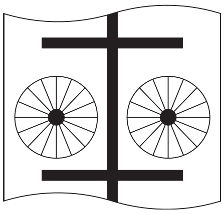
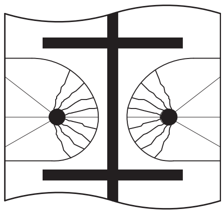
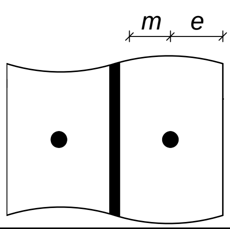
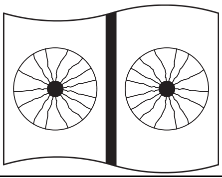
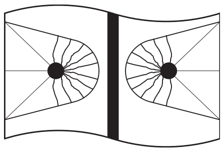
			Side yielding (m_x and e_x large) $\ell_{eff,nc} = 4m + 1.25e$
(c) Pair of bolts in a column flange below a stiffener (or cap plate) or in an end plate below the beam flange			
Circular patterns		Non-circular patterns	
	Circular yielding $\ell_{eff,cp} = 2\pi m$		Side yielding near beam flange or a stiffener $\ell_{eff,nc} = \alpha m$
(d) Pair of bolts in a column flange between two stiffeners or in an end plate between stiffeners and beam flange *NB This pattern is not included in BS EN 1993-1-8*			
Circular patterns		Non-circular patterns	
	Circular yielding $\ell_{eff} = 2\pi m$		Side yielding between two stiffeners $\ell_{eff} = \alpha m + \alpha' m - (4m + 1.25e)$ α is calculated using m_{2U} α' is calculated using m_{2L}
See Notes on Page 14			

Table 2.2 (continued)

(e) Pair of bolts in a column flange away from any stiffener or in an end plate, away from the flange or any stiffener			
Circular patterns		Non-circular patterns	

STEP 1A T-STUB FLANGE IN BENDING

	Circular yielding $\ell_{eff} = 2\pi m$		Side yielding $\ell_{eff} = 4m + 1.25e$

Notes:
For each of the situations above, where there is more than one pattern of a type (circular or non-circular), use the smallest value of $\ell_{eff,cp}$ or $\ell_{eff,nc}$ as appropriate (see Tables 6.4, 6.5 and 6.6 of BS EN 1993-1-8).

The value of α depends on dimensions m and m_2 and is determined from the chart in Appendix G.

L_{eff} is the length of the equivalent T-stub, not the length of the pattern shown.

For consideration as an effective stiffener or flange in restricting the yield line patterns, see the limiting minimum value of b in STEP 1.

STEP 1A T-STUB FLANGE IN BENDING

Table 2.3 Effective lengths for bolt rows acting in combination

Where there is no stiffener (or beam flange) between rows, a yield line pattern can develop around that group of bolts. This is referred to as rows acting in combination. When there is a stiffener or flange on the side considered that falls within a group, the resistance of that group is not evaluated.

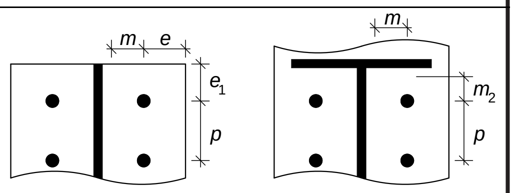
When rows act in combination, the group comprises a top row, one or more middle rows (if there are at least three rows in the group) and a bottom row. The effective length for the group is the summation of the lengths for each of the rows as part of a group. The lengths are given below.

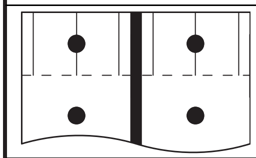
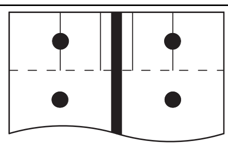
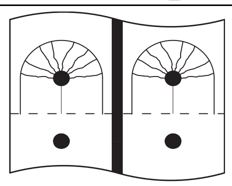
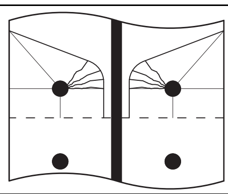
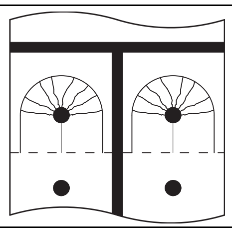
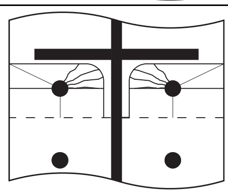
(a) Top row

In an unstiffened column

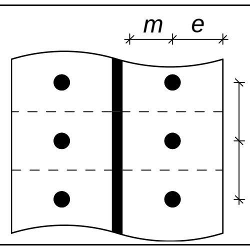
In a stiffened extended end plate (when there is more than one row)

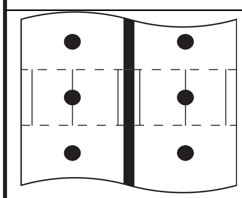
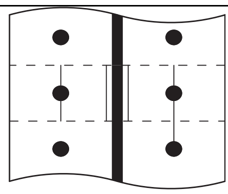
Below a column stiffener or below a beam flange



Circular patterns		Non-circular patterns	
	Close to a free edge $\ell_{eff,cp} = 2e_1 + p$		Close to a free edge $\ell_{eff,nc} = e_1 + 0.5p$
	Away from a free edge $\ell_{eff,cp} = \pi m + p$		Away from a free edge $\ell_{eff,nc} = 2m + 0.625e + 0.5p$
	Close to stiffener/flange $\ell_{eff,cp} = \pi m + p$		Close to stiffener/flange $\ell_{eff,nc} = \alpha m - (2m + 0.625e) + 0.5p$

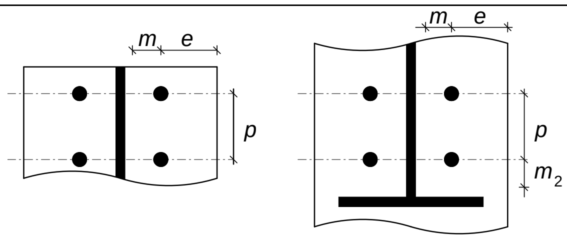
(b) Internal row

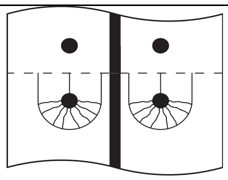
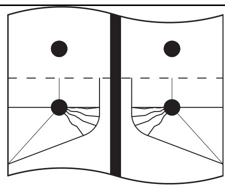
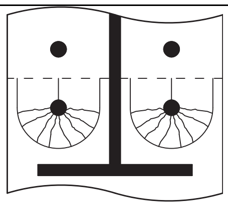
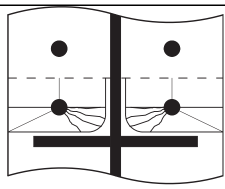


Circular patterns		Non-circular patterns	
	$\ell_{eff,cp} = 2p$		$\ell_{eff,nc} = p$

STEP 1A T-STUB FLANGE IN BENDING

Table 2.3 (continued)

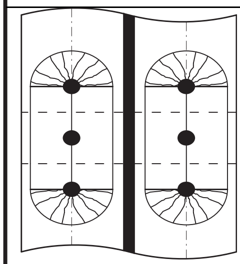
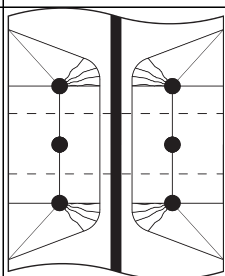
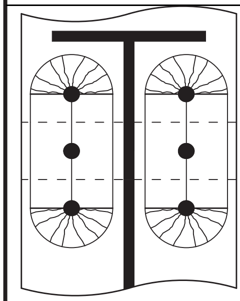
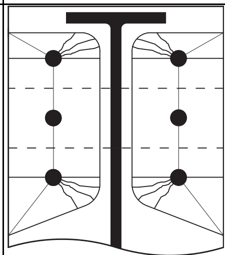
(c) Bottom row	

Circular patterns		Non-circular patterns	
	Away from stiffener/flange $\ell_{eff,cp} = \pi m + p$		Away from stiffener/flange $\ell_{eff,nc} = 2m + 0.625e + 0.5p$
	Close to stiffener/flange $\ell_{eff,cp} = \pi m + p$		Close to stiffener/flange $\ell_{eff,nc} = \alpha m - (2m + 0.625e) + 0.5p$

The value of α depends on dimensions m and m_2 and is determined from the chart in Appendix G.

Table 2.4 shows how effective lengths are built up from the contributions of individual rows.

Table 2.4 Typical examples of effective lengths for bolt rows acting in combination

Circular patterns		Non-circular patterns	
	Group of three rows in a clear length $\ell_{eff,cp} = \pi m + p$ $+ 2p$ $+ \pi m + p$ $= 2\pi m + 4p$		Group of three rows in a clear length $\ell_{eff,nc} = 2m + 0.625e + 0.5p$ $+ p$ $+ 2m + 0.625e + 0.5p$ $= 4m + 1.25e + 2p$
	Group of three rows adjacent to a stiffener/flange $\ell_{eff,cp} = \pi m + p$ $+ 2p$ $+ \pi m + p$ $= 2\pi m + 4p$		Group of three rows adjacent to a stiffener/flange $\ell_{eff,nc} = \alpha m - (2m + 0.625e) + 0.5p$ $+ p$ $+ 2m + 0.625e + 0.5p$ $= \alpha m + 2p$

STEP 1B WEB TENSION IN BEAM OR COLUMN

GENERAL

The tension resistance of the equivalent T-stub is also limited by the tension resistance of an unstiffened column web or beam web.

Column Web

The tension resistance of the effective length of column web for a row or a group of bolt rows is given by:

$$F_{t,wc,Rd} = \frac{\omega b_{eff,t,wc}\, t_{wc}\, f_{y,wc}}{\gamma_{M0}}$$

where:

ω is a reduction factor that takes account of the interaction with shear, and which depends on the transformation parameter β, see Table 2.5.

$b_{eff,t,wc}$ is the effective length of column web (= ℓ_{eff})

ℓ_{eff} is the effective length of the equivalent T-stub on the column side

t_{wc} is the thickness of the column web

$f_{y,wc}$ is the yield strength of the column web (= $f_{y,c}$ for a rolled section)

Stiffened Column Web

Web tension will not govern for any row or group of bolts where stiffeners are adjacent to or between the bolt rows being considered. A stiffener is considered adjacent if it is within 0.87w of the bolt row (where w is the bolt gauge). Stiffeners will need to be designed as described in STEP 6.

Beam Web

The tension resistance of the effective length of beam web for a row or a group of bolt rows (other than adjacent to the beam flange) is given by:

$$F_{t,wb,Rd} = \frac{b_{eff,t,wb}\, t_{wb}\, f_{y,wb}}{\gamma_{M0}}$$

where:

$b_{eff,t,wb}$ is the effective length of beam web (= ℓ_{eff})

ℓ_{eff} is the effective length of the equivalent T-stub on the beam side

t_{wb} is the thickness of the beam web

$f_{y,wb}$ is the yield strength of the beam web (= $f_{y,b}$ for a rolled section)

Table 2.5 Reduction factor for interaction with shear

Transformation parameter β	Reduction factor ω
$0 \le \beta \le 0.5$	$\omega = 1$
$0.5 < \beta < 1$	$\omega = \omega_1 + 2(1 - \beta)(1 - \omega_1)$
$\beta = 1$	$\omega = \omega_1$
$1 < \beta < 2$	$\omega = \omega_1 + (\beta - 1)(\omega_2 - \omega_1)$
$\beta = 2$	$\omega = \omega_2$

$$\omega_1 = \frac{1}{\sqrt{1 + 1.3\left(b_{eff,t,wc} t_{wc} / A_{vc}\right)^2}} \qquad \omega_2 = \frac{1}{\sqrt{1 + 5.2\left(b_{eff,t,wc} t_{wc} / A_{vc}\right)^2}}$$

A_{vc} is the shear area of the column

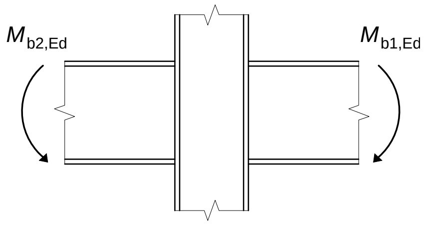
β is the transformation parameter, see below

$b_{eff,t,wc}$ is the effective width for tension in the web

Note: This Table may also be used for the web in compression (STEP 2) by using $b_{eff,c,wc}$ in the expressions for ω_1 and ω_2

STEP 1B WEB TENSION IN BEAM OR COLUMN

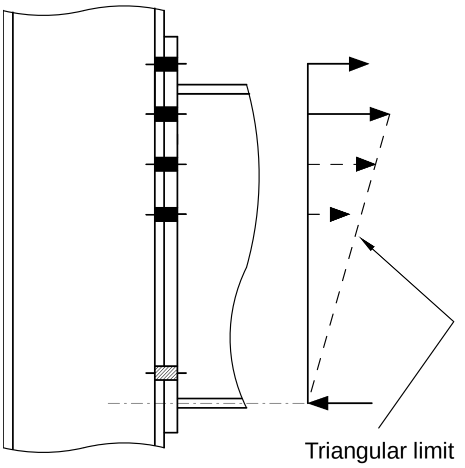
Table 2.5 (continued)

Type of joint configuration	**Action**	**Value of β**
Single-sided	$M_{b1,Ed}$ ($M_{b2,Ed} = 0$)	$\beta = 1$
Double-sided 	$M_{b1,Ed} = M_{b2,Ed}$	$\beta_1 = \beta_2 = 0$
	$M_{b1,Ed} + M_{b2,Ed} = 0$	$\beta_1 = \beta_2 = 2$
	(all other values)	$\beta_1 = \left\lvert 1 - \frac{M_{b2,Ed}}{M_{b1,Ed}} \right\rvert \leq 2$
		$\beta_2 = \left\lvert 1 - \frac{M_{b1,Ed}}{M_{b2,Ed}} \right\rvert \leq 2$

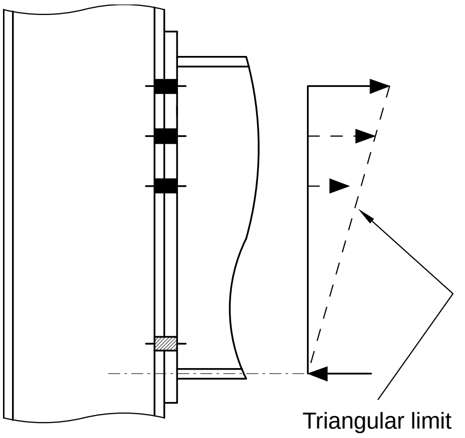
STEP 1C PLASTIC DISTRIBUTION LIMIT

PLASTIC DISTRIBUTION LIMIT

Realising the full tensile resistance of more than one bolt row requires significant ductility in the bolt rows furthest from the centre of rotation. Where the resistance depends on the deformation of the T-stubs in bending (Modes 1 or 2), sufficient ductility is generally available. If the connection is not ductile, the bolt row forces must be limited (the force in any lower row must not exceed a value pro rata to the distance from the centre of rotation, the compression flange). This is commonly referred to as a 'triangular limit' to bolt forces – see Figure 2.6.



Extended end plate



Full depth end plate

Figure 2.6 Triangular limit to bolt forces

The UK NA states that a plastic distribution can be assumed (i.e. there is sufficient ductility) when either:

$$F_{tx,Rd} \leq 1.9\, F_{t,Rd}$$

or

$$t_p \leq \frac{d}{1.9}\sqrt{\frac{f_{ub}}{f_{y,p}}}$$

or

$$t_{fc} \leq \frac{d}{1.9}\sqrt{\frac{f_{ub}}{f_{y,fc}}}$$

where:

$F_{tx,Rd}$ is the effective design tension resistance of one of the previous (higher) bolt rows x

$F_{t,Rd}$ is the design tension resistance of an individual bolt

t_p is the end plate thickness

t_{fc} is the column flange thickness

d is the diameter of the bolt

$f_{y,p}$ is the design strength of the end plate

$f_{y,fc}$ is the design strength of the column flange (= $f_{y,c}$ for a rolled section)

f_{ub} is the ultimate tensile strength of the bolt (referred to in the NA as f_u)

The first limit ensures that Mode 3 does not govern (other than for the first bolt row). The second and third limits ensure that, even if Mode 3 governs, there is significant deformation in the T-stub on at least one side of the connection.

If a plastic distribution cannot be assumed (i.e. none of the criteria are met), then the resistance of each lower bolt row r from that point on must be limited, such that:

$$F_{tr,Rd} \leq F_{tx,Rd}\frac{h_r}{h_x}$$

where:

h_x is the distance of bolt row x (the bolt row furthest from the centre of compression that has a design tension resistance greater than $1.9\, F_{t,Rd}$)

h_r is the distance of the bolt row r from the centre of compression

The centre of compression is taken as the centre line of the beam flange (see STEP 2) and the 'triangular' limit originates there, as shown in Figure 2.6.

STEP 2 COMPRESSION ZONE

GENERAL

The compression resistance is assumed to be provided at the level of the bottom flange of the beam. On the beam side, this resistance is assumed to be provided by the flange, including some contribution from the web. On the column side, the length of column web that resists the compression depends on the dispersion of the force through the end plate and column flange. If the column web is inadequate in compression, a stiffener may be provided – see STEP 6B.

Resistance of Column Web

The area of web providing resistance to compression is given by the dispersion length shown in Figure 2.7.

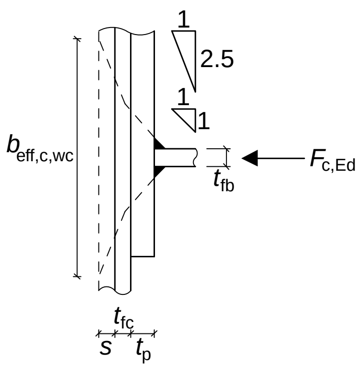


Figure 2.7 Force dispersion for column web

The design resistance of an unstiffened column web in transverse compression is determined from:

$$F_{c,wc,Rd} = \frac{\omega\ k_{wc}\ b_{eff,c,wc}\ t_{wc}\ f_{y,wc}}{\gamma_{M0}}$$

but

$$F_{c,wc,Rd} \le \frac{\omega\ k_{wc}\ \rho b_{eff,c,wc}\ t_{wc}\ f_{y,wc}}{\gamma_{M1}}$$

where:

$b_{eff,c,wc} = t_{fb} + 2s_f + 5\,(t_{fc} + s) + s_p$

ω is a reduction factor that takes account of the interaction with shear, see Table 2.5

t_{wc} is the thickness of the column web

$f_{y,wc}$ is the yield strength of the column web (= $f_{y,c}$ for a rolled section)

k_{wc} is a reduction factor, allowing for coexisting longitudinal compressive stress in the column

ρ is a reduction factor, allowing for plate buckling in the web

s = r_c for rolled I and H column sections

= $\sqrt{2}\ a_c$ for welded column sections, in which a_c is the throat thickness of the fillet weld between the column web and flange

s_f is the leg length of the fillet weld between the compression flange and the end plate (= $\sqrt{2}\ a_p$)

s_p = $2t_p$ (provided that the dispersion line remains within the end plate)

The reduction factor for maximum coexisting longitudinal compression stress in the column web $\sigma_{com,Ed}$ is given by:

When $\sigma_{com,Ed} \le 0.7\ f_{y,wc}$ $\quad k_{wc} = 1.0$

When $\sigma_{com,Ed} > 0.7\ f_{y,wc}$ $\quad k_{wc} = 1.7 - \dfrac{\sigma_{y.wc}}{f_{y.wc}}$

The stress $\sigma_{com,Ed}$ is the sum of bending and axial design stresses in the column, for the design situation at the connection.

The stress $\sigma_{com,Ed} = \dfrac{M_{Ed}}{W_{el}} + \dfrac{N_{Ed}}{A}$ but $\le f_y$. If the web is in tension throughout, k_{wc} = 1.0. In most situations this would not exceed 0.7 $f_{y,wc}$ and thus k_{wc} = 1.0. Conservatively, k_{wc} could be taken as 0.7.

The reduction factor for plate buckling is given by:

If $\bar{\lambda}_p \le 0.72$ $\quad \rho = 1.0$

If $\bar{\lambda}_p > 0.72$ $\quad \rho = \dfrac{\bar{\lambda}_p - 0.2}{\bar{\lambda}_p^{\,2}}$

In which $\bar{\lambda}_p = 0.932\sqrt{\dfrac{b_{eff,c,wc}\ d_{wc}\ f_{y.wc}}{E\ t_{wc}^{\,2}}}$

$$d_{wc} = h_c - 2(t_{fc} + s)$$

STEP 2 COMPRESSION ZONE

Resistance of the Beam Flange

The compression resistance of the combined beam flange and web in the compression zone is given by:

$$F_{c,fb,Rd} = \frac{M_{c,Rd}}{h_b - t_{fb}}$$

In a haunched section (whether from a rolled section or fabricated from plate), a convenient assumption is to calculate the resistance of the haunch flange as $1.4A_{fb}f_y$. If, for more precision, the compression zone is taken as a Tee (flange plus part of the web), the resistance of the Tee should be limited to $1.2\,A_{Tee}f_y$.

where:

$M_{c,Rd}$ is the design bending resistance of the beam cross section. For a haunched beam, $M_{c,Rd}$ may be calculated neglecting the intermediate flange

h_b is the depth of the connected beam

t_{fb} is the flange thickness of the beam (for a haunched beam, use the mean thickness of tension and compression flanges)

A_{fb} is the area of the compression flange of the beam (or the flange of the haunch, in a haunched beam)

A_{Tee} is the area of the Tee in compression

f_y is the yield strength of the beam

When the vertical shear force (V_{Ed}) is less than 50% of the vertical shear resistance of the beam cross section (V_{Rd}):

$$M_{c,Rd} = \frac{Wf_y}{\gamma_{M0}}$$

For Class 1 and 2 sections $W = W_{pl}$

For Class 3 sections $W = W_{el}$

For Class 4 sections $W = W_{eff,min}$

Where $V_{Ed} > 0.5V_{Rd}$, the bending resistance should be determined using 6.2.8 of BS EN 1993-1-1. To determine V_{Rd}, refer to 6.2.5 of BS EN 1993-1-1.

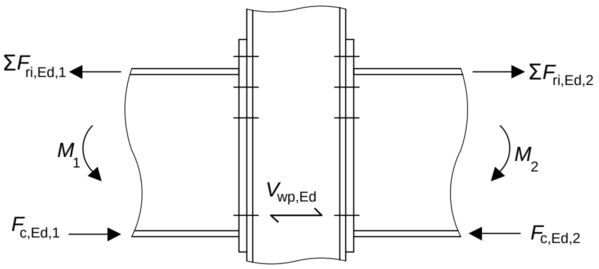
STEP 3 COLUMN WEB SHEAR

GENERAL

In single-sided beam to column connections and double-sided connections where the moments from either side are not equal and opposite, the moment resistance of the connection might be limited by the shear resistance of the column web panel.

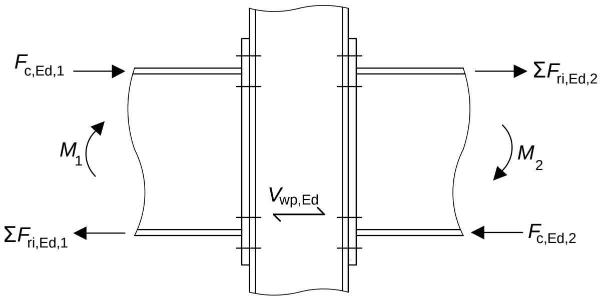
Shear Force in Column Web Panel

For a single-sided connection with no axial force in the beam, the shear force in the column web $V_{wp,Ed}$ may be taken as equal to the compression force $F_{C,Ed}$ (which equals the total tension force) – in practice the shear would be reduced by the horizontal shear force in the column below the connection but, conservatively, this reduction may be neglected.



$$V_{wp,Ed} = \left|F_{c,Ed,1} - F_{c,Ed,2}\right|$$

Moments in opposing directions



$$V_{wp,Ed} = F_{c,Ed,1} + F_{c,Ed,2}$$

Moments in the same direction

Note: The total tension forces ($\sum F_{ri,Ed}$) are shown acting at the flange centroid, for convenience. Their exact effective lines of action depend on the bolt configurations and resistances.

Figure 2.8 Shear force in web panel

For a two-sided connection with moments in opposing directions (and either no axial forces or balanced axial forces), the shear force may be taken as the difference between the two compression forces. See Figure 2.8.

For a two-sided connection with moments acting in the same direction, such as in a continuous frame, the shear force may be taken as the sum of the two compression forces. See Figure 2.8. The shear force is reduced by the horizontal shear forces that necessarily arise in the column above and below the connection.

If there is axial force in the beam of a single-sided connection or unbalanced axial forces in the beams of a double-sided connection, the consequent horizontal shear forces in the column above and below the connection should be taken into account when determining the shear force in the column web.

Shear Resistance of Web Panel

For single-sided or double-sided joints where the beams are of similar depth, the resistance of the column web panel in shear for an unstiffened web may be determined as follows:

If $\dfrac{d_c}{t_{wc}} \leq 69\varepsilon$ $\quad V_{wp,Rd} = \dfrac{0.9 f_{yc} A_{vc}}{\gamma_{M0} \times \sqrt{3}}$

BS EN 1993-1-8 gives no value for shear resistance of more slender webs. In the absence of advice, it is suggested that 90% of the shear buckling resistance may be used. Thus:

If $\dfrac{d_c}{t_{wc}} > 69\varepsilon$ $\quad V_{wp,Rd} = 0.9 V_{bw,Rd}$

where:

d_c is the clear depth of the column web $= h_c - 2(t_{fc} + s)$

h_c is the depth of the column section

s $= r_c$ for a rolled section $= \sqrt{2}\, a_c$ for a welded sections

t_{fc} is the thickness of the column flange

t_{wc} is the thickness of the column web

f_{yc} is the yield strength of the column

A_{vc} is the shear area of the column

STEP 3 COLUMN WEB SHEAR

$$\varepsilon = \sqrt{\frac{235}{f_{y,c}}}$$

$V_{bw,Rd}$ is the shear buckling resistance of the web, calculated in accordance with BS EN 1993-1-5, Clause 5.2(1).

For rolled I and H sections:

$$A_{vc} = A_c - 2 b_c t_{fc} + (t_{wc} + 2 r_c) t_{fc}$$

but

$$A_{vc} \leq \eta h_{wc} t_{wc}$$

in which $h_{wc} = h_c - 2 t_{fc}$ and η may be taken as 1.0 (according to the UK NA).

The resistance of stiffened columns (with supplementary web plates or diagonal stiffeners) is covered in STEP 6C and STEP 6D.

STEP 4 MODIFICATION OF BOLT FORCE DISTRIBUTION AND CALCULATION OF MOMENT RESISTANCE

GENERAL

The method given in STEPS 1A, 1B and 1C for assessing the resistances in the tension zone produces a set of effective resistances for each bolt row, limited if necessary to a 'triangular' distribution of forces in lower rows.

However, the total tensile resistance of the rows may exceed the compression resistance of the bottom flange, in which case not all the tension resistances can be realised simultaneously.

Similarly, in single-sided connections and double-sided connections where the moments on either side are not equal and opposite, the development of forces in the tension zone may be limited by the column web panel shear resistance.

This STEP determines the tension forces in the bolt rows that can be developed and calculates the moment resistance that can be achieved.

Reduction of Tension Row Forces

The tension forces in the bolt rows and the compression force at bottom flange level must be in equilibrium with any axial force in the beam. The forces cannot exceed the compression resistance of the joint, nor, where applicable, the shear resistance of the web panel.

Thus:

$\Sigma F_{ri} + N_{Ed} \leq F_{c,Rd}$

where:

N_{Ed} is the axial force in the beam (positive for compression)

$F_{c,Rd}$ is the lesser of the compression resistance of the joint (STEP 2) and, if applicable, the shear resistance of the web panel (STEP 3)

ΣF_{ri} is the sum of forces in all of the rows of bolts in tension

When the sum of the effective design tension resistances $\Sigma F_{ri,Rd}$ exceeds $F_{c,Rd} - N_{Ed}$, an allocation of reduced bolt forces must be determined that satisfies equilibrium.

To achieve a set of bolt row forces that is in equilibrium, the effective tension resistances should be reduced from the values calculated in STEP 1, starting with the bottom row and working up progressively, until equilibrium is achieved. This allocation achieves the maximum value of moment resistance that can be realised.

If there is surplus compression resistance (i.e. the value of $\Sigma F_{ri,Rd}$ determined by STEP 1 is less than $F_{c,Rd} - N_{Ed}$), no reduction needs to be made.

Moment Resistance

Once the bolt row forces have been determined, accounting for equilibrium, the moment resistance of the connection is given by:

$$M_{c,Rd} = \sum F_{ri,Rd}\, h_i$$

where:

$F_{ri,Rd}$ is the effective tension resistance of the *i*-th row (after any reduction to achieve equilibrium or to limit web shear)

h_i is the distance from the centre of compression to row *i*

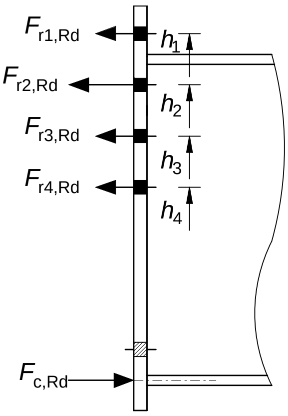


Figure 2.9 Tension and compression resistances contributing to moment resistance

Effective Applied Moment in the Presence of Axial Force

The above procedure determines a value of moment resistance about the centre of compression, through which any axial force is effectively transferred. However, where there is an axial force in the beam, the line of that force is along the centroid of the beam. The effective applied moment about the beam centroid is then determined by modifying the applied moment by adding or subtracting (as relevant) the product of the axial force and the lever arm.

This modification is illustrated in Figure 2.10 for a haunched connection.

STEP 4	**MODIFICATION OF BOLT FORCE DISTRIBUTION AND CALCULATION OF MOMENT RESISTANCE**

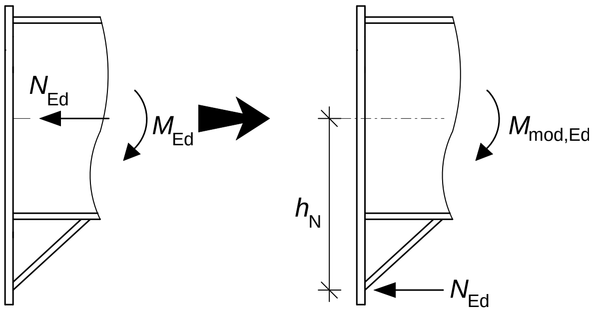


Figure 2.10 Modification of design moment for axial force

The modified design moment $M_{mod,Ed}$ is given by:

$M_{mod,Ed} = M_{Ed} - N_{Ed} \times h_N$

where:

N_{Ed} is the design force (compression positive)

h_N is the distance of the axial force from the centre of compression.

Note that, in portal frame design, the presence of the haunch is not usually assumed to affect the position of the beam centroid. The design moment and axial force derived in the global analysis will then be consistent with this assumption and the above modification to the design moment depends on the lever arm to the beam centroid, as shown in Figure 2.10.

STEP 5 SHEAR RESISTANCE OF BOLT ROWS

GENERAL

The resistance of the connection to vertical shear is provided by the bolts acting in shear. The design resistance of (non preloaded) bolts acting in shear is the lesser of the shear resistance of the bolt shank and the bearing resistance of the connected parts. The bearing resistance depends on bolt spacing and edge distance, as well as on the material strength and thickness. Where a bolt is in combined tension and shear, an interaction criterion must be observed.

BS EN 1993-1-8 allows a simplification that the vertical shear may be considered carried entirely by the bolts in the compression zone (i.e. those not required to carry tension).

If necessary, the bolts required to carry tension can also carry some shear, as discussed below.

Resistance of Fasteners in Bearing/Shear

Shear resistance

The shear resistance of an individual fastener (on a single shear plane) is given by:

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

where:

α_v = 0.5 for property class 10.9 bolts

= 0.6 for property class 8.8 bolts

A_s is the tensile stress area of the bolt

Bearing resistance

The bearing resistance of an individual fastener is given by:

$$F_{b,Rd} = \frac{k_1 \alpha_b f_u d t}{\gamma_{M2}}$$

The bearing resistances of the end plate and the column flange are calculated separately and the bearing resistance for the fastener is the lesser of the two values.

where:

$$k_1 = \min\left(2.8\frac{e_2}{d_0} - 1.7; \quad 2.5\right)$$

$$\alpha_b = \min\left(\alpha_d; \quad \frac{f_{ub}}{f_u}; \quad 1.0\right)$$

$$\alpha_b = \frac{p_1}{3d_0} - \frac{1}{4} \text{ for inner bolts}$$

$$\alpha_b = \frac{e_1}{3d_0} \text{ for end bolts}$$

d is the diameter of the bolt

d_0 is the diameter of the bolt hole

f_{ub} is the ultimate strength of the bolt

f_u is the ultimate strength of the column or end plate

t is the thickness of the column flange or end plate

γ_{M2} = 1.25, as given in the UK NA to BS EN 1993-1-8.

Dimensions e_2 and p_1 correspond to the edge distance e (for the column flange or end plate) and vertical spacing of the bolts p, as defined in Figure 2.5.

Shear Forces on Individual Fasteners

The vertical force is allocated first to the bolts in the compression zone and then, if necessary, any remaining shear force may be shared between the bolts in the tension zone.

Bolts not required to transfer tension (in the compression zone) can provide their full shear resistance (although the resistance may be limited by bearing if the column flange or end plate is thin). Bolts subject to combined tension and shear are limited by the following interaction criterion:

$$\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1.4F_{t,Rd}} \leq 1.0$$

where:

$F_{v,Ed}$ is the shear force on the bolt

$F_{t,Ed}$ is the tension force on the bolt

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

$F_{t,Rd}$ is the design tension resistance of the bolt

Calculation of the actual tension force in the bolts would require detailed evaluation of prying forces, which would be difficult to evaluate, but the above criterion allows a bolt that is subject to a tension force equal to its tension resistance to resist a coexisting shear force of 28% of its shear resistance.

STEP 5 SHEAR RESISTANCE OF BOLT ROWS

Conservatively, it may therefore be assumed that all the bolts in the tension zone can provide a resistance equal to 28% of their design shear resistance (i.e. of a bolt without tension). The distinction between bolts providing full shear resistance and reduced shear resistance is illustrated in Figure 2.11.

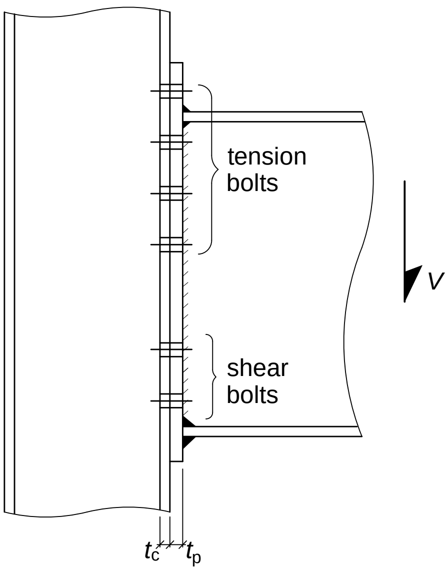


Figure 2.11 Tension and shear bolts

For ease of reference, the shear resistance of typical bolt sizes is given in Table 2.6.

Table 2.6 Shear resistances of individual bolts (property class 8.8)

Bolt size	Shear resistance $F_{v,Rd}$ (kN)	28% of $F_{v,Rd}$ (kN)
M20	94.1	26.3
M24	136	38.1
M30	215	60.2

Resistances are based on the tensile area of the bolt (i.e. assuming that the shear plane passes through the threaded portion of the bolt).

For bolts through thin elements (e.g. a thin column flange), the bearing resistance might be less than the shear resistance of the fastener.

STEP 6 COLUMN STIFFENERS

GENERAL

There are several means to increase the resistance of the column side of the connection. The following types of stiffening are covered on subsequent pages:

Tension stiffeners (STEP 6A)

Compression stiffeners (STEP 6B)

Supplementary web plates (STEP 6C)

Diagonal stiffeners (STEP 6D)

Backing plates (STEP 6E)

Tension stiffeners will increase the bending resistance of the column flange and the tension resistance of the column web.

Compression stiffeners will increase the compression resistance of the column web.

Supplementary web plates will increase the shear resistance of the web panel and, to a limited extent, the tension and compression resistances of the column web. They are likely to be used on relatively light column sections (thin flanges and thin webs) and may well need to be used in conjunction with tension and compression stiffeners.

Diagonal web stiffeners will increase the shear resistance of the web panel and will also act as tension and/or compression stiffeners. There are several forms of diagonal stiffener.

Supplementary backing plates will enhance the Mode 1 bending resistance of the flange. However, their effect is limited to making Mode 2 the dominant mode of failure (which they do not enhance). It is difficult to use them when there are tension stiffeners and they are generally only used in remedial or strengthening situations.

STEP 6A TENSION STIFFENERS

GENERAL

Tension stiffeners should be provided symmetrically on either side of the column web and may be either full depth or partial depth, as shown in Figure 2.12.

The design rules given here for partial depth stiffeners would apply equally to stiffeners on the beam side, although such stiffeners are rarely provided.

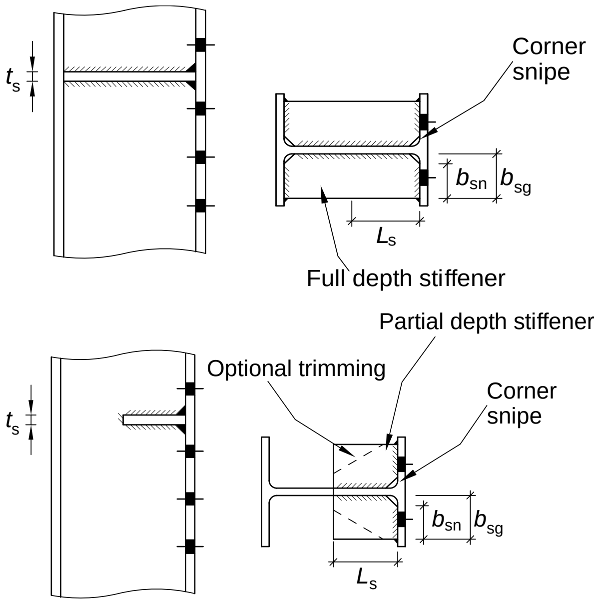


Figure 2.12 Tension stiffeners

Minimum Width

It is recommended that the overall (gross) width of each stiffener should be such that:

$$b_{sg} \geq \frac{0.75\left(b_c - t_{wc}\right)}{2}$$

In addition, a minimum width is required to ensure that the yield line patterns are constrained, as noted in STEP 1. The requirement may be expressed as:

$2b_{sg} + t_{wc} \geq 1.33\, w$

Minimum Area of Stiffener

The stiffeners act both to supplement the tension resistance of the column web and as a stiffener that restricts the bending of the flange (and thus enhances its bending resistance).

The stiffeners should be designed to carry the greater of the force needed to ensure adequate tension resistance of the web and the force due to the share of support that it provides to the flange.

The first requirement leads to a design force for each stiffener (either side of the column web) given by:

$$F_{s,Ed} = \left(F_{ri,Rd} + F_{rj,Rd} - \frac{L_{wt} t_{wc} f_{y,c}}{\gamma_{M0}}\right) \Big/ 2$$

where:

$F_{ri,Rd}$ is the effective tension resistance of the bolt row above the stiffener

$F_{rj,Rd}$ is the effective tension resistance of the bolt row below the stiffener

$f_{y,c}$ is the yield strength of the column

L_{wt} is the length of web in tension, assuming a spread of load at 60° from the bolts to the mid-thickness of the web (but not more than half way to the adjacent row or the width available at the top of a column) – see Figure 2.13.

t_{wc} is the column web thickness

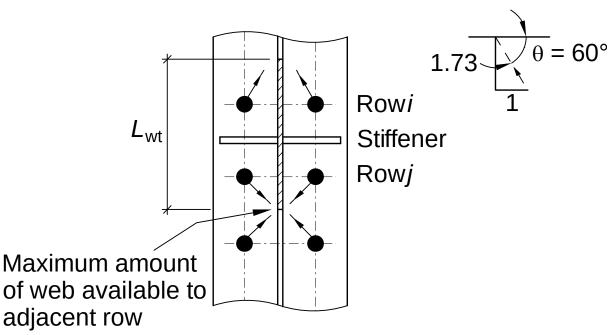


Figure 2.13 Effective length of web in tension

The second requirement is usually more onerous and leads to a design force in each stiffener given by:

$$F_{s,Ed} = \frac{m_1}{2}\left[\frac{F_{ri,Rd}}{\left(m_1 + m_{2L}\right)} + \frac{F_{rj,Rd}}{\left(m_1 + m_{2U}\right)}\right]$$

where m_1 m_{2L} and m_{2U} are as shown in Figure 2.14.

STEP 6A TENSION STIFFENERS

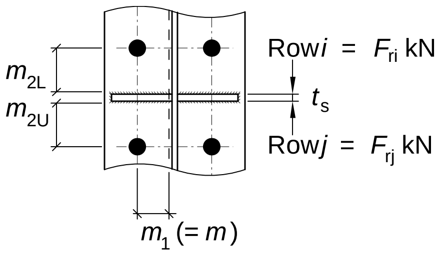


Figure 2.14 Dimensions for determining design force in web stiffeners

The net area of each stiffener should be such that:

$$A_{sn} \geq \frac{F_{s,Ed}\gamma_{M0}}{f_{y,s}}$$

where:

A_{sn} = $b_{sn}t_s$

$F_{s,Ed}$ is the greater of the above two values of design force in the stiffener.

$f_{y,s}$ is the yield strength of the stiffener

Partial Depth Stiffeners

Partial depth stiffeners must be long enough to prevent shear failure in the stiffener, web tension failure at the end of the stiffener and shear failure in the column web.

Minimum length for shear in the stiffener:

The length of a partial depth tension stiffener should be sufficient that its shear resistance (parallel to the web) is greater than the design force. Thus:

$$V_{s,Rd} = \frac{0.9L_s t_s f_{y,s}}{\sqrt{3}\gamma_{M0}} \geq F_{s,Ed} \text{ or } L_s \geq \frac{F_{s,Ed}\sqrt{3}\gamma_{M0}}{0.9t_s f_{y,s}}$$

This requirement is always satisfied (i.e. for a design force up to the full tension resistance of the stiffener) if the length of the stiffener is at least 1.9 times as long as the width b_{sn}.

Minimum length for shear in the column web:

Partial depth tension stiffeners need to be long enough to carry the applied force in shear (see above) and to transfer the applied force into the web of the column.

The local shear stress in the web must be limited to $f_{y,c}/\sqrt{3}$. There are two shear planes in the web for each pair of web stiffeners, as shown in Figure 2.15.

For a total force of 2 $F_{s,Ed}$ in a pair of web tension stiffeners, the limiting length of stiffener is given by:

$$L_s \geq \frac{F_{s,Ed}\sqrt{3}\gamma_{M0}}{t_{wc}f_{y,c}}$$

where L_s is the length of the stiffener.

For a partial depth cap plate, where there is only one shear plane, the required length is double that given by the above expression.

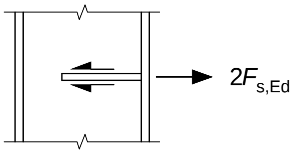


Figure 2.15 Shear force on stiffened web

The resistance of the web in tension should be verified at the end of partial depth stiffeners if the connection is double sided with opposite moments, as shown in Figure 2.16.

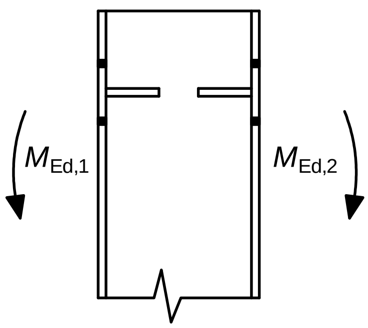
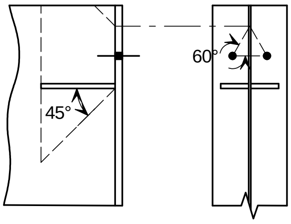


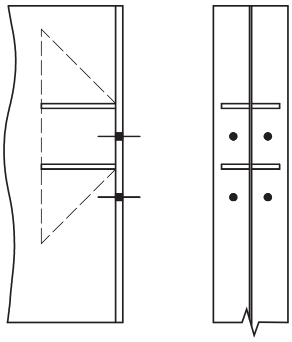
Figure 2.16 Double sided connection with opposing moments and partial depth stiffeners

The web tension resistance at the end of partial depth stiffeners should be considered row by row and by combination of rows following the principles in STEP 1. No verification is needed in single-sided connections, or if full depth tension stiffeners are provided. The design force is the *total* force in the web at that location. The design resistance should be calculated based on a length of web assuming a 45° distribution from the flange to the projection of the stiffener. In a bolted connection, a 60° distribution from the centre of the bolt to the web may be assumed. The available length may be truncated by the physical dimensions of the column. Figure 2.17, which is part of a double-sided connection, shows a range of situations with corresponding lengths of web to be considered.

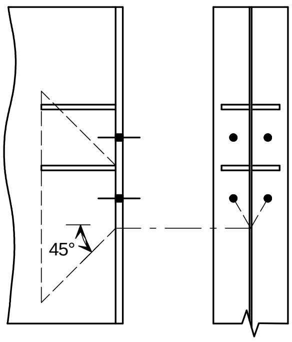
STEP 6A TENSION STIFFENERS



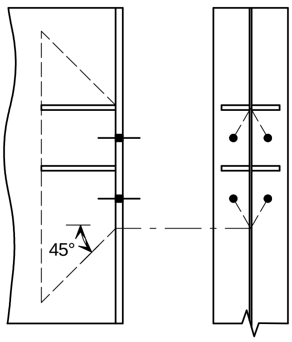
Length limited by the column top



Row 1 alone



Row 2 alone



Rows 1 and 2 together

Figure 2.17 Typical web tension checks with partial depth stiffeners (double sided connections only)

Weld Design

Welds to the flange should be capable of carrying the design force in the stiffener, taken as $F_{s,Ed}$ for each stiffener.

Welds to the web should be capable of transferring the force in the stiffener to the web.

If the length of the stiffener is less than 1.9 b_{sn} (i.e. the length has been chosen to suit a design force less than the full tension resistance of the stiffener), the fillet welds to the web and flange welds should be designed for combined transverse tension and shear (see STEP 7), assuming rotation about the root of the section, as shown in Figure 2.18.

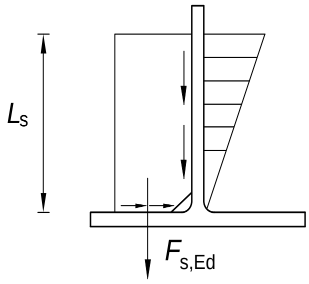


Figure 2.18 Weld design for 'short' tension stiffeners

Column Cap Plates

Where column cap plates are provided, they should be designed as a tension stiffener, as noted above.

Commonly, a full width cap plate is provided.

STEP 6B COMPRESSION STIFFENERS

GENERAL

Compression stiffeners should be provided symmetrically on either side of the column web and should be full depth, as shown in Figure 2.19. (The guidance below does not apply to partial depth stiffeners, which would require a more complex consideration of web buckling due to transverse force.)

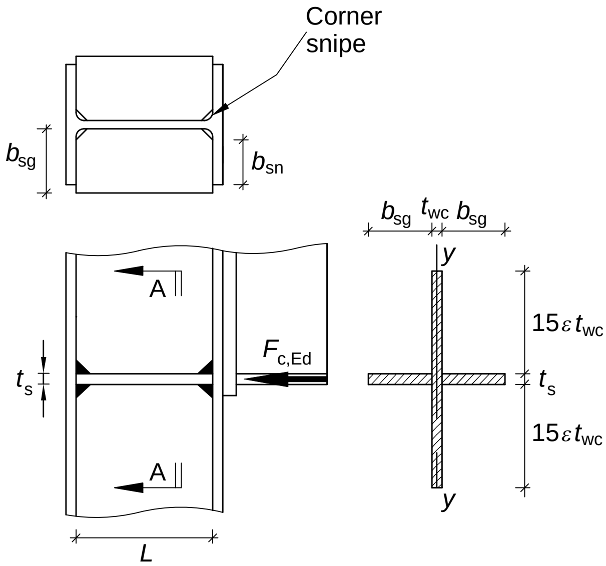


Figure 2.19 Compression stiffeners

The resistance of the effective stiffener cross section and the buckling resistance of the stiffener must be at least equal to the design force at the compression flange, which is taken as equal to the total tension resistance of the bolt rows (see STEP 4), adjusted as necessary for any axial force N_{Ed}.

The effective stiffener section for buckling resistance comprises a cruciform made up of a length of web and the stiffeners on either side. The length of web considered to act as part of the stiffener section is taken as $15\varepsilon t_{wc}$ either side of the stiffener, where $\varepsilon = \sqrt{235/f_y}$.

The width/thickness ratio of the stiffener outstand needs to be limited to prevent torsional buckling; this can be achieved by observing the Class 3 limit for compression flange outstands. Thus:

$$b_{sg}/t_s \leq 14\varepsilon$$

A greater outstand can be provided, up to $20\varepsilon t_s$, but the excess over $14\varepsilon t_s$ should be neglected in determining the effective area.

The effective area for buckling resistance is thus:

$$A_{s,eff} = (30\varepsilon t_w + t_s)t_w + 2b_{sg}t_s$$

The second moment of area of the stiffener may be taken as:

$$I_s = \frac{(2\,b_{sg} + t_{wc})^3\,t_s}{12}$$

Cross-sectional Resistance

The cross-sectional resistance of the effective compression stiffener section is:

$$N_{c,Rd} = \frac{A_{s,eff}\,f_y}{\gamma_{M0}}$$

For cross-sectional resistance, $A_{s,eff}$ is the area of stiffener in contact with the flange plus that of a length of web given by dispersal from the beam flange, see Figure 2.7.

Flexural Buckling Resistance

The flexural buckling resistance of the stiffener depends on its non-dimensional slenderness, given by:

$$\bar{\lambda} = \frac{\ell}{i_s\,\lambda_1}$$

where:

λ_1 = 93.9 ε

ℓ is the critical buckling length of the stiffener

$i_s = \sqrt{I_s/A_{s,eff}}$

For connections to columns without any restraint against twist about the column axis, assume that $\ell = h_w$. If the column is restrained against twist, a smaller length may be assumed, but not less than 0.75 h_w, where h_w is defined in STEP 3.

If the slenderness $\bar{\lambda} \leq 0.2$, which is likely for UKC sections, the flexural buckling resistance of the compressions stiffener may be ignored (only the resistance of the cross-section needs to be considered).

For $\bar{\lambda} > 0.2$, the flexural buckling resistance is given by:

$$N_{b,Rd} = \frac{\chi \times A_{s,eff} \times f_y}{\gamma_{M1}}$$

STEP 6B COMPRESSION STIFFENERS

where:

χ = $\dfrac{1}{\Phi + \sqrt{(\Phi^2 - \bar{\lambda}^2)}} \leq 1.0$

Φ = $0.5 \times \left(1 + \alpha\left(\bar{\lambda} - 0.2\right) + \bar{\lambda}^2\right)$

α = 0.49

f_y is lesser of the yield strengths of the column and the stiffener

Weld to Column Flange

The stiffener is normally fabricated with a bearing fit to the inside of the column flange. In this case the weld to the flange need only be a nominal (6 mm leg length) fillet weld. If a bearing fit is not possible, the welds should be designed to carry the force in the stiffeners.

STEP 6C SUPPLEMENTARY WEB PLATES

GENERAL

A supplementary web plate (SWP) may be provided to increase the resistance of the column web. Based on the minimum requirements below, the effect is:

- To increase web tension resistance by 50%, with a plate on one side, or by 100%, with plates on both sides
- To increase web compression resistance, by increasing the effective web thickness by 50% with a plate on one side, or by 100% with plates on both sides
- To increase the web panel shear resistance by about 75% (plates on both sides do not provide any greater increase than a plate on one side).

Dimensions and Material

The requirements for a SWP are:

- The steel grade should be the same as that of the column.
- The thickness of the SWP should be at least that of the column web.
- The width of the SWP should extend to the fillets of the column (see further detail below).
- The width should not exceed $40\varepsilon t_s$.
- The length of the SWP should extend over at least the effective lengths of the tension and compression zones of the column web.
- The welds should be designed for the forces transferred to the SWP.

The extent of the SWP is shown in Figure 2.20. The minimum lengths for dimensions L_1 and L_3 are half the values of $b_{eff,t,wc}$ and $b_{eff,c,wc}$ respectively, as determined in STEP 1B and STEP 2.

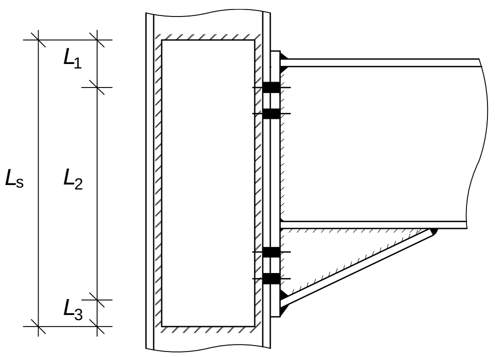


Figure 2.20 Dimensions of a SWP

Where the SWP is only required for shear, the width may be such that the toes of the perimeter fillet welds just reach the fillets of the column section. Where the SWP is required to supplement the tension or compression resistances, the longitudinal welds should be an infill weld. These two options are shown in Figure 2.21.

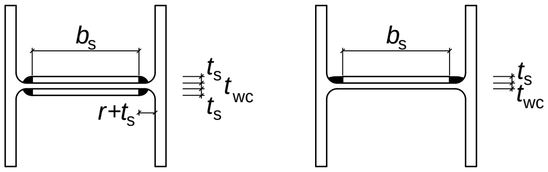


Figure 2.21 Width of supplementary web plates

To transfer the shear, the perimeter fillet welds should have a leg length equal to the thickness of the SWP.

The limiting width of $40\varepsilon t_s$ for the SWP may require a thickness greater than that of the column web. The option of using a thinner plate (than would comply with the limit) in conjunction with plug welds to the column web is outside the scope of the Eurocode rules.

Shear Resistance

In determining the shear resistance of a web panel (STEP 3), the shear area of a column web panel with SWPs is increased by an area equal to $b_s\, t_{wc}$. Thus, only one supplementary plate contributes to the shear area and the increase is independent of the thickness of the SWP (but it must be at least as thick as the web, as noted above).

Tension Resistance

The contribution to tension resistance (STEP 1B) depends on the throat thickness of the welds connecting the SWP to the web.

For infill welds (as recommended above), which are effectively butt welds, a single SWP may be assumed to increase the effective thickness of the web by 50% and two SWPs may be assumed to increase it by 100% (i.e. $t_{w,eff} = 1.5t_{wc}$ for one plate, $t_{w,eff} = 2t_{wc}$ for two plates).

Compression Resistance

For the column web in compression (STEP 2), the effective web thickness, t_{eff} should be taken as:

For a SWP on one side only, $t_{eff} = 1.5t_{wc}$

For SWPs on both sides, $t_{eff} = 2t_{wc}$

where t_{wc} is the column web thickness.

The compression resistance should be calculated using this thickness in STEP 2 and in that calculation the value of the reduction factor ω may be based on the increased shear area noted above for calculation of shear resistance.

STEP 6D DIAGONAL STIFFENERS

GENERAL

Three types of diagonal shear stiffener are shown in Figure 2.22. In all cases, the ends of the stiffeners are usually sniped to avoid the fillets of the column section, as shown for tension stiffeners in Figure 2.12 and compression stiffeners in Figure 2.19.

'K' Stiffener

This type of stiffener is used when the connection depth is large compared with the depth of the column.

Care should be taken to ensure adequate access for fitting and tightening bolts.

The bottom half of a 'K' stiffener acts in compression and should be designed as a compression stiffener, as in STEP 6B. The top half acts in tension and should be designed as a tension stiffener, as in STEP 6A.

'N' Stiffener

An 'N' stiffener (a single diagonal across the column web, forming a letter N with the two flanges) is usually placed so that it acts in compression due to problems of bolt access if placed so as to act in tension. It should then be designed to act as a compression stiffener, STEP 6B, unless a horizontal compression stiffener is also present.

Morris Stiffener

The Morris stiffener is structurally efficient and overcomes the difficulties of bolt access associated with the other forms of diagonal stiffener.

It is particularly effective for use with UKBs as columns, but is difficult to accommodate in the smaller UKC sizes.

The horizontal portion of the stiffener acts as a tension stiffener and should be designed as in STEP 6A. The length should be sufficient to provide for bolt access (say 100 mm).

Compression stiffeners are often provided at the bottom of a Morris stiffener, to enhance the compression resistance of the thin web.

Area of Stiffeners

The gross area of the stiffeners, A_{sg} should be such that:

$$A_{sg} \geq \left(V_{wp,Ed} - V_{wp,Rd}\right) / f_y \cos\theta$$

where:

A_{sg} = 2 x b_{sg} x t_s

b_{sg} is the width of stiffener on each side

t_s is the thickness of stiffener

$V_{wp,Ed}$ is the design shear force (see STEP 3)

$V_{wp,Rd}$ is the resistance of the unstiffened column web panel (see STEP 3)

f_y is the lesser of the design strengths of the stiffener and the column

θ is the angle of the stiffener (see Figure 2.22).

The net area of the stiffeners should also be sufficient to transfer the tension or compression forces (STEP 6A and STEP 6B).

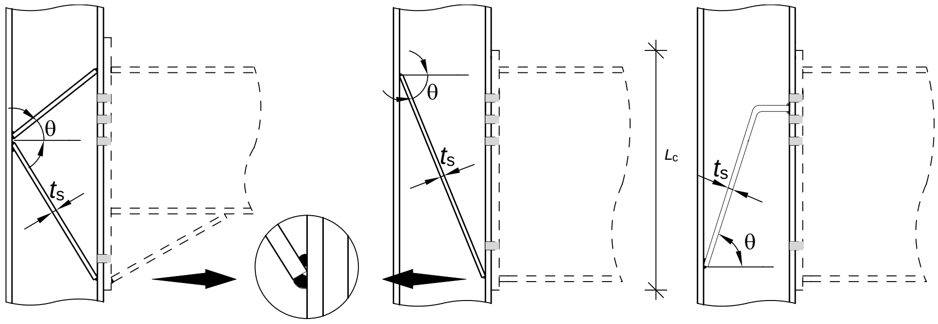
Welds

Welds connecting diagonal stiffeners to the column flange should be 'fill-in' welds, with a sealing run providing a combined throat thickness equal to the thickness of the stiffener, as shown in Figure 2.22.

Welds connecting the horizontal portion of Morris stiffeners to the column flange should be designed for the force in the stiffener, $F_{s,Ed}$ (see STEP 6A).

The welds to the column web are usually nominal 6 mm or 8 mm leg length fillet welds.

STEP 6D DIAGONAL STIFFENERS



K stiffener N stiffener Morris stiffener

Figure 2.22 Diagonal stiffeners

STEP 6E FLANGE BACKING PLATES

GENERAL

The bending resistance of a column flange can be increased by providing backing plates, as shown in Figure 2.23.

This type of strengthening increases the resistance to a Mode 1 failure. Mode 2 and Mode 3 resistances are not affected.

The width of the backing plate, b_{bp} should not be less than the distance from the edge of the flange to the toe of the root radius, and it should fit snugly against the root radius.

The length of the backing plate should be such that it extends not less than 2*d* beyond the bolts at each end (where *d* is the bolt diameter).

Enhanced Resistance in Mode 1

The tension resistance of the equivalent T-stub when there are column flange backing plates is given by:

$$F_{T,Rd} = \frac{(8n - 2e_w)M_{pl,Rd} + 4nM_{bp,Rd}}{2mn - e_w(m + n)}$$

Where all the parameters are as defined in STEP 1A except that:

$$M_{bp,Rd} = \frac{0.25\Sigma\ell_{eff,1}t_{bp}^2 f_{y,bp}}{\gamma_{M0}}$$

t_{bp} is the thickness of the backing plates

$f_{y,bp}$ is the yield strength of the backing plates

The effective length that is used for $M_{pl,1,Rd}$ and $M_{bp,Rd}$ will normally be that determined for the column flange alone but, for the end row, the length to the free end of the backing plate might be less than the corresponding part of the column flange forming the T-stub. In that case, the effective length should be calculated for the backing plate and should be used conservatively for both the backing plate and the flange.

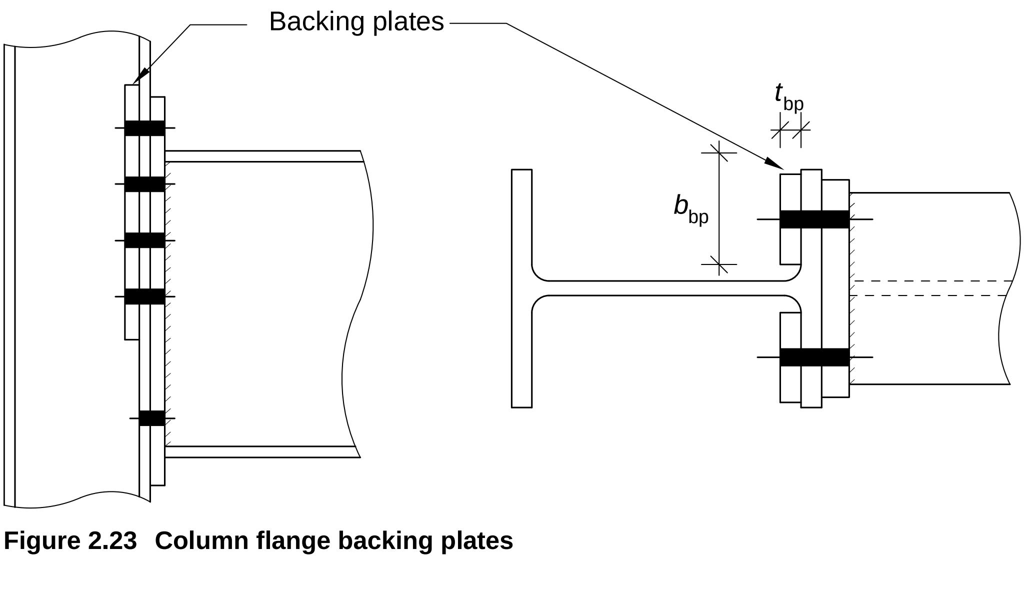


Figure 2.23 Column flange backing plates

STEP 7 DESIGN OF WELDS

GENERAL

Welds are used to transfer shear forces (along their length), tension forces (transverse to their length) and a combination of both shear and tension.

Resistance of Fillet Welds

Resistance in shear

The design shear resistance of a fillet weld (shear per unit length) is given by:

$$F_{\mathrm{vw,Rd}} = af_{\mathrm{vw,d}} = \frac{af_{\mathrm{u}}/\sqrt{3}}{\beta_{\mathrm{w}}\gamma_{\mathrm{M2}}}$$

where:

f_{u} is the ultimate tensile strength of the weaker part joined

β_{w} is the correlation factor according to the strength of the weaker part taken as 0.85 for S275 and 0.90 for S355

a is the throat thickness of the weld

γ_{M2} = 1.25, as given in the UK NA to BS EN 1993-1-8.

Resistance to transverse forces

The design resistance of a fillet weld subject to transverse force is given by:

$$F_{\mathrm{nw,Rd}} = K\frac{af_{\mathrm{u}}/\sqrt{3}}{\beta_{\mathrm{w}}\gamma_{\mathrm{M2}}}$$

where:

$$K = \sqrt{\frac{3}{1+2\cos^{2}\theta}}$$

θ is the angle between the direction of the force and the throat of the weld (see Figure 2.24).

For $\theta = 45°$ $K = 1.225$

For a pair of symmetrically disposed welds subject to a transverse force, as in Figure 2.24, a 'full strength' connection (i.e. one that has a resistance equal to or greater than that of the tension element) can be made with fillet welds. For joints between elements of the same steel grade, a full strength weld can be provided by fillet welds with a total throat thickness equal to that of the element, for a joint made in S275 material, or 1.2 times the element thickness for a joint made in S355 material.

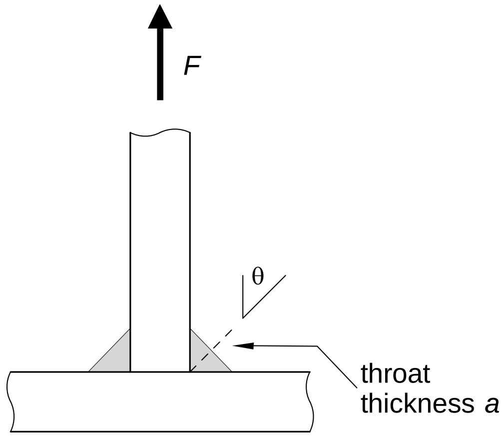


Figure 2.24 Weld subject to transverse force

Resistance of Butt Welds

The design resistance of a full penetration butt weld may be taken as the strength of the weaker part that is joined.

A partial penetration butt weld reinforced by fillet welds may be designed as a deep penetration fillet weld, taking account of the minimum throat thickness and its angle relative to the direction of the transverse force.

Weld Zones

For convenience, the beam to end plate welds may be considered in zones, as shown in Figure 2.25.

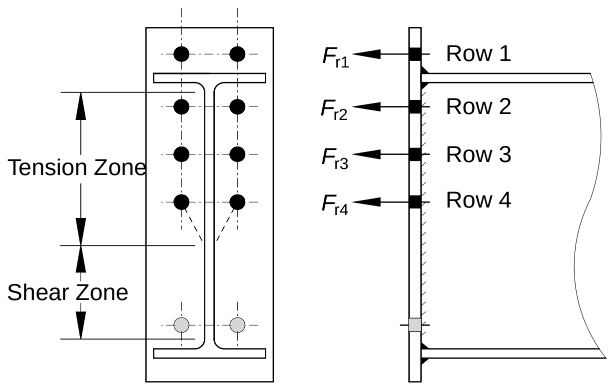


Figure 2.25 Beam web to end plate weld zones

STEP 7 DESIGN OF WELDS

Welds to Tension Flange and Beam Web

a) Beam flange to end plate

The welds between the tension flange and the end plate may be full strength or designed to provide a resistance that is equal to the total resistance of the bolt rows above and below it (if that total is less).

For a full strength weld, either provide a full penetration butt weld or fillet welds with sufficient throat thickness to resist a force equal to the resistance of the tension flange.

For a partial strength connection, provide fillet welds with a resistance at least equal to a design force given by:

(a) For an extended end plate, the total tension force in the top three bolt rows:

$$F_{w,Ed} = (F_{r1} + F_{r2} + F_{r3})$$

(b) For a full depth end plate. the total tension force in the top two bolt rows:

$$F_{w,Ed} = (F_{r1} + F_{r2})$$

For most small and medium sized beams, the tension flange welds will be symmetrical, full strength fillet welds. Once the leg length of the required fillet weld exceeds 12 mm, a detail with partial penetration butt welds and superimposed fillets may be a more economical solution.

Care should be taken not to undersize the weld to the tension flange. A simple and safe solution is to provide full strength welds.

b) Beam web to end plate

For many beams, a simple and conservative solution is to provide full strength welds to the entire web. Two 8 mm leg fillet welds provide a full strength weld for S275 webs up to 11.3 mm and for S355 webs up to 9.4 mm thick.

If the web is thick, and a full strength weld is uneconomic, the web may be spilt into two zones – the tension zone and the shear zone. In each zone, the weld may be sized to carry the design forces.

Web welds in the tension zone

It is recommended that the welds to the web in the tension zone are full strength, unless the web is thick, and has a much higher resistance than the design resistances of the T-stubs in the tension zone. In these circumstances, instead of providing a large full strength weld, the weld may be designed for the effective resistances of the T-stubs (see STEP 4).

Where the size of the web fillet welds is smaller than that for the flange welds, the transition between the flange weld and the web weld should be detailed where the fillet meets the web.

Welds in the shear zone

For simplicity, web welds in the shear zone may be full strength. Alternatively, the welds may be sized to carry the vertical shear force, assuming that all the shear is resisted by this zone. A minimum size of 6 mm leg length is recommended.

Welds to Compression Flange

In cases where the compression flange has a properly sawn end, a bearing fit can be assumed between the flange and end plate and a 6 mm or 8 mm leg length fillet weld on both faces will suffice.

Adequate bearing may be assumed for sawn plain beams and for haunches which have been sawn from UKBs or UKCs. Guidance on the necessary tolerances for bearing fit can be found in the NSSS [4].

If a bearing fit cannot be assumed, then the weld should be designed to carry a force equal to the force in the compression flange for the design moment resistance and, if present, any axial force in the beam (as described for compression stiffeners in STEP 6B).

Welds to Stiffeners

Design requirements for welds to stiffeners are given in STEP 6A and 6B.

STEP 8 HAUNCHED CONNECTIONS

GENERAL

Haunches may be used to:

- Provide a longer lever arm for the bolts in tension;
- Increase the member size over part of its length.

The principal dimensions of haunch depth and haunch length are indicated in Figure 2.26.

The haunch depth is chosen to achieve the required moment resistance for the member.

The haunch length is chosen to ensure that the resistance of the beam at the end of the haunch is adequate for the moment at that location (which is usually significantly less than at the column).

The haunch should be arranged with:

- Steel grade to match that of the member
- Flange size not less than that of the member
- Web thickness not less than that of the member
- The angle of the haunch flange to the end plate not less than 45°. See Figure 2.26.
- The fit of the haunch to the end plate should be to the same tolerance as for the bearing fit of a beam to an end plate (see the NSSS).

The haunch is usually cut from a rolled section (in most cases, the same section as the beam).

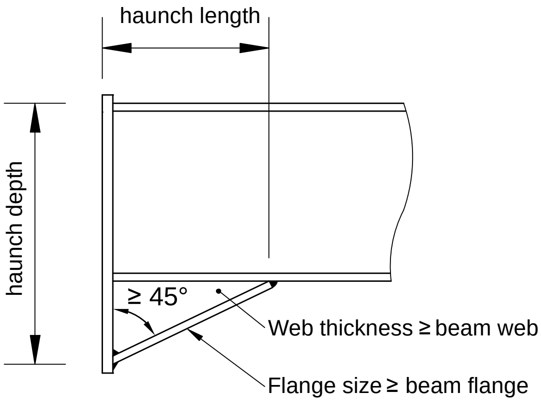


Figure 2.26 Haunch dimensions and fit-up

Haunch Design

The haunch flange and its web provide the compression resistance, as for an unhaunched beam, and the compression zone is designed as in STEP 2. At the end plate, the lower flange of the beam is not assumed to provide any compression resistance.

To determine the force in the haunch flange at the sharp end, it may be assumed that, immediately adjacent to the end of the haunch, the force in the plain beam flange is equal to the design moment divided by the depth of the section, but not more than the resistance of the flange itself (flange area × f_y). This force can then be distributed to the haunch flange and beam flange in proportion to their areas, but no more than 50% of the total force should be allocated to the haunch flange. The component of force perpendicular to the beam can then be determined.

The beam should be checked for a point load equal to the transverse component of force from the haunch flange, in a similar manner to the column web in compression, although the force is only likely to be significant when there is a plastic hinge at this location. It may be assumed that the stiff bearing length is based on a weld leg equal to the thickness of the haunch flange thickness. If the transverse force exceeds 10% of the shear resistance of the cross- section, web stiffeners must be provided within $h/2$ of the plastic hinge location, where h is the beam depth.

Haunch Welds

Haunch flange to end plate

As described in STEP 7, if the haunch is sawn from a rolled section, only a 6 mm or 8 mm leg length fillet weld is required. If the connection experiences moment reversal, the weld should be designed for the appropriate tension force (see STEP 7)

Lower beam flange to end plate

In a haunched connection, it is assumed that at the end plate, all the compression is in the haunch flange and adjacent web; it is assumed that the lower beam flange does not carry significant force. A 6 mm or 8 mm fillet weld will suffice around the lower beam flange, unless the beam flange acts as a tension stiffener in a reversal case.

Haunch flange to beam flange (sharp end of the haunch).

The transverse weld across the end of the haunch flange should be designed for the force transferred into the haunch flange, as described above.

Generally, a fillet weld with a leg length equal to the thickness of the haunch flange will be satisfactory. When cut from a rolled section, the usual geometry of the haunch cutting suits a fillet weld at this location, as shown in Figure 2.27.

STEP 8 HAUNCHED CONNECTIONS

More complex calculations to determine the proportion of force in the weld can be carried out, if necessary. The weld capacity is limited by the physical geometry; if the calculations indicate a larger force than the maximum fillet weld can carry, the excess force should be included in the design of the haunch web to beam flange. This approach should not be used to reduce the fillet weld across the end of the haunch flange – a fillet weld with a leg length equal to the depth of the haunch flange should generally be provided.

Haunch web to beam flange

The weld between the underside of the beam and the haunch must carry the difference between the force applied at the column, and that allocated to the haunch flange at the sharp end of the haunch. For most orthodox haunches, the length of weld available means that the resistance of a 6 mm or 8 mm leg length fillet weld exceeds the design forced by a considerable margin. Suitably designed intermittent welds may be used where aesthetics and corrosion conditions permit.

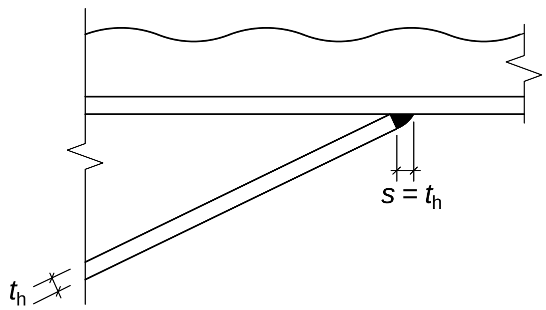


Figure 2.27 Weld at sharp end of the haunch

3 WELDED BEAM TO COLUMN CONNECTIONS

3.1 SCOPE

This Section deals with the design and detailing of shop welded beam to column connections.

The intention with shop welded construction is to ensure that the main beam to column connections are made in a factory environment. To achieve this, while still keeping the piece sizes small enough for transportation, short stubs of the beam section are welded to the columns. The connection of the stub to the rest of the beam is normally made with a bolted cover plate splice.

A typical arrangement for a multi-storey building is shown in Figure 3.1.

3.2 SHOP WELDED CONNECTIONS

A typical shop welded connection, as shown in Figure 3.2, consists of short beam section stubs, shop welded on to the column flanges, and tapered stubs welded into the column inner profile on the other axis. The stub sections are prepared for bolting or welding with cover plates to the central portion of beam. The benefits of this approach are:

- Efficient, full strength moment-resisting connections.
- All the welding to the column is carried out under factory conditions.
- The workpiece can be turned to avoid or minimise positional welding.

The disadvantages are:

- More connections and therefore higher fabrication costs.
- The 'column tree' stubs make the component difficult to handle and transport.
- The beam splices have to be bolted or welded in the air some distance from the column.
- The flange splice plates and bolts may interfere with some types of flooring such as pre-cast units or metal decking.

Practical considerations

Continuous fillet welds are the usual choice for most small and medium sized beams with flanges up to 17 mm thick. However, many fabricators prefer to switch to partial penetration butt welds with superimposed fillets, or full penetration butt welds, rather than use fillet welds larger than 12 mm.

To help provide good access for welding during fabrication, the column shafts can be mounted in special manipulators and rotated to facilitate welding in a 'downhand' position to each stub (Figure 3.3).

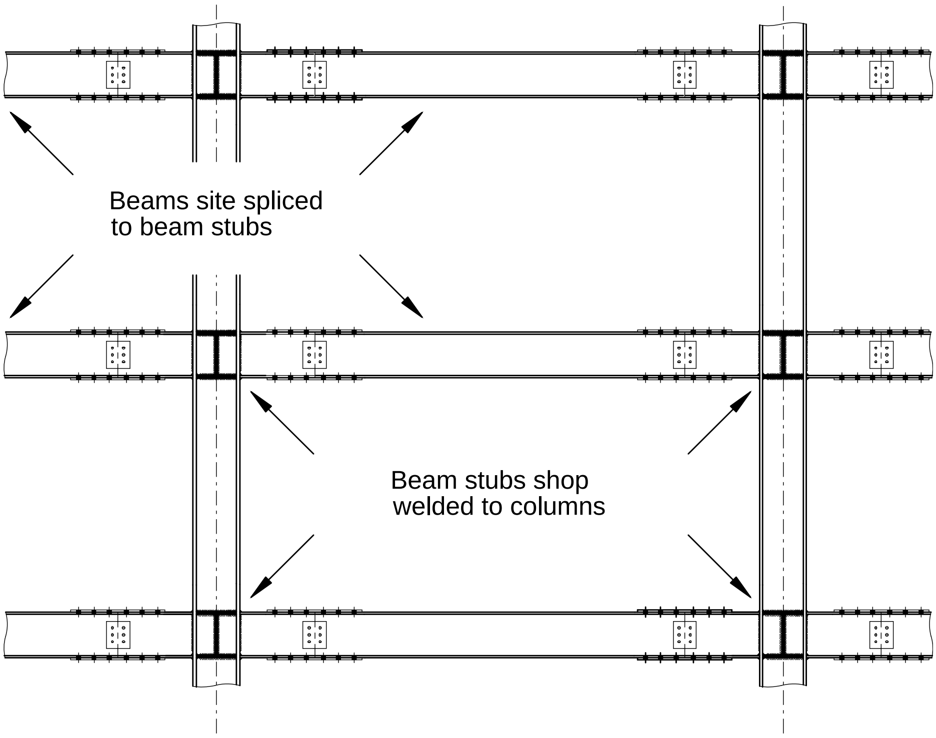


Figure 3.1 Shop welded beam to column connections

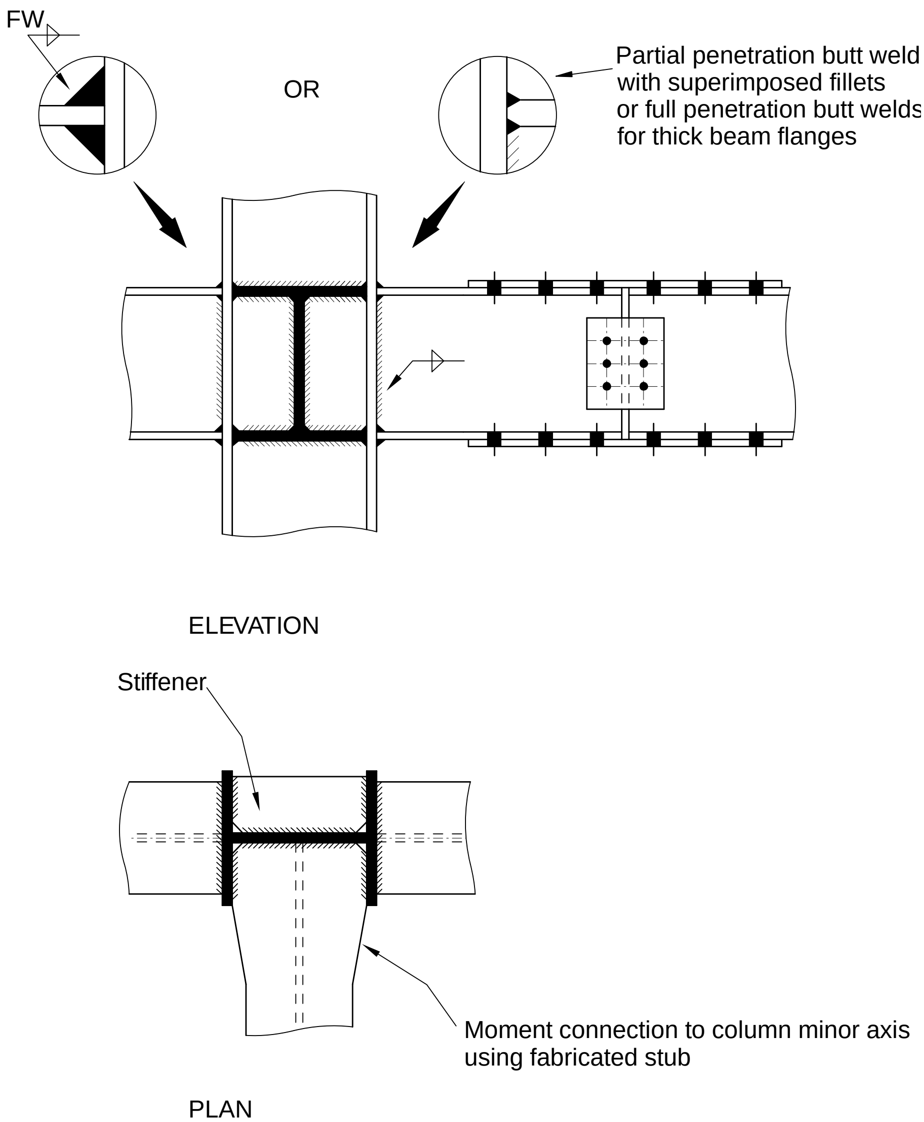


Figure 3.2 Shop welded beam stub connection

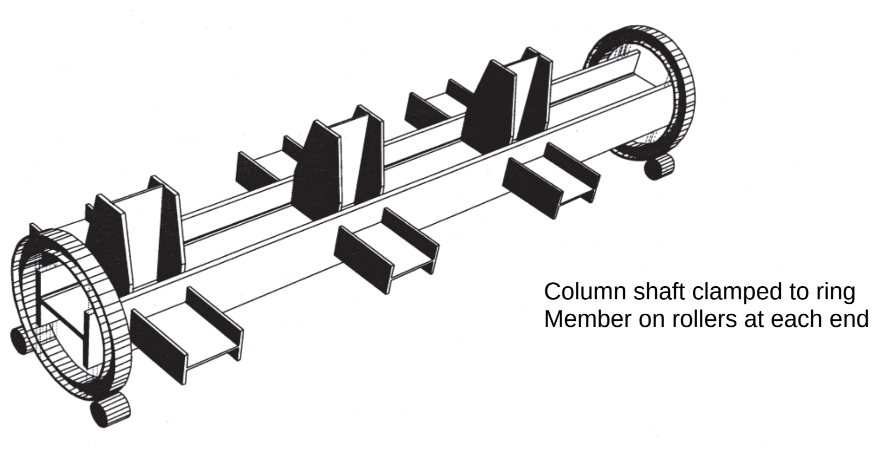


Figure 3.3 Column manipulator for welding beam stubs to columns

3.3 DESIGN METHOD

In statically determinate frames, a partial strength connection, adequate to resist the design moment is satisfactory.

If the frame is statically indeterminate, the connections must have sufficient ductility to accommodate any inaccuracy in the design moment arising, for example, from frame imperfections or settlement of supports. In a semi-continuous frame, ductility is necessary to permit the assumed moment redistributions. To achieve this, in statically indeterminate frames and in semi-continuous frames, the welds in the connection must be made full strength.

The verification of the resistance of a welded beam to column connection is summarised in the five STEPS outlined below. The components that need to be considered are illustrated in Figure 3.4.

STEP 1

Calculate the design forces in the tension and compression flanges of the beam. The presence of the web may be neglected when determining these forces.

STEP 2

Calculate the resistances in the tension zone and verify their adequacy. If, for an unstiffened column, the resistances are inadequate, determine the resistance for a stiffened column and verify its adequacy.

STEP 3

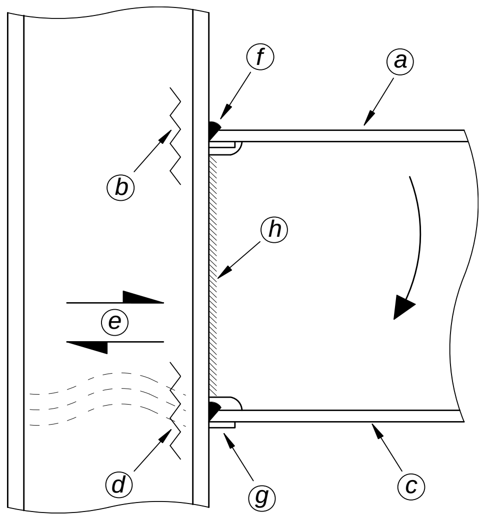
Calculate the resistances in the compression zone and verify their adequacy. If, for an unstiffened column, the resistances are inadequate, determine the resistance for a stiffened column and verify its adequacy.

STEP 4

Verify the adequacy of the column web panel in shear. If the unstiffened panel is inadequate, it may be stiffened, as for an end plate connection – see Section 2.

STEP 5

Verify the adequacy of the welds to the flanges and web.



ZONE	REF	COMPONENT	Procedure
TENSION	a	Beam flange	STEP 2
	b	Column web	STEP 2
COMPRESSION	c	Beam flange	STEP 3
	d	Column web	STEP 3
HORIZONTAL SHEAR	e	Column web panel shear	STEP 4
WELDS	f,g	Flange welds	STEP 5

Figure 3.4 Components to be evaluated in design procedure

3.4 DESIGN STEPS

The following STEPS set out the details of the five STEPS described above. A worked example illustrating the procedure is given in Appendix F.

STEP 1 WELDED CONNECTIONS – DISTRIBUTION OF FORCES IN THE BEAM

GENERAL

The design forces in the beam tension flange $F_{T,Ed}$ and in the beam compression flange $F_{C,Ed}$, shown in Figure 3.5, are given by:

$$F_{t,Ed} = \frac{M_{Ed}}{(h_b - t_{fb})} - \frac{N_{Ed}}{2}$$

$$F_{c,Ed} = \frac{M_{Ed}}{(h_b - t_{fb})} + \frac{N_{Ed}}{2}$$

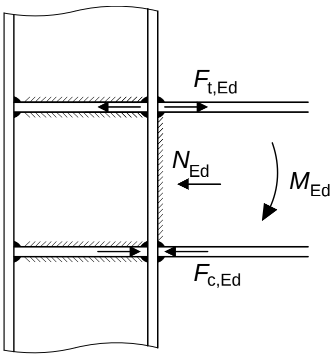


Figure 3.5 Distribution of forces in welded beam to column connection

where:

h_b is the overall depth of the beam section

t_{fb} is the thickness of the beam flange

M_{Ed} is the design bending moment in the beam (positive for top flange in tension)

N_{Ed} is the design axial force in the beam (positive for compression)

STEP 2 WELDED CONNECTIONS – RESISTANCE IN TENSION ZONE

GENERAL

Column tension stiffeners are not required if the resistances of the beam flange and column web are adequate, that is

If $F_{t,Ed} \leq F_{t,fb,Rd}$ and $F_{t,Ed} \leq F_{t,wc,Rd}$

Unstiffened Column

Resistance of beam flange

The resistance of the beam flange depends on its effective width, as shown in Figure 3.6.

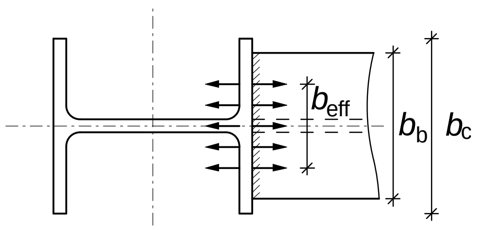


Figure 3.6 Effective width of beam flange

The effective width of a beam flange connected to an unstiffened column depends on the dispersion of force from the column web to the beam.

For an unstiffened I or H section

$$b_{eff} = t_{wc} + 2s + 7kt_{fc}$$

but $\leq b_b$ and b_c

For I and H sections:

If $b_{eff} < \left(\frac{f_{y,b}}{f_{u,b}}\right) b_b$, stiffeners are required.

For other sections see 4.10 of BS EN 1993-1-8.

The design resistance of the effective breadth of beam flange shown in Figure 3.6 is given by:

$$F_{t,fb,Rd} = \frac{b_{eff} t_{fb} f_{y,fb}}{\gamma_{M0}}$$

where:

$$k = \left(\frac{t_{fc}}{t_{fb}}\right)\left(\frac{f_{y,c}}{f_{y,b}}\right) \text{ but } k \leq 1$$

b_c is the width of the column

t_{wc} is the thickness of the column web

t_{fc} is the thickness of the column flange

b_b is the width of the beam

t_{fb} is the thickness of the beam flange

r_c is the root radius of the column

s $= r_c$ (for a rolled I or H section)

s $= \sqrt{2}a$ (for a welded I or H section)

where a is the throat thickness of the weld between the web and flange.

$f_{y,c}$ is the design yield strength of the column

$f_{y,b}$ is the design yield strength of the beam

$f_{u,b}$ is the ultimate tensile strength of the beam

A_{sn} is the net stiffener area

$f_{y,s}$ is the design yield strength of the stiffener

$F_{t,Ed}$ is the design tension force (see STEP 1)

STEP 2 WELDED CONNECTIONS – RESISTANCE IN TENSION ZONE

Resistance of column web

The spread of tension force $F_{t,Ed}$ into the column web is taken as 1:2.5, as shown in Figure 3.7. When the beam is near an end of the column the effective length of web must be reduced to that available.

Thus:

$$b_{eff,t,wc} = t_{fb} + 2s_f + 5(s + t_{fc})$$

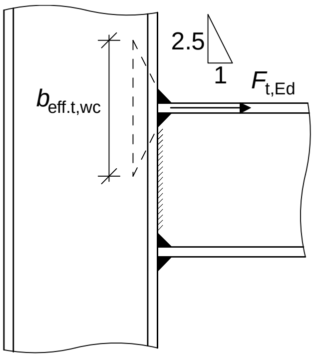


Figure 3.7 Length of column web resisting tension

The resistance of the column web is given by:

$$F_{t,wc,Rd} = \frac{\omega b_{eff,t,wc} t_{wc} f_{y,wc}}{\gamma_{M0}}$$

Stiffened Column

If column web stiffeners are required, a pair of stiffeners should be provided, either partial depth or full depth, as shown in Figure 2.12 for bolted end plate connections.

The strength of the stiffeners and the welds attaching them to the column web and flange should be verified in the same way as for stiffeners for a bolted end plate connection (see STEP 6A in Section 2.5). The design force may be taken conservatively as:

$$F_{s,Ed} = F_{t,fb,Rd} - \omega b_{eff,t,wc} t_{wc} f_{y,c} / \gamma_{M0}$$

When column stiffeners are provided, the entire beam flange is effective.

where:

ω is a reduction factor for the interaction with shear that is determined using the method given in STEP 1B for the bolted end plate (Section 2), using $b_{eff,t,wc}$

$b_{eff,t,wc}$ is as given above

s for a rolled section is the root radius r or, for a welded section, the leg length of the column web to flange fillet welds

t_{fb} is the beam flange thickness

t_{fc} is the column flange thickness

t_{wc} is the column web thickness

$f_{y,wc}$ is the yield strength of the column web (= $f_{y,c}$ for a rolled section)

The resistance of the web in tension should be verified at the end of partial depth stiffeners in double-sided connections, following the guidance in Section 2.5, STEP 6A.

STEP 3 WELDED CONNECTIONS – RESISTANCE IN COMPRESSION ZONE

GENERAL

Column compression stiffeners are not required if the resistances of the beam flange and column web are adequate, that is

If $F_{c,Ed} \leq F_{c,fb,Rd}$ and $F_{c,Ed} \leq F_{b,wc,Rd}$

Unstiffened Column

Resistance of beam flange

The effective width of the beam flange is as given for the tension flange in STEP 2.

For I and H sections:

If $b_{eff} < \left(\frac{f_{y,b}}{f_{u,b}}\right) b_b$, stiffeners are required.

For other sections, see 4.10 of BS EN 1993-1-8.

The design compression resistance of the effective breadth of beam flange connection shown in Figure 3.6 is that given in STEP 2 as:

$$F_{c,fb,Rd} = \frac{b_{eff} t_{fb} f_{y,fb}}{\gamma_{M0}}$$

Column Web – Unstiffened Column

The compression resistance of the web is given by:

$$F_{c,wc,Rd} = \frac{\omega k_{wc} b_{eff,c,wc} t_{wc} f_{y,wc}}{\gamma_{M0}}$$

but

$$F_{c,wc,Rd} \leq \frac{\omega k_{wc} \rho b_{eff,c,wc} t_{wc} f_{y,wc}}{\gamma_{M1}}$$

For the determination of the variables in the above equations, see STEP 2 in Section 2, using $b_{eff,c,wc}$ in Table 2.5.

Stiffened Column

The effective width and resistance of the beam flange are determined as for tension stiffeners in STEP 2.

For the resistance of the compression zone of a stiffened column, refer to Section 2.5, STEP 6B.

STEP 4 WELDED CONNECTIONS – COLUMN WEB PANEL SHEAR

GENERAL

In single-sided beam to column connections and double-sided connections where the moments from either side are not equal and opposite, the moment resistance of the connection might be limited by the shear resistance of the column web panel.

The forces in the column web and the resistance of the column web may be determined as in STEP 3 for a bolted beam to column connection.

STEP 5 WELDED CONNECTIONS – WELDS

GENERAL

The flange to column welds for the tension flange and compression flange should normally be full strength if the frame is statically indeterminate. Full strength welds are the default requirement in 4.10(5) of BS EN 1993-1-8.

For determinate frames, or connections with thick beam flanges but low design moments, the weld may be designed for the force in the flange. For this purpose, it is generally satisfactory to assume that the flanges of the beam carry the design moment.

If the weld is less than full strength, the weld should be sufficient to resist the design force, distributed over the effective width b_{eff} calculated in STEP 2. The same size weld should be specified around the entire flange. If the column is stiffened, the design force should be distributed over the lesser of b_c and b_b when designing the flange to column weld.

In continuous frames, moment reversal is expected, meaning the compression flange weld needs to be designed as a tension flange weld to cover this reversal. If the force in the flange can only ever be compression, and the beam has a sawn end in direct bearing, a 6 mm or 8 mm leg length fillet weld will suffice.

The beam web to column welds should be full strength.

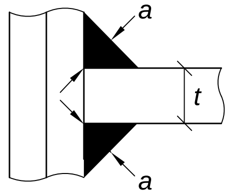
Full Strength Welds

For elements such as flanges, which are principally subject to direct tension or compression, a full strength weld may be provided by a pair of symmetrically disposed fillet welds. For such a detail to be full strength, the following weld sizes are required, based on the rules for weld strength given in STEP 7 of Section 2.5.

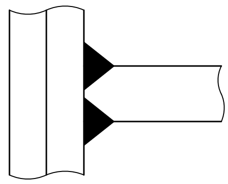
Table 3.1 Dimensions of fillet welds for full strength connection for transverse force

Steel grade	Weld size, as a proportion of the thickness of the connected part	
	Throat	Leg
S275	0.5	0.71
S355	0.6	0.85

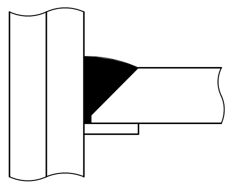
Full strength welds may also be achieved by partial penetration welds with superimposed fillet welds (used when the fillet weld would otherwise be very large) or butt welds, as shown in Figure 3.8.



Fillet weld



Partial penetration with superimposed fillet



Full penetration butt weld

Figure 3.8 Weld types

4 SPLICES

4.1 SCOPE

This Section deals with the design of beam and column splices between H or I sections that are subjected to bending moment, axial force and transverse shear force. The following types of joint are covered:

- Bolted cover plate splices.
- Bolted end plate splices.
- Welded splices.

The design of bolted column splices that are subject to predominant compressive forces is covered in *Joints in Steel Construction – Simple Joints to Eurocode 3* (P358)[5].

Column splices can be either of bearing or non-bearing type. Design guidance for bearing type column splices is given in P358[5]. Non-bearing column splices may be arranged and designed as for beam splices.

4.2 BOLTED COVER PLATE SPLICES

Connection details

Typical bolted cover plate splice arrangements are shown in Figure 4.1.

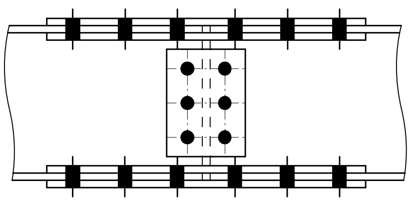
In a beam splice there is a small gap between the two beam ends. For small beam sections, single cover plates may be adequate for the flanges and web. For symmetric cross sections, a symmetric arrangement of cover plates is normally used, irrespective of the relative magnitudes of the design forces in the flanges.

Design basis

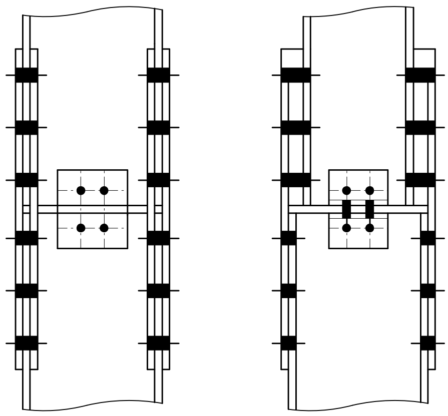
A beam splice (or a non-bearing column splice) resists the coexisting design moment, axial force and shear in the beam by a combination of tension and compression forces in the flange cover plates and shear, bending and axial force in the web cover plates.

To achieve a rigid joint classification, the connections must be designed as slip resistant connections. It is usually only necessary to provide slip resistance at SLS (Category B according to BS EN 1993-1-8, 3.4.1) although if a rigid connection is required at ULS, slip resistance at ULS must be provided (Category C connection).

In elastically analysed structures, bolted cover plate splices are not required to provide the full strength of the beam section, only to provide sufficient resistance against the design moments and forces at the splice location. Note, however that when splices are located in a member away from a position of lateral restraint, a design bending moment about the minor axis of the section, representing second order effects, must be taken into account. Guidance can be found in Advisory Desk Notes 243, 244 and 314[6].



Beam splice



Column splices

Figure 4.1 Typical bolted cover plate splices

Stiffness and continuity

Splices must have adequate continuity about both axes. The flange plates should therefore be, at least, similar in width and thickness to the beam flanges, and should extend for a minimum distance equal to the flange width or 225 mm, on either side of the splice.

Design method

The design process for a beam splice involves the choice of the sizes of cover plates and the configuration of bolts that will provide sufficient design resistance of the joint. The process has a number of distinct stages, which are outlined below.

STEP 1

Calculate design tension and compression forces in the two flanges, due to the bending moment and axial force (if any) at the splice location. These forces can be determined on the basis of an elastic stress distribution in the beam section or, conservatively, ignoring the contribution of the web.

Calculate the shear forces, axial forces and bending moment in the web cover plates. The bending moment in the cover plates is that portion of the moment on the whole section that is carried by the web (irrespective of any conservative redistribution to the flanges – see BS EN 1993-1-8, 6.2.7.1(16)) plus the moment due to the eccentricity of the bolt group resisting shear from the centreline of the splice.

Calculate the forces in the individual bolts.

STEP 2

Determine the bolt resistances and verify their adequacy, in the flanges and in the web.

STEP 3

Verify the adequacy of the tension flange and the cover plates.

STEP 4

Verify the adequacy of the compression flange and the cover plates.

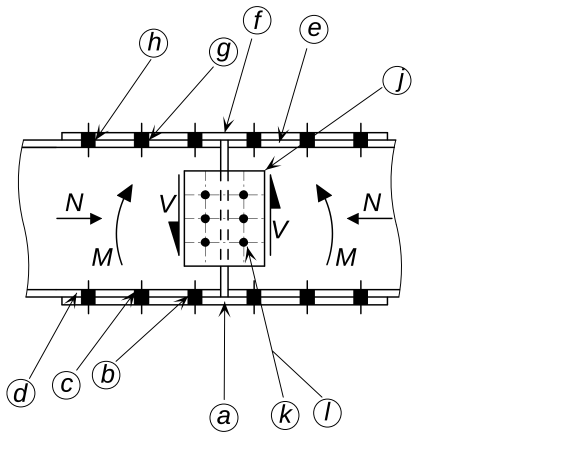
STEP 5

Verify the adequacy of the web and cover plates.

STEP 6

Ensure that there is a minimum resistance for continuity of the member.

The above STEPS involve the determination of resistance values of 11 distinct components of a bolted splice. The component resistances to be verified are illustrated in Figure 4.2.



Zone	*Ref*	*Resistance*	*STEP*
Tension	*a*	*Flange cover plate(s)*	*3*
	b	*Bolt shear*	*2*
	c	*Bolt bearing*	*2*
	d	*Flange*	*3*
Compression	*e*	*Flange*	*4*
	f	*Flange cover plate(s)*	*4*
	g	*Bolt shear*	*2*
	h	*Bolt bearing*	*2*
Shear	*j*	*Web cover plate(s)*	*5*
	k	*Bolt shear*	*5*
	l	*Bolt bearing*	*5*

Figure 4.2 Splice component resistances to be verified

4.3 DESIGN STEPS

The following pages set out the details of the 6 Design STEPS described above for a bolted cover plate splice. A worked example illustrating the procedure is given in Appendix D.

STEP 1 DISTRIBUTION OF INTERNAL FORCES

GENERAL

As noted on page 51, moment-resisting splices must be designed as slip resistant at SLS or, in less common situations, slip resistant at ULS. Resistances of the beam and cover plates must be verified at ULS in both cases. Consequently, internal forces at the splice must usually be determined at both SLS and ULS (i.e. values of N_{Ed}, M_{Ed} and V_{Ed} at ULS and $N_{Ed,ser}$, $M_{Ed,ser}$ and $V_{Ed,ser}$ at SLS).

The internal forces in the components of the beam splice, due to member forces N, M and V are as follows:

The force in each flange due to moment:

$$F_{f,M} = \left(1 - \frac{I_{y,web}}{I_y}\right)\frac{M}{(h_b - t_{fb})}$$

The force in each flange due to axial force:

$$F_{f,N} = \left(1 - \frac{A_w}{A}\right)\frac{N}{2}$$

Total force in the tension flange:

$$F_{tf} = F_{f,M} - F_{f,N}$$

Total force in the compression flange:

$$F_{cf} = F_{f,M} + F_{f,N}$$

Moment in the web (at the centreline of the splice):

$$M_w = \left(\frac{I_{y,web}}{I_y}\right)M$$

There will be an additional moment in the web at the centroid of the bolt groups equal to the product of the vertical shear and the eccentricity of the group from the centreline of the splice.

$$M_{ecc} = Ve$$

Force in the web due to axial force:

$$F_{w,N} = \left(\frac{A_w}{A}\right)N$$

Force in the web due to vertical shear:

$$F_{w,V} = V$$

where:

M = M_{Ed} or $M_{Ed,ser}$

M_{Ed} is the design moment for ULS

$M_{Ed,ser}$ is the design moment for SLS

N = N_{Ed} or $N_{Ed,ser}$

N_{Ed} is the design axial force in the member (ULS) (compression is positive)

$N_{Ed,ser}$ is the design axial force in the member (SLS) (compression is positive)

V = V_{Ed} or $V_{Ed,ser}$

V_{Ed} is the vertical design shear force (ULS)

$V_{Ed,ser}$ is the vertical design shear force (SLS)

e is the eccentricity of the bolt group from the centreline of the splice

h_b is the height of beam

t_{wb} is the beam web thickness

t_{fb} is the beam flange thickness

A_w is the area of the member web

$= (h - 2t_f)t_w$

I_y is the second moment of area about the major (y) axis of the beam

$I_{y,web}$ is the second moment of area of the web

$$= \frac{(h_b - 2t_{fb})^3 t_w}{12}$$

STEP 1 DISTRIBUTION OF INTERNAL FORCES

Forces in Bolts

In all cases, the bolts are acting in shear and consequently the subscript 'v' is used in all symbols for bolt force.

The subscripts 'Ed' and 'Ed,ser' should be added to the symbols below, to indicate forces at ULS and SLS respectively.

Forces in Flange Bolts

In the compression flange:

$$F_{\mathrm{cf,v}} = \frac{F_{\mathrm{cf}}}{n_{\mathrm{cf}}}$$

In the tension flange:

$$F_{\mathrm{tf,v}} = \frac{F_{\mathrm{tf}}}{n_{\mathrm{tf}}}$$

Usually the number of bolts in each flange splice will be the same and then the maximum design force on a bolt is:

$$F_{\mathrm{v}} = \max\left(F_{\mathrm{cf,v}}\,;F_{\mathrm{tf,v}}\right)$$

where:

n_{cf} is the number of bolts in the compression flange splice (on one side of the centreline of the splice)

n_{tf} is the number of bolts in the tension flange splice (on one side of the centreline of the splice)

Forces in Web Bolts

Forces at ULS

Vertical forces per bolt due to shear

$$F_{\mathrm{z,V}} = \frac{F_{\mathrm{w,V}}}{n}$$

Horizontal forces per bolt due to axial force

$$F_{\mathrm{x,N}} = \frac{F_{\mathrm{w,N}}}{n}$$

For a single vertical line of bolts either side of the web cover plate, the horizontal force on the top and bottom bolts due to moment is determined using:

$$F_{\mathrm{x,M}} = \frac{\left(M_{\mathrm{w}} + M_{\mathrm{ecc}}\right)z_{\mathrm{max}}}{I_{\mathrm{bolts}}}$$

The resultant force on an extreme bolt (for a single line of bolts) is thus:

$$F_{\mathrm{v}} = \sqrt{\left(F_{\mathrm{x,N}} + F_{\mathrm{x,M}}\right)^2 + \left(F_{\mathrm{z,V}}\right)^2}$$

For a double vertical line of bolts either side of the centreline of the splice, the horizontal and vertical components of the resultant force on the most highly stressed bolt due to moment is determined using:

$$F_{\mathrm{z,M}} = \frac{\left(M_{\mathrm{w}} + M_{\mathrm{ecc}}\right)x_{\mathrm{max}}}{I_{\mathrm{bolts}}}$$

$$F_{\mathrm{x,M}} = \frac{\left(M_{\mathrm{w}} + M_{\mathrm{ecc}}\right)z_{\mathrm{max}}}{I_{\mathrm{bolts}}}$$

The maximum resultant force on an extreme bolt (for a double line of bolts) is thus:

$$F_{\mathrm{v}} = \sqrt{\left(F_{\mathrm{x,V}} + F_{\mathrm{x,M}}\right)^2 + \left(F_{\mathrm{z,N}} + F_{\mathrm{z,M}}\right)^2}$$

where:

$F_{\mathrm{w,V}}$, $F_{\mathrm{w,N}}$, M_{w} and M_{ecc} are values determined above

n is the number of bolts in the web (on one side of the splice)

I_{bolts} is the second moment of the bolt group (on one side of the splice) = $\sum_{i=1}^{n}\left(x_i^2 + z_i^2\right)$ in which x_i and z_i are the x and z coordinates of i-th bolt relative to the centroid of the bolt group

x_{max} is the horizontal distance of the extreme bolt from the centroid of the group

z_{max} is the vertical distance of the extreme bolt from the centroid of the group (= 0 for a single vertical line of bolts)

STEP 2 BOLT RESISTANCES

GENERAL

The resistances of individual preloaded bolts in shear, bearing and slip resistance are given by Section 3.9 of BS EN 1993-1-8.

More conveniently, SCI publication P363 (2013 update)[7] provides resistance tables for property class 8.8 and 10.9 preloaded bolts. The relevant tables are as follows:

For preloaded hexagonal headed bolts in S275:

	Page numbers for resistance Tables	
	Grade 8.8	**Grade 10.9**
Slip at SLS	C-386	C-387
Slip at ULS	C-388	C-389

For preloaded hexagonal headed bolts in S355:

	Page numbers for resistance Tables	
	Grade 8.8	**Grade 10.9**
Slip at SLS	D-386	D-387
Slip at ULS	D-388	D-389

As well as slip resistance, the tables for SLS give the shear resistance values at ULS. Bearing resistances at ULS may be taken from the tables for non-preloaded bolts.

RESISTANCE OF BOLTS IN FLANGE SPLICES

Resistance at ULS

If the chosen bolt configuration is such that any of the edge distance, end distance, pitch or gauge dimensions is less than that in the P363 Resistance Table, or the bolts are not in normal holes, the actual resistances in bearing at ULS should be determined using Table 3.4 of BS EN 1993-1-8.

Resistance at SLS

If the preloaded bolts are not in normal holes or the surfaces are not Class A, the values of slip resistance should be calculated in accordance with clause 3.9.1 of BS EN 1993-1-8.

Long Joints

If the flange splice is 'long', the shear and bearing resistances at ULS and the slip resistance at SLS should be reduced.

A 'long' joint is one in which the length between the extreme bolts (on one side of the joint) L_j is such that:

$L_j > 15 \times d$

where:

d is the diameter of the bolt

L_j is the length between the centres of the extreme bolts in the direction of the force (see Figure 4.3)

The resistance of each bolt should then be reduced by a factor β_{Lf} given by:

$$\beta_{Lf} = 1 - \frac{L_j - 15d}{200d} \quad \text{but, } 0.75 \le \beta_{Lf} \ge 1.0$$

Note: guidance given in 3.8(1) of BS EN 1993-1-8 would suggest that the factor β_{Lf} is only applied to the shear resistance. However, it is considered that for long joints β_{Lf} should also be applied to bearing and slip resistances.

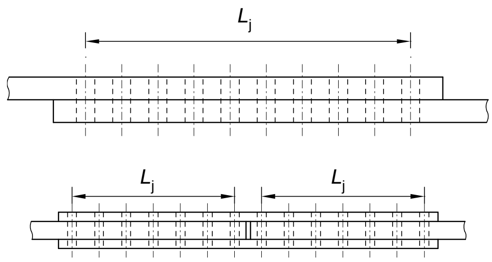


Figure 4.3 Length of joint

RESISTANCE OF BOLTS IN WEB SPLICES

Resistance at ULS

In general, the force on the most heavily loaded bolt will not be perpendicular to any edge of the cover plate or web. The distinction between edge and end is therefore difficult to apply and the use of the Resistance Tables is inappropriate.

Conservatively, the edge and end distances may be taken as those which give the lesser value of bearing resistance using Table 3.4 of BS EN 1993-1-8. This can be significantly conservative in some instances, especially when the direction of the force is away from the nearest edge.

As an alternative, the bearing resistance may be determined separately for vertical and horizontal forces, taking proper account of appropriate edge and end distances, etc. The bearing resistance under the combined vertical and horizontal forces may then be verified assuming a linear interaction, which can be expressed as:

$$\frac{F_{x,Ed}}{F_{x,b,Rd}} + \frac{F_{z,Ed}}{F_{z,b,Rd}} \le 1$$

STEP 3 RESISTANCE OF TENSION FLANGE & COVER PLATE

Resistance of Tension Flange and Cover Plate

The resistances of the flange and the cover plate are each the minimum of the resistances of the gross section and net section.

Resistance of the gross section

$$F_{\mathrm{pl,Rd}} = \frac{A_{\mathrm{g}} f_{\mathrm{y}}}{\gamma_{\mathrm{M0}}}$$

A_{g} $= b_{\mathrm{f}} \times t_{\mathrm{f}}$ for the flange

$= b_{\mathrm{fp}} \times t_{\mathrm{fp}}$ for a single cover plate

Resistance of the net section

$$F_{\mathrm{,u,Rd}} = \frac{0.9\, A_{\mathrm{net}}\, f_{\mathrm{u}}}{\gamma_{\mathrm{M2}}}$$

A_{net} $= (b_{\mathrm{f}} - 2\, d_0)\, t_{\mathrm{f}}$ for the flange

$= (b_{\mathrm{fp}} - 2\, d_0)\, t_{\mathrm{fp}}$ for a single cover plate

Additionally, if the arrangement of bolts is unorthodox, for example with only one row either side and where the edge distances are particularly large, block tearing might be possible. Guidance on evaluation of block tearing resistance is given in P358[5].

Resistance at SLS

If the preloaded bolts are not in normal holes or the surfaces are not Class A, the values of slip resistance should be calculated in accordance with Clause 3.9.1 of BS EN 1993-1-8.

where:

b_{fp} is the width of the flange cover plate

d_0 is the diameter of the bolt hole

f_{y} is the yield strength of the flange cover plate $f_{\mathrm{y,fp}}$ or of the flange $f_{\mathrm{u,bf}}$ as appropriate

f_{u} is the ultimate strength of the flange cover plate $f_{\mathrm{u,fp}}$ or of the flange $f_{\mathrm{u,bf}}$ as appropriate

t_{f} is the thickness of the flange

t_{fp} is the thickness of the flange cover plate

γ_{M0} = 1.0

γ_{M2} = 1.1 (given in the UK NA to BS EN 1993-1-1)

Note: the value for γ_{M2} is taken from the UK NA to BS EN 1993-1-1 because it relates to the area of the flange cover plate in tension.

STEP 4 RESISTANCE OF COMPRESSION FLANGE AND COVER PLATE

RESISTANCE OF COMPRESSION FLANGE AND COVER PLATE

The compression resistance of the cross sections of the flange and the cover plate may be based on the gross section, ignoring bolt holes filled with fasteners.

If the cover plate is thin and the bolt rows are widely spaced longitudinally, the buckling resistance of the cover plate should be considered.

Local buckling of the compression flange cover plate between the rows of bolts needs to be considered only if:

$$\frac{p_1}{t_{fp}} > 9\varepsilon$$

where:

$$\varepsilon = \sqrt{\frac{235}{f_{y,fp}}}$$

$$p_1 = \max\{p_{1,fp}; p_{1,j}\}$$

If this is the case, then the buckling resistance of the cover plate is given by:

$$N_{b,fp,Rd} = \frac{\chi A_{fp} f_{y,fp}}{\gamma_{M1}}$$

in which the reduction factor χ for flexural buckling is given by:

$$\chi = \frac{1}{\Phi + \sqrt{(\Phi^2 - \bar{\lambda}^2)}} \text{ but } \chi \leq 1.0$$

and

$$\Phi = 0.5 + \left(1 + \alpha\left(\bar{\lambda} - 0.2\right) + \bar{\lambda}^2\right)$$

For Class 1, 2 and 3 cross-sections

$$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} = \left(\frac{L_{cr}}{i}\right)\left(\frac{1}{\lambda_1}\right)$$

L_{cr} may be taken as $0.6p_1$

$\lambda_1 = 93.9\varepsilon$

$$i = i_z = \frac{t_{fp}}{\sqrt{12}}$$

where:

A_{fp} is the cross-sectional area of the flange cover plate (= $b_{fp}t_{fp}$)

b_{fp} is the width of the flange cover plate

t_{fp} is the thickness of the flange cover plate

$f_{y,fp}$ is the yield strength of the flange cover plate

$p_{1,fp}$ is the spacing of the bolts in the direction of the force

$p_{1,j}$ is the spacing of the bolts across the joint in the direction of the force

α is the imperfection factor
= 0.49 (for solid sections)

γ_{M1} = 1.0 (UK NA to BS EN 1993-1-1[8])

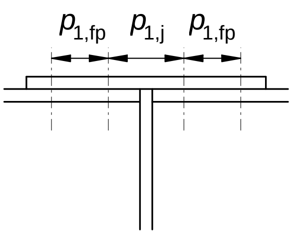


Figure 4.4 Spacing of bolts (pitch) on flange cover plate

STEP 5 RESISTANCE OF WEB SPLICES

RESISTANCE OF WEB COVER PLATE IN SHEAR

The resistance of the web cover plate is the minimum resistance of the gross shear area, net shear area and block tearing.

Resistance of the Gross Shear Area

For a single web cover plate the shear resistance of the gross area is:

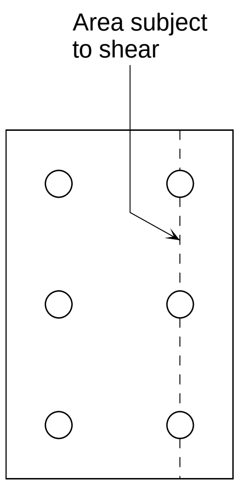
$$V_{\text{wp,g,Rd}} = \frac{h_{\text{wp}} t_{\text{wp}}}{1.27} \frac{f_{\text{y,wp}}}{\sqrt{3}\,\gamma_{\text{M0}}}$$

Resistance of the Net Shear Area

For a single web cover plate the shear resistance of the net area is:

$$V_{\text{wp,net,Rd}} = \frac{A_{\text{v,wp,net}}\left(f_{\text{u,wp}}/\sqrt{3}\right)}{\gamma_{\text{M2}}}$$

$$A_{\text{v,wp,net}} = \left(h_{\text{wp}} - n_{2,\text{wp}} d_0\right) t_{wp}$$



Here, $n_{2,\text{wp}} = 3$

Figure 4.5 Net shear area of a cover plate

Additionally, if the number of horizontal lines of bolts is small and the edge distances at the top and bottom of the web cover are particularly large, block tearing should be considered. Guidance on evaluation of block tearing resistance is given in P358[5] (for fin plate connections).

where:

d_0 is the bolt hole diameter

h_{wp} is the height of web cover plate

$n_{2,\text{wp}}$ is the number of bolt holes in the area subject to shear as shown in Figure 4.5

t_{wp} is the thickness of web cover plate

$f_{\text{y,wp}}$ is the yield strength of the web cover plate

γ_{M0} = 1.0 (UK NA to BS EN 1993-1-1)

γ_{M2} = 1.1 (UK NA to BS EN 1993-1-1)

STEP 5 RESISTANCE OF WEB SPLICES

BENDING RESISTANCE OF WEB COVER PLATE

The bending resistance of each web cover plate is given by:

$$M_{c,wp,Rd} = \frac{W_{wp}(1-\rho)f_{y,wp}}{\gamma_{M0}}$$

where:

W_{wp} is the elastic modulus for the cover plate

ρ is a reduction parameter for coexisting shear (where required)

It is recommended that the modulus of the gross cross section is used.

Cover Plates Subject to Bending, Compression and Shear

Based on clause 6.2.10 and 6.2.9.2, the following expression should be satisfied:

$$\frac{N_{wp,Ed}}{N_{wp,Rd}} + \frac{M_{wp,Ed}}{M_{c,wp,Rd}} \leq 1.0$$

An allowance for the effects of shear on the resistance of the web cover plates should be made if:

$$V_{Ed,} > \frac{V_{pl,wp,Rd}}{2}$$

where:

$V_{pl,wp,Rd}$ = min $\{V_{wp,Rd}; V_{wp,net,Rd}\}$, as determined in STEP 4

When coexisting shear must be allowed for, ρ is given by:

$$\rho = \left(\frac{2V_{Ed}}{V_{pl,wp,Rd}} - 1\right)^2$$

When coexisting shear does not need to be allowed for, $\rho = 0$

where:

$f_{y,wp}$ is the yield strength of the web cover plates

$N_{wp,Ed}$ is the design compression on the web plates

$N_{wp,Rd}$ is the compression resistance of the web plates

$M_{wp,Ed}$ is the design bending moment applied to the web plates

$M_{c,wp,Rd}$ is the bending resistance of the web plates

γ_{M0} = 1.0 (UK NA to BS EN 1993-1-1)

RESISTANCE OF THE BEAM WEB

The resistance of the beam web is the minimum resistance of the gross shear area, net shear area and block tearing.

Resistance of the Gross Shear Area

$$V_{w,g,Rd} = A_{v,w}\frac{f_{y,w}}{\sqrt{3}\,\gamma_{M0}}$$

$A_{v,w} = A - 2bt_f + (t_w + 2r)t_f$ but not less than $\eta h_w t_w$

$$h_w = h - 2(t_f + r)$$

η = 1.0 (conservatively)

Resistance of the Net Shear Area

$$V_{w,n,Rd} = \frac{A_{v,net}\left(f_{u,w}/\sqrt{3}\right)}{\gamma_{M2}}$$

$$A_{v,net} = A_{v,w} - \eta_{2,w}\, d_0 t_w$$

where:

$A_{v,w}$ is the shear area of the gross section

d_0 is the diameter of the bolt hole

h_{wp} is the height of the web cover plate

$n_{2,w}$ is the number of horizontal rows of bolts in the web

$p_{1,wp}$ is the bolt spacing in the direction of the force

t_w is the thickness of beam web

t_{wp} is the thickness of web cover plates

$f_{y,w}$ is the yield strength of the beam web

$f_{y,wp}$ is the yield strength of the web cover plates

γ_{M0} = 1.0 (UK NA to BS EN 1993-1-1)

γ_{M2} = 1.1 (UK NA to BS EN 1993-1-1)

Resistance to Block Tearing

Block tearing resistance is only applicable to the web of a notched beam. It is not applicable to the beam web in a beam splice.

STEP 6 MINIMUM REQUIREMENTS FOR CONTINUITY

GENERAL

Although there is no explicit requirement for minimum resistance or stiffness in flexural members (members resisting bending moments) in BS EN 1993-1-8, it is prudent to provide a minimum resistance for bending about the major axis and, when not restrained laterally at the splice, a minimum resistance about the minor axis.

The following guidance is based on BS EN 1993-1-8 clause 6.2.7.1(13), which specifies minimum requirements for splices in compression members.

A minimum resistance about the major axis is achieved by ensuring that the value of M_{Ed} in STEP 1 is at least equal to $0.25M_{c,y,Rd}$, the bending resistance of the beam.

A minimum resistance about the minor axis is only required if the splice is located away from a point of lateral restraint. If required, the minimum resistance in the minor axis should be taken as $0.25M_{c,z,Rd}$.

It is recommended that these minimum resistance requirements (which are really requirements to achieve a minimum stiffness) are checked independently, and in isolation, i.e. forces resulting from externally applied actions are not included in the checks of minimum resistance.

4.4 BOLTED END PLATE SPLICES

Connection details

Bolted end plate connections, as splices or as apex connections in portal frames, are effectively the beam side of the connections covered in Section 2, mirrored to form a pair. This form of connection has the advantage over the cover plate type in that preloaded bolts (and the consequent required preparation of contact surfaces) are not required. However, they are less stiff than cover plate splice details.

Typical details are shown in Figure 4.6.

Design method

The design method is essentially that described in Section 2, omitting the evaluation of column resistances. The relevant STEPS are summarised below.

STEP 1

Calculate the tensile resistances of each bolt row in the tension zone, as described in STEP 1 in Section 2.3.

The conclusion of this stage is a set of effective tension resistances, one value for each bolt row, and the summation of all bolt rows to give the total resistance of the tension zone.

STEP 2

Calculate the resistance of the compression flange.

STEP 3

If the total tension resistance exceeds the compression resistance (in STEP 2), adjust the tension forces in the bolt rows to ensure equilibrium. If a reduction is required, the force allocated to the row of bolts nearest the compression flange (i.e. with the shortest lever arm) is reduced first and then the other rows, as required, in turn.

Calculate the moment resistance. This is merely the summation of the bolt row forces multiplied by their respective lever arms, calculated from the centre of compression.

STEP 4

Calculate the shear resistance of the bolt rows. The resistance is the sum of the full shear resistance of the bolt rows in the compression zone, which are not assumed to resist tension, plus (conservatively) 28% of the shear resistance of the bolts in the tension zone.

STEP 5

Verify the adequacy of the welds in the connection.

Welds sizes are not critical in the preceding calculations. Components in compression in direct bearing need only a nominal weld, unless moment reversal must be considered.

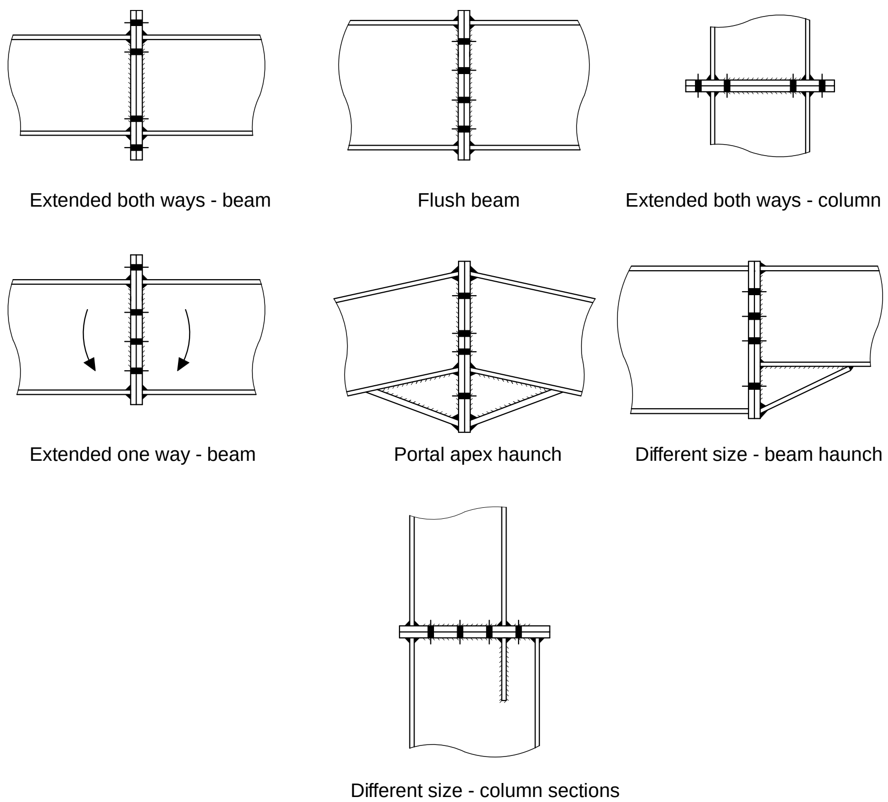


Figure 4.6 Typical bolted end plate splices

Joint stiffness and classification

It was noted in Section 1.3 that, for multi-storey unbraced frames, a well-proportioned connection following the recommendations for standardisation in this guide and designed for strength alone can be assumed to be ‘rigid’ provided that Mode 3 is the critical mode and the triangular limit is applied as described in STEP 1C. This assumption may be taken to apply to bolted end plate splices, such as those shown in Figure 4.6, but several consequent limitations must be noted:

- The end plate will need to be quite thick in order to ensure that Mode 3 is critical. This is particularly so for extended end plates.
- The moment resistance is likely to be less than that of the beam section, particularly for non-extended end plates.

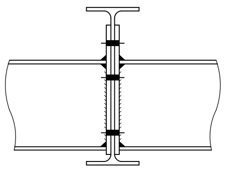
The ‘portal apex haunch’ splice shown in Figure 4.6, is regularly used in single storey portal frames and is commonly assumed to be 'rigid' for the purposes of elastic global analysis.

BS EN 1993-1-8 provides rules for evaluating joint stiffness but they are likely to prove complex for these splice connections. Determination of rotational stiffness is not covered in this guide.

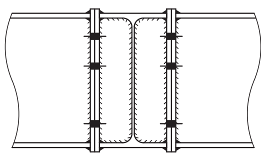
4.5 BEAM-THROUGH-BEAM MOMENT CONNECTIONS

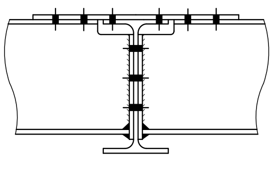
Connection details

Beam-through-beam joints are usually made using end plate connections with non-preloaded bolts; typical details are shown in Figure 4.7. Non-preloaded bolts may be used when there are only end plates, but when a cover plate is used as well, preloaded bolts should be used, to prevent slip at ULS.



Extended end plates to beam web





End plate and cover plate

Figure 4.7 Typical beam-through-beam splices

Design method

Where there is no cover plate, the design method for end plate splices (given above) may be used. Where a cover plate is used, it should be designed as for a cover plate splice; it may be assumed conservatively that the end plate bolts carry the vertical shear.

The connection between the cover plate and the supporting beam is usually only nominal, as the moment transferred in torsion to the supporting beam is normally very modest: the connection is designed to transfer moment from one side to the other. The usual limits on bolt spacing should be observed.

Joint stiffness and classification

As noted in Section 4.3, connections in multi-storey unbraced frames need to be rigid and this can be achieved only with relatively thick end plates. Beam-through-beam connections are rarely fundamental to frame stability and when they do not contribute to frame stability do not need to be rigid.

4.6 WELDED SPLICES

Connection details

Typical welded splices are shown in Figure 4.8.

Shop welded splices are often employed to join shorter lengths delivered from the mills or stockists. In these circumstances the welds are invariably made 'full strength' by butt welding the flanges and the web. Small cope holes may be formed in the web to facilitate welding of the flange.

Where the sections being joined are not from the same 'rolling' and consequently vary slightly in size because of rolling tolerances a division plate is commonly provided between the two sections. When joining components of a different serial size by this method, a web stiffener may be needed in the larger section (aligned to the flange of the smaller section), or a haunch may be provided to match the depth of the larger size

A site splice can be made with fillet welded cover plates, as an alternative to a butt welded detail. Bolts may be provided in the web covers for temporary connection during erection.

Design basis

For welded splices the general design basis is:

- In statically indeterminate frames, whether designed plastically or elastically, full strength welds should be provided to the flanges and the web.
- In statically determinate frames, splices may be designed to resist a design moment that is less than the member moment resistance, in which case:
  - The flange welds should be designed to resist a force equal to the design moment divided by the distance between flange centroids.
  - The web welds should be designed to resist the design shear.
  - If there is an axial force it should be shared between the flanges, and the welds designed for this force in addition to that due to the design moment.

The full strength requirement is needed to ensure that a splice has sufficient ductility to accommodate any inaccuracy in the design moment, arising for example, from frame imperfections, modelling approximations or settlement of supports.

Additional considerations for division plates

The division plate should be of the same grade as the components it connects and have a thickness at least equal to the flange thickness.

The division plate will need to have certified through-thickness ductility if the flanges butt welded to it are thicker than 25 mm (see advice in PD 6695-1-10, clause 3.3[9]). There are no special requirements (for through thickness ductility) for the division plate if the flanges are thinner or are fillet welded to it.

Where fillet welds are used, the weld should be continuous round the profile of the section.

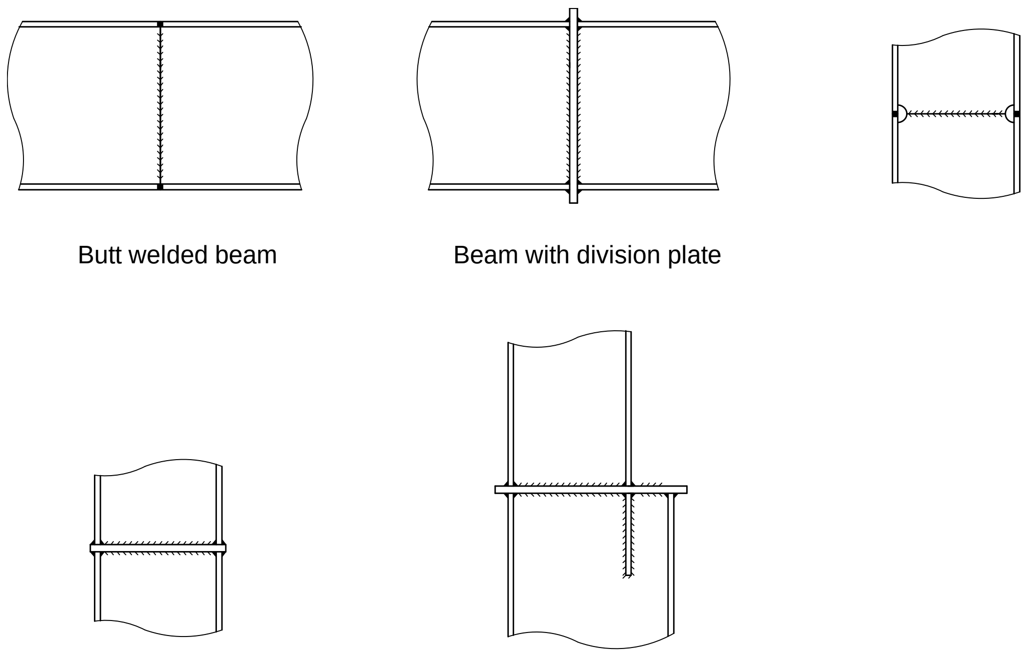


Figure 4.8 Typical welded splices

5 COLUMN BASES

5.1 SCOPE

This Section covers the design of connections which transmit moment and axial force between steel members and concrete substructures at the base of columns. The same principles may be applied to non-vertical members. Typical details of an unstiffened base plate connection are shown in Figure 5.1. Stiffened base plate connections are not covered in this guide, nor are column bases cast in pockets.

5.2 DESIGN BASIS

In terms of design, a column base connection is essentially a bolted end plate connection with certain special features:

- Axial forces are more likely to be important than is generally the case in end plate connections.
- In compression, the design force is distributed over an area of steel-to-concrete contact that is determined by the strength of the concrete and the packing mortar or grout.
- In tension, the force is transmitted by holding down bolts that are anchored in the concrete substructure.
- Unlike steel-to-steel contact in end plate connections, concrete on the tension side cannot be relied upon to generate prying forces (and thus to improve the resistance of the end plate). The base plate must be considered to bend in single curvature.

As a consequence, an unstiffened base plate tends to be very thick, by comparison with end plates of beam to column connections.

More often than not, the moment may act in either direction and symmetrical details are chosen. However, there may be circumstances (e.g. some portal frames) in which asymmetrical details may be appropriate.

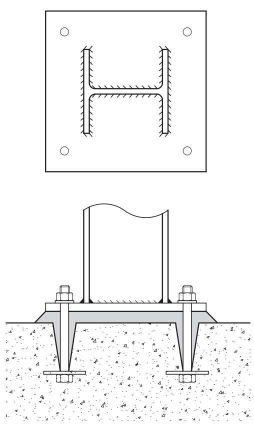


Figure 5.1 Typical column base

The connection will usually be required to transmit horizontal shear, either by friction or via the bolts. It is not reasonable to assume that horizontal shear is distributed evenly to all the bolts passing through clearance holes in the baseplate, unless washer plates are welded over the bolts in the final position. If the horizontal shear is large, a shear stub welded to the underside of the baseplate may be more appropriate, as discussed in STEP 4. In all cases, the grouting of the base is a critical operation, and demands special attention.

More complicated bases (e.g. asymmetric base plates with more than one tensile bolt row) can be treated similarly but are not discussed in detail here.

5.3 TYPICAL DETAILS

Moment-resisting base plates are less amenable to standardisation than steel-to-steel connections, as more variables are involved. However some general recommendations are given here.

Before steelwork is erected, holding down bolts are vulnerable to damage. Every care should be taken to avoid this, but it is prudent to specify with robustness in mind. Larger bolts in smaller numbers are preferred. Size should relate to the scale of the construction, including the anchorage available in the concrete.

In many cases, M24 bolts will be appropriate, but M30 is often a practical size for more substantial bases. M20 is the smallest bolt which should be considered. A preferred selection of bolt lengths and anchor plate sizes based on these diameters is given in Table 5.1.

All holding down bolts should be provided with an embedded anchor plate for the head of the bolt to bear against. Sizes of anchor plates are also given in Table 5.1; they are chosen to apply not more than 30 N/mm^2 at the concrete interface, assuming 50% of the plate is embedded in concrete. Holding down bolts are often square in cross section under the head – a square hole in the washer plate will prevent the bolt turning. If the bolt is not shaped, a keep strip welded to the underside of the washer plate adjacent to the bolt head may be used to stop the holding down bolt rotating.

When necessary, more elaborate anchorage systems (e.g. angles, or channel sections) can be designed. If a combined anchor plate for a group of bolts is used as an aid to maintaining bolt location, the anchor plates may need large holes to facilitate concrete placing.

When the moments and forces are high, it is likely that the holding down system will need to be designed in conjunction with the reinforcement in the base.

Table 5.1 Preferred sizes of holding down bolts and anchor plates

	Bolt size (property class 8.8)		
	M20	**M24**	**M30**
Length of holding down bolt (mm)	300 375 450	375 450 600	 450 600
Anchor plate size (mm x mm)	100 x 100	120 x 120	150 x 150
Anchor plate thickness (mm)	15	20	25

5.4 BEDDING SPACE FOR GROUTING

The thickness of bedding material is typically chosen to be between 20 mm and 40 mm (although in practice, the actual space is often greater). A 20 mm to 40 mm dimension gives reasonable access for grouting the bolt sleeves (necessary to prevent corrosion), and for thoroughly filling the space under the base plate. It also makes a reasonable allowance for levelling tolerances.

In base plates of size 700 mm × 700 mm or larger, 50 mm diameter holes should be provided to allow trapped air to escape and for inspection. A hole should be provided for each 0.5 m^2 of base area. If it is intended to place grout through these holes the diameter should be increased to 100 mm.

5.5 DESIGN METHOD

The design process requires an iterative approach in which a trial base plate size and bolt configuration are selected and the resistances to the range of combined axial force and moment are then evaluated. The following design process describes the evaluation of resistance for a given configuration.

STEP 1

Determine the design forces in the equivalent T-stubs for both flanges. For a flange in compression, the force may be assumed to be concentric with the flange. For a flange in tension, the force is assumed to be along the line of the holding down bolts.

STEP 2

Determine the resistance of the equivalent T-stub in compression.

STEP 3

Determine the resistance of the equivalent T-stub in tension.

STEP 4

Verify the adequacy of the shear resistance of the connection.

STEP 5

Verify the adequacy of the welds in the connection.

STEP 6

Verify the anchorage of the holding down bolts.

5.6 CLASSIFICATION OF COLUMN BASE CONNECTIONS

The rigidity of the base connection has generally greater significance on the performance of the frame than other connections in the structure. Fortunately, most unstiffened base plates are substantially stiffer than a typical end plate detail. The thickness of the base plate and pre-compression from the column contribute to this.

However, no base connection is stiffer than the concrete and, in turn, the soil to which its moment is transmitted.

Much can depend on the characteristics of these other components, which include propensity to creep under sustained loading.

The base connection cannot be regarded as 'rigid' unless the concrete base it joins is itself relatively stiff. Often this will be evident by inspection.

5.7 DESIGN STEPS

The following STEPS set out the details of the 6 STEPS described above. A worked example illustrating the procedure is given in Appendix E.

STEP 1 BASE PLATE – DESIGN FORCES IN T-STUBS

GENERAL

For combined axial force and bending at the base of a column, the design model in BS EN 1993-1-8 assumes that resistance is provided by two T-stubs in the base plate, one in tension, one in compression. The resistance to the tension T-stub is provided by holding down bolts, outside the flange of the column, and to the compression T-stub by a compression zone in the concrete, concentric with the column flange.

This model has limitations where the bending moment is either small or large, in relation to the axial force. Where the moment is small, with no net tension, there is no account taken of the compression resistance under the web. Where the moment is large, it ignores the possibility of greater moment resistance due to a compression zone that is wholly outside the column. The former can be overcome by evaluating the force in both flanges and the web and comparing them with available resistance. The latter can be overcome by selecting an eccentric compression zone (provided that the T-stub is designed for the eccentricity).

The range of situations is shown in Figure 5.2. Although only the first situation is explicitly covered in BS EN 1993-1-8, the other two situations can be designed according to the principles of the Standard.

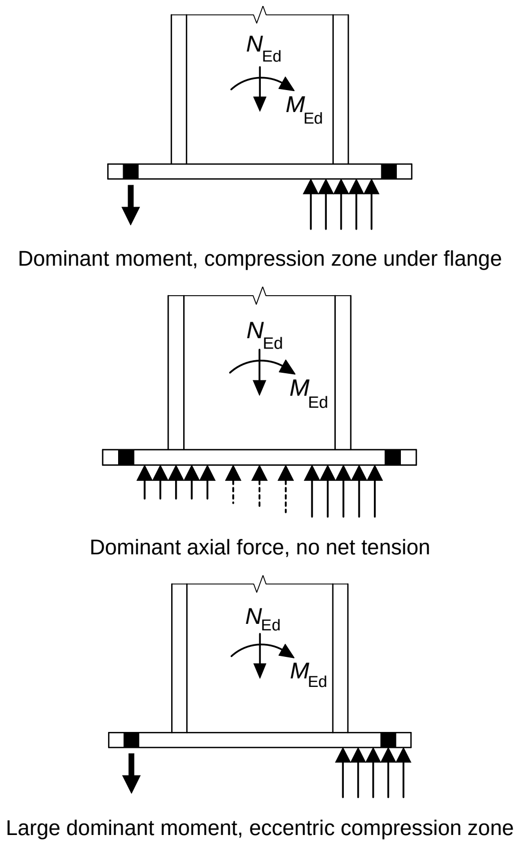


Figure 5.2 Range of design situations

STEP 1 BASE PLATE – DESIGN FORCES IN T-STUBS

FORCES IN T-STUBS

To evaluate the forces in the T-stubs when one flange is in compression and the other in tension, consider the positions of the reactions in relation to the column centreline, as shown in Figure 5.3.

Tensile reactions are resisted at the centres of the holding down bolts, at a distance z_t from the column centreline on either side.

As noted above, the design model in BS EN 1993-1-8 assumes that compressive reactions are usually resisted centrally under the column flange, at a distance z_c from the column centreline on either side. However, it may be possible to use an eccentric compression zone, in which case the z_c dimension will be greater.

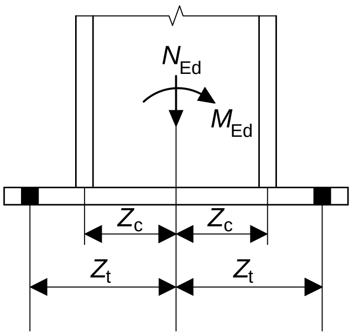


Figure 5.3 Arrangement of column base

The procedure for determining the reactions, based on these reaction positions, is as follows:

Determine the forces in the two column flanges, ignoring the force in the web, assuming that compression in the column is positive (Note, this is the opposite to the convention in BS EN 1993-1-8, Table 6.7) and that the bending moment is positive in elevation as shown above (with 'left' and 'right' corresponding to that elevation). The forces are given by:

$$N_{\mathrm{L,f}} = \frac{N_{\mathrm{Ed}}}{2} - \frac{M_{\mathrm{Ed}}}{(h - t_{\mathrm{f}})}$$

$$N_{\mathrm{R,f}} = \frac{N_{\mathrm{Ed}}}{2} + \frac{M_{\mathrm{Ed}}}{(h - t_{\mathrm{f}})}$$

Where $N_{\mathrm{L,f}}$ and $N_{\mathrm{R,f}}$ are the forces in the left and right flanges.

This will indicate, for the two sides, whether the flanges are in tension (a negative value of N) or compression (a positive value of N) and thus whether the resistance is provided by a T-stub in tension or compression.

The forces in the two T-stubs are then given by:

$$N_{\mathrm{L,T}} = \frac{N_{\mathrm{Ed}} \times z_{\mathrm{R}}}{(z_{\mathrm{L}} + z_{\mathrm{R}})} - \frac{M_{\mathrm{Ed}}}{(z_{\mathrm{L}} + z_{\mathrm{R}})}$$

$$N_{\mathrm{R,T}} = \frac{N_{\mathrm{Ed}} \times z_{\mathrm{L}}}{(z_{\mathrm{L}} + z_{\mathrm{R}})} + \frac{M_{\mathrm{Ed}}}{(z_{\mathrm{L}} + z_{\mathrm{R}})}$$

Where z_{L} and z_{R} correspond to either z_c or z_t, depending on whether the flange on that side (left or right) is in tension or compression.

STEP 2 BASE PLATE - COMPRESSION T-STUB

GENERAL

The resistance of the compression T-stub is the smaller value of the resistances of the following components:

- the resistance of the foundation in bearing
- the resistance of the base plate in bending

Resistance of Foundation

The resistance of the foundation in bearing depends on the effective area resulting from the dispersal of the compression force by the base plate in bending. The dispersal is limited by the bending resistance of the base plate, as described below, and by the physical dimensions of the base plate. The area is defined by an 'additional bearing width' around the perimeter of the steel section, as shown in Figure 5.4.

Note that this area may be physically restricted by the size of the base (in which case the centre of pressure would be inward from the flange centreline). It is also possible to ignore some or all of the area inside the flange (if the resistance is sufficient), thus increasing the lever arm between the compression zone and the holding down bolts.

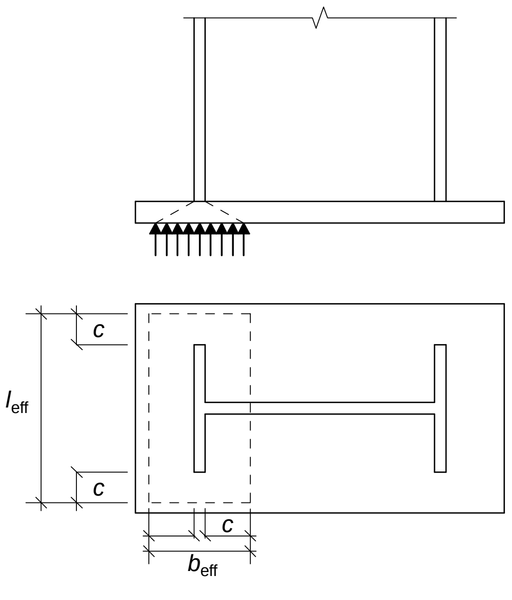


Figure 5.4 Effective bearing area

The design compression resistance of the foundation is given by:

$$F_{C,Rd} = f_{jd} b_{eff} l_{eff}$$

where:

f_{jd} is the design bearing strength of the joint, given by $f_{jd} = \beta_j \, \alpha \, f_{cd}$

β_j may be taken as $2/3$ (see note 1)

α $= \min\left[\left(1+\dfrac{d_f}{\max(h_p, b_p)}\right); \left(1+2\dfrac{e_h}{h_p}\right); \left(1+2\dfrac{e_b}{b_p}\right); 3\right]$

(see note 2)

d_f is the depth of the concrete foundation

f_{cd} $= \alpha_{cc} \dfrac{f_{ck}}{\gamma_c}$

α_{cc} = 0.85 (UK National Annex to BS EN 1992-1-1[10])

γ_c is the material factor for concrete (γ_c = 1.5 as given in the UK NA)

b_{eff} and l_{eff} are as shown in Figure 5.4

c is the limiting width of base plate (see next page)

Notes:

1) In accordance with BS EN 1993-1-8, 6.2.5(7), the use of β_j = 2/3 requires that:

   The grout has a compressive strength at least equal to 0.2 f_{cd}, and:

   The thickness of grout is less than 0.2 h_p and 0.2 b_p.

   The grout has a compressive strength at least equal to f_{cd} if over 50 mm thick.

2) Where the dimensions of the foundation are unknown, but will be orthodox (i.e. not narrow or shallow) it is reasonable to assume α = 1.5, and hence

   $$f_{jd} = f_{cd} = 0.85 \frac{f_{ck}}{\gamma_c}$$

Normal practice is to choose a bedding material (grout) at least equal in strength to that of the concrete base. It can be mortar, fine concrete or one of many proprietary non-shrink grouts. Typical concrete strengths are given in Table 5.2.

It must be emphasised that the use of high strength bedding material implies special control over the placing of the material to ensure that it is free of voids and air bubbles, etc. In the absence of such special control, a design strength limit of 15 N/mm^2 is recommended, irrespective of concrete grade.

STEP 2 BASE PLATE - COMPRESSION T-STUB

Table 5.2 Concrete strengths

	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

Resistance of Base Plate

The bending resistance of the base plate limits the additional width *c*, assuming that the width *c* is a cantilever subject to a uniform load equal to the design bearing strength of the joint. Since the bending resistance depends on the thickness and yield strength of the base plate, the limiting additional width is given by:

$$c = t\left(\frac{f_y}{3f_{jd}\lambda_{M0}}\right)^{0.5}$$

Resistance of Column Flange

The compression resistance of the column flange and web in the compression zone is given by:

$$F_{c,fc,Rd} = \frac{M_{c,Rd}}{h_c - t_{fc}}$$

STEP 3 BASE PLATE - TENSION T-STUB

GENERAL

The design of the tension T-stub is similar to that for a beam to column connection, except that there is no 'column side' to be verified (instead the anchorage of the holding down bolts must be verified).

The T-stub model in BS EN 1993-1-8 is generally expressed only for two bolts in each row. The expressions for equivalent length of T-stub and tension resistance must be modified when there are more than two bolts across the width of the base plate.

The guidance below is described only for a single row of bolts outside the tension flange. If additional bolts are provided between the flanges, these can be taken into account by adapting the guidance in Section 2 for end plate connections.

The resistance of the tension T-stub is the smallest value of the resistances of the following components:

- The resistance of the base plate in bending.
- The resistance of the holding down bolts.
- The resistance of the column flange and web in tension.

Resistance of Base Plate in Bending

The design procedure is similar in principle to STEP 1A for unstiffened extensions of bolted end plates except that no prying is assumed and there is a single expression for resistance in place of the separate expressions in Modes 1 and 2.

The design resistance in bending is given by:

$$F_{t,pl,,Rd} = \frac{2M_{pl,1,Rd}}{m_x}$$

where:

$M_{pl,1,Rd}$ is given by: $M_{pl,1Rd} = \dfrac{0.25\Sigma \ell_{eff,1} t_{pl}^2 f_y}{\gamma_{M0}}$

$\ell_{eff,1}$ is the effective length of the equivalent T-stub

t_{pl} is the thickness of the base plate

f_y is the yield strength of the base plate

m_x is the distance from the bolt centreline to the fillet weld to the column flange (measured to a distance into the fillet equal to 20% of its size)

The effective length of the T-stubs can be determined from Table 5.3. If the corner bolts are located outside the tips of the column flanges, the designer should check whether the yield line patterns shown in the Table are still appropriate.

Resistance of bolts in tension

With a single row of bolts, the design resistance is given simply by:

$F_{t,pl,,Rd} = n\, F_{t,,Rd}$

where:

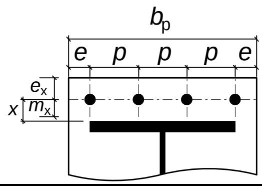
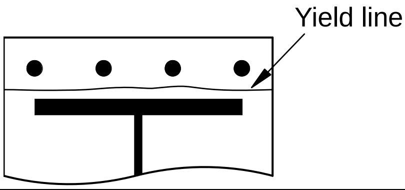
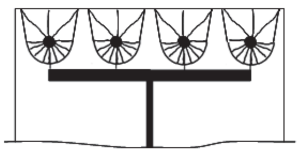
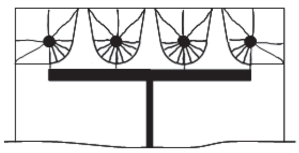
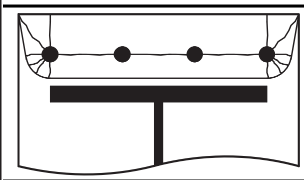
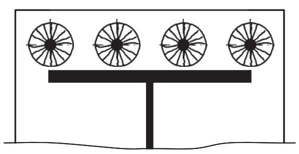
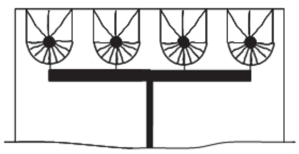
n is the number of bolts

$F_{t,,Rd}$ is the design tensile resistance of a single bolt

If there is a second row of bolts, inside the tension flange, the resistance of those bolts should be limited by a triangular distribution from the centre of rotation, as for bolts in end plate connections when the outer tension row resistance is determined by Mode 3 failure (see STEP 1C in Section 2.5).

STEP 3 BASE PLATE - TENSION T-STUB

Table 5.3 Effective lengths for base plate T-stubs

Single row of *n* bolts	
Non-circular patterns	
	Single curvature $\ell_{eff,nc} = \frac{b_p}{2}$
	Individual end yielding $\ell_{eff,nc} = \frac{n}{2}(4m_x + 1.25e_x)$
	Corner yielding of outer bolts, individual yielding between $\ell_{eff} = 2m_x + 0.625e_x + e + (n-2)(2m_x + 0.625e_x)$
	Group end yielding $\ell_{eff} = 2m_x + 0.625e_x + \frac{(n-1)p}{2}$
Circular patterns	
	Individual circular yielding $\ell_{eff,cp} = n\pi m_x$
	Individual end yielding $\ell_{eff,cp} = \frac{n}{2}(\pi m_x + 2e)$
Circular group patterns are not shown here as individual circular yielding will have lesser ℓ_{eff} in all practical situations	Circular group yielding
n is the number of bolts (4 bolts shown for illustrative purpose)	

STEP 4 BASE PLATE - SHEAR

GENERAL

In principle, shear may be transferred between the base plate and concrete in four ways:

- By friction. A resistance of 0.3 times the total compression force may be assumed.
- In bearing, between the shafts of the bolts and the base plate and between the bolts and the concrete surrounding them.
- Directly, by installing tie bars
- Directly, by setting the base plate in a shallow pocket which is filled with concrete
- Directly, by providing a shear key welded to the underside of the plate.

The simplest option is to demonstrate that friction alone is sufficient to transfer the shear. When friction alone is insufficient, common practice in the UK is to assume the shear is transferred via the holding down bolts. Although experience has demonstrated that this is generally satisfactory, designers may need to consider special arrangements if the base is subject to high shear.

- The shear is unlikely to be shared equally among the bolts. As the bolts are in clearance holes, some may not be in contact with the plate at all. This may be overcome by assuming that not all the bolts are effective. Alternatively, washer plates with precise holes can be positioned over the bolts, and site welded to the base, ensuring that the bolts are all in bearing and that load is distributed evenly.
- The shear, applied through the base, or washer plates, may be at a significant distance above the concrete. If the bolts are subject to bending (because, for example, the grout is incomplete), the resistance is severely reduced.
- The resistance of the bolts in the base assumes that the bolts are cast solidly into the concrete. The assumption of cast in bolts needs to be realised in practice, demanding that the entire grouting operation is undertaken with care, including proper preparation of the base, cleanliness, mixing and careful placing of grout.

The position of bolts in the foundation needs careful consideration - bolt resistance will be reduced close to an edge, for example.

High Shear

If the shear force cannot be transferred by friction or by the holding down bolts, a number of approaches are available to transfer the base shear. Significant shears may be transferred by:

- A shear stub welded to the underside of the base, located in a pocket in the foundation.
- Embedding the column in the foundation
- Installing tie bars (or similar) between the column and (for example) a concrete floor slab
- Casting a slab around the column

Each of these solutions requires liaison between the steel designer and others, to ensure that the foundations and other elements are appropriately detailed and reinforced.

Holding Down Bolts in Shear

The bolts should be verified in shear, in bearing on the base plate and in bearing on the concrete.

When bolts are solidly cast into concrete the bolts can be relied upon to resist shear. The design may be based on an effective bearing length in the concrete of 3*d* and an average bearing stress of $2f_{cd}$, where f_{cd} is the design compressive strength of the foundation concrete (or the grout, whichever is weaker). When this approach is used, all bolts must be completely surrounded by reinforcement and bolts whose centre is less than 6*d* from the edge of the concrete in the direction of loading should not be considered.

Shear and Tension

Holding down bolts will invariably be subject to combined shear and tension. This condition must be checked by verifying:

$$\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1.4\,F_{t,Rd}} \leq 1.0$$

Shear Stubs

Shear stubs are commonly I sections welded to the underside of the base plate, as shown in Figure 5.5.

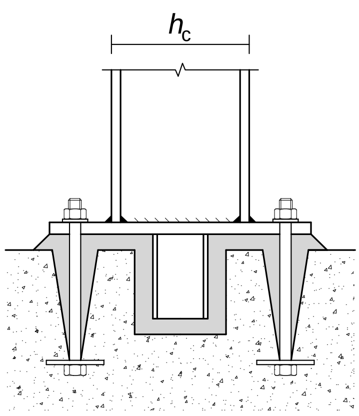


Figure 5.5 Shear stub details: I section

STEP 4 BASE PLATE - SHEAR

Rules of thumb for sizing an I section shear stub are that the section depth of the stub should be approximately 0.4 × the column section depth. The effective depth should be greater than 60 mm, but not more than 1.5 × the section depth of the stub.

For an I section, the slenderness of the flange outstand should be limited such that $b_n/t_{fn} \leq 20$.

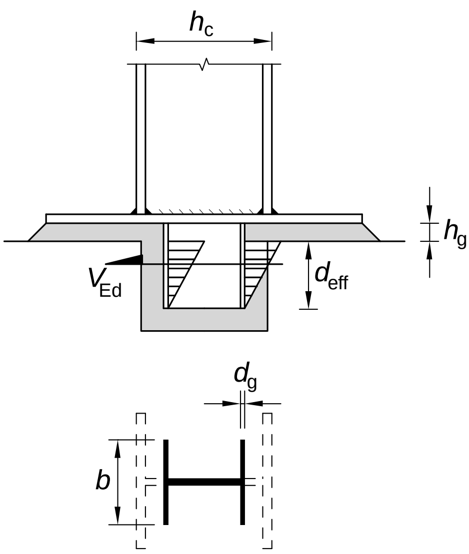


Figure 5.6 Shear stubs – design model

The design model is straightforward, as shown in Figure 5.6. The load is assumed to be transferred in bearing on the vertical faces of the stub. A triangular distribution is assumed, and the nominal grout space is ignored, to allow for any inconsistencies in that zone. The maximum bearing stress is taken as f_{cd}, the design compressive strength of the concrete (or bedding, whichever is weaker) leading to a resistance as follows:

For a two flanged section (typically an I- or H section):

$V_{Rd} = b_s d_{eff} f_{cd}$

The eccentricity between the applied shear and the horizontal reaction on the stub causes a secondary moment, $M_{sec,Ed}$ assumed to be resisted by a couple comprising a compression force under one flange and (conservatively) a tension concentric with the shear stub, as shown in Figure 5.7.

$$M_{sec,Ed} = V_{Ed}\left(h_g + d_{eff}/3\right)$$

The force in the flange of the stub, $N_{sec,Ed}$ is given by:

$$N_{sec,Ed} = M_{sec,Ed} / \left(h_s - t_{fs}\right)$$

The resistance of the flange of the stub is given by:

$$b_s t_{fs} f_{ys} / \gamma_{M0}$$

The weld between the flanges of the stub and the underside of the base plate should be designed as a transverse weld for the design force $N_{sec,Ed}$. The welds between the web of the stub and the underside of the base plate should be designed as a longitudinal weld for the design shear force, $V_{Ed.}$

The web of the column should be checked for the concentrated force applied by the flange of the stub, based on an effective breadth, b_{eff}, given by:

$$b_{eff} = t_{fs} + 2s + 5t_p$$

where:

s is the leg length of the weld to the flanges of the stub

t_p is the thickness of the base plate

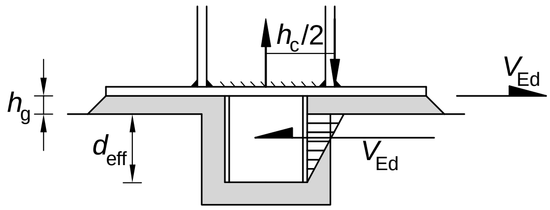


Figure 5.7 Secondary moment

Shear resistance of stub

The shear resistance of the stub must be verified. For an I section, the shear resistance of the section may be calculated following the normal rules for section resistance, as follows:

$$V_{Rd} = \frac{A_{vs} f_{ys}}{\gamma_{M0}\sqrt{3}}$$

where:

A_{vs} is the shear area of the shear stub

f_{ys} is the yield strength of the shear stub

STEP 4 BASE PLATE - SHEAR

GENERAL

It is generally convenient to assume that the flanges carry the bending moments and the web carries the shear, and design the welds accordingly.

Bending moments on bases may generally act in both directions, meaning there is no “compression” flange – the welds to both flanges must be designed for the tension in the flange. If a compression case is considered, a sawn end on the column member is generally sufficient for contact in direct bearing and only nominal welds (6 mm or 8 mm) would be required.

Flange Welds

The design force in the tension flange should be taken as the lesser of:

- The tension resistance of the flange, $b \times t_{fc} \times f_y$
- The force in the flange, taken as the force in the flange due to the moment, reduced by the effect of any compression, $\frac{M_{Ed}}{h_c - t_f} - N_{Ed}\frac{bt_{fc}}{A}$

Web Welds

The welds to the web should be designed to carry the base shear.

STEP 5 BASE PLATE - WELDS

GENERAL

Normally the objective is to ensure that the anchorage is as strong as the bolt that is used.

In principle, anchorage can be developed either by bond along the embedded length or by bearing via an anchor plate at the end of the bolt. However, reliance on bond may only be used for bolts with a yield strength up to 300 N/mm^2 (i.e. only for property class 4.6 bolts). For moment-resisting bases, property class 8.8 bolts will normally be used, for economy, and thus anchor plates or other load distributing members within the concrete will be used.

Commonly used bolt sizes and lengths are given in Table 5.1.

Individual anchor plates are generally square and of the approximate sizes given in Table 5.1. Individual anchor plates are commonly used but, when necessary, more elaborate anchorage systems, such as back-to-back channel sections, can be designed.

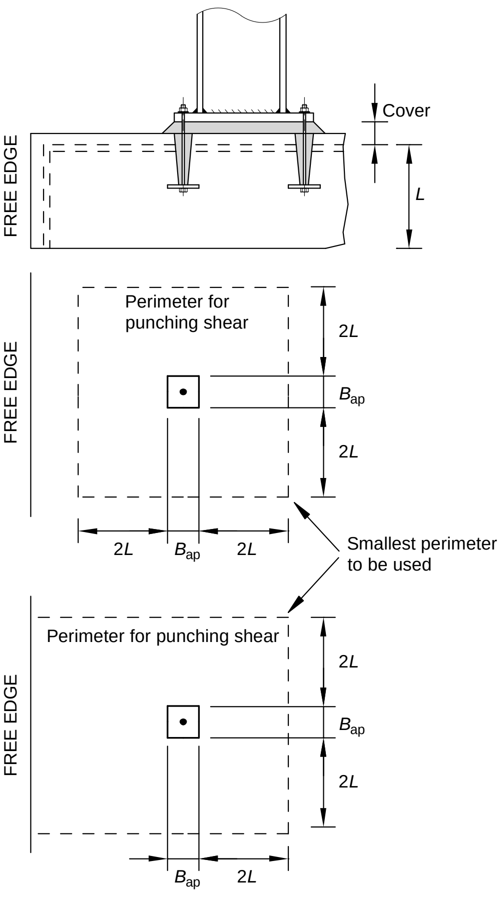


Figure 5.8 Basic control perimeter for a single bolt

If a combined anchor plate for a group of bolts is used as an aid to maintaining bolt location, such plates may need large holes to facilitate concrete placing.

If combined anchor plates are made to serve two or more bolts, a similar area should be provided symmetrically disposed about each bolt location.

The design resistance of the anchorage should be based on determination of resistance to punching shear in accordance with BS EN 1992-1-1. The following procedure is based on that Part.

Punching shear is considered at a basic control perimeter a distance outside the loaded area that is twice the effective depth of a slab. For a column base the effective depth is taken as the length of the anchor bolts, as shown in Figure 5.8. The perimeter will be reduced by proximity to a free edge.

If bolts are placed such that their basic control perimeters overlap, they should be checked as a group with a single perimeter as shown in Figure 5.9.

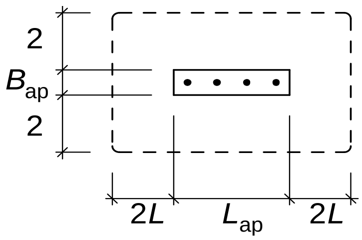


Figure 5.9 Basic control perimeter for a bolt group

Basic requirement is:

$V_{Rd,cs} > V_{Ed}$

where:

V_{Ed} is the design shear force, taken as the total tension force in the bolts being considered within the control perimeter

$V_{Rd,cs}$ is the resistance to punching shear, determined in accordance with BS EN 1992-1-1, Section 6.4.

6 REFERENCES

1 BS EN 1993-1-8:2005
Eurocode 3: Design of steel structures. Part 1-8: Design of joints (incorporating corrigenda December 2005, September 2006, July 2009 and August 2010)
BSI, 2010

2 NA to BS EN 1993-1-8:2005
UK National Annex to Eurocode 3: Design of steel structures. Part 1-8: Design of joints
BSI, 2008

3 Joints in Steel Construction – Moment Connections (P207/95)
SCI and BCSA, 1997

4 National Structural Steelwork Specification for Building Construction 5th Edition, CE Marking Version (BCSA Publication No. 52/10)
BCSA, 2010

5 Joints in Steel Construction – Simple Joints to Eurocode 3 (P358)
SCI and BCSA, 2011

6 AD 243 Splices within unrestrained lengths
AD 244 Second order moments
AD 314 Column splices and internal moments
(All available from www.steelbiz.org)

7 Steel Building Design: Design Data (Updated 2013) (P363)
SCI and BCSA, 2013

8 NA to BS EN 1993-1-1:2005
UK National Annex to Eurocode 3: Design of steel structures. Part 1-1: General rules and rules for buildings
BSI, 2008

9 PD 6695-1-10:2009
Recommendations for the design of structures to BS EN 1993-1-10
BSI, 2009

10 NA to BS EN 1992-1-1:2004
UK National Annex to Eurocode 2: Design of Concrete Structures. General rules and rules for buildings (incorporating National Amendment No. 1)
BSI, 2009

APPENDIX A EXAMPLES OF DETAILING PRACTICE

Figure A.1 and Figure A.2 show typical examples of end plates for universal beam sections. The bolt pitches (vertical spacing) and gauge (horizontal spacing) shown are 'industry standard' dimensions that are widely adopted.

For beams sizes of 533 UKB and above, a 25 thick end plate would normally be used, with M24 bolts.

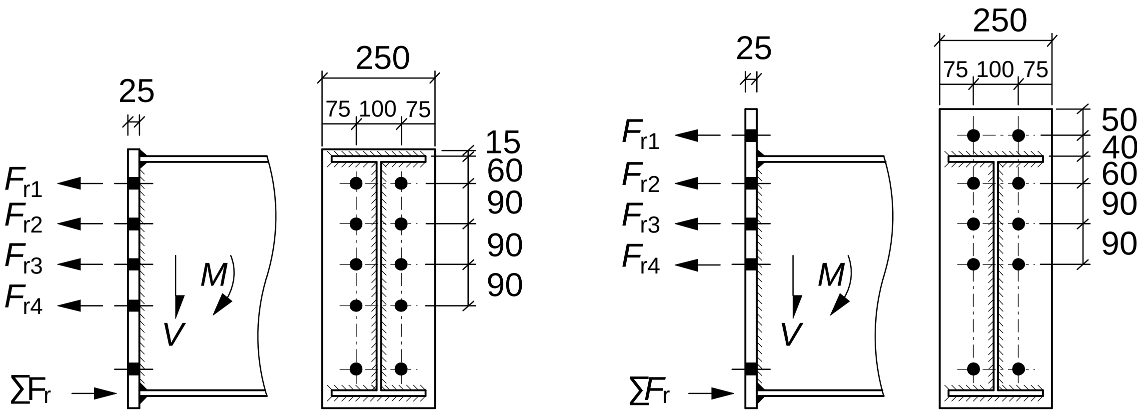


Figure A.1   Recommended connection details for beam sizes 533 UKB and above

For beams sizes of 457 UKB and below, a 20 mm thick end plate would normally be used, with M20 bolts. Figure A.2 shows configurations with three rows of bolts in the tension zone.

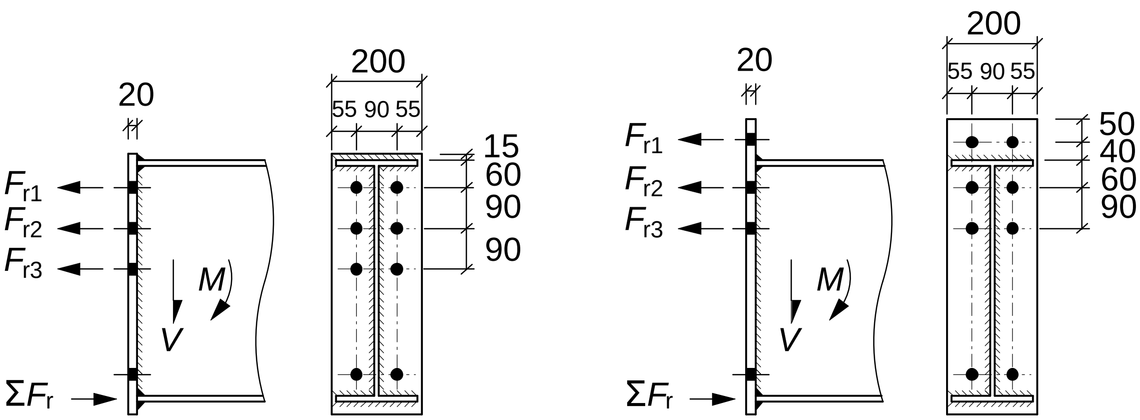


Figure A.2   Recommended connection details for beam sizes 457 UKB and below

APPENDIX B INDICATIVE CONNECTION RESISTANCES

Figure B.1 and Figure B.2 show indicative moment resistances for various beam sizes in S355 steel, with S275 end plates. Figure B.1 covers beams with an extended end plate, of the form shown in Figure A.1. Figure B.2 covers full depth end plates, of the form shown in Figure A.2.

In all cases, the details of bolt diameters, end plate thickness and connection geometry follow the recommendations shown in Appendix A. Note that for 533 UKB and above, the connection is configured with M24 bolts and a 25 by 250 mm end plate. With the exception of the two smallest beam sizes, all connections have three rows of bolts. Particularly for the deeper beams, an increased resistance could be achieved by increasing the number of bolt rows.

Two resistances are shown – for the heaviest beam in the serial size and for the lightest. In every case, the calculated resistance assumes that nothing on the 'column side' will govern – meaning the beam side resistances can be achieved. Because of the end plate thickness, a triangular limit has been applied when determining the bolt row resistances.

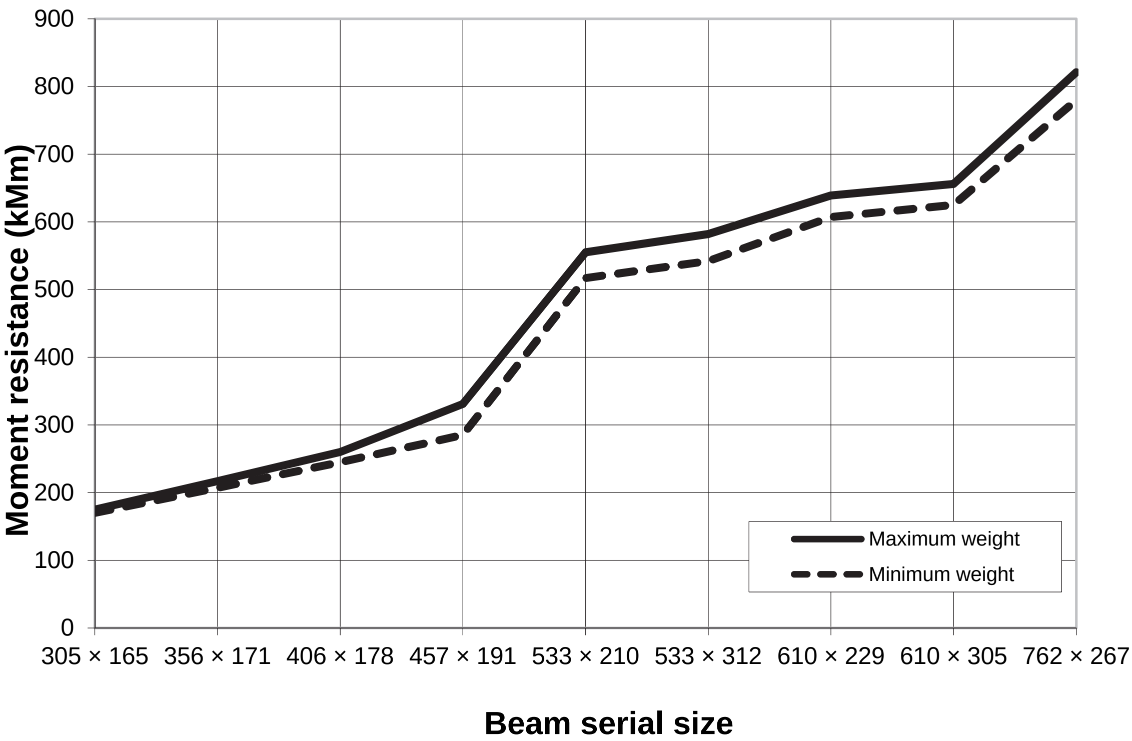


Figure B.1 Moment resistance of extended end plate connection, with one bolt row above the beam and two bolt rows below the top flange

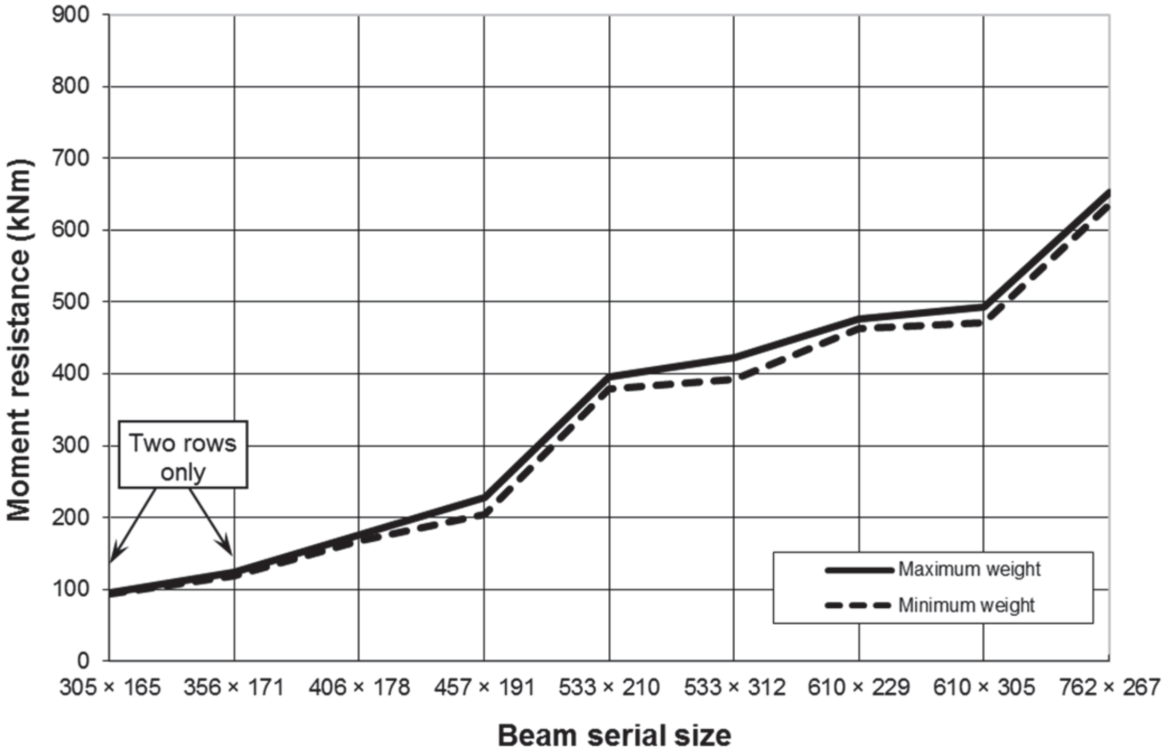


Figure B.2 Moment resistance of end plate connection with three bolt rows below the beam flange

APPENDIX C WORKED EXAMPLES – BOLTED END PLATE CONNECTIONS

Five worked examples are presented in this Appendix:

Example C.1 Bolted end plate connection to a column (unstiffened)

Example C.2 Connection with column web compression stiffener

Example C.3 Connection with column web tension stiffener

Example C.4 Connection with supplementary web plates to column

Example C.5 Connection with Morris stiffener to column

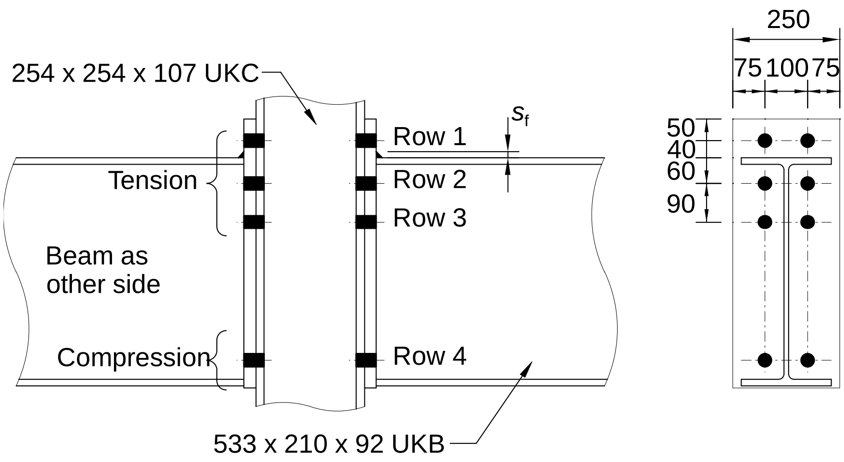
Each example follows the recommendations in the main text. Additionally, references to the relevant clauses, Figures and Tables in BS EN 1993-1-8 and its UK National Annex are given where appropriate; these are given simply as the clause, Figure or Table number. References to clauses, etc. in other standards are given in full. References to Tables or Figures in the main text are noted accordingly; references to STEPS are to those in Section 2 of the main text.

CALCULATION SHEET	
	Job No. *CDS 324* — *Sheet 1 of 23*
	Title *Example C.1 – Bolted end plate connection (unstiffened)*
	Client
	Calcs by *MEB* — Checked by *DGB* — Date *Nov 2012*

JOINT CONFIGURATION AND DIMENSIONS

Determine the resistances for the extended end plate connection shown below.

It may be assumed that the design moments in the two beams are equal and opposite.



Column	254 × 254 × 107 UKC in S275
Beam	533 × 210 × 92 UKB in S275
End plate	670 × 250 × 25 in S275
Bolts	M24 class 8.8
Welds	Fillet welds. Assumed weld sizes:
	s_f = 12 mm
	s_w = 8 mm

References to clauses, etc. are to BS EN 1993-1-8 and its UK NA, unless otherwise stated.

Title	Example C.1 – Bolted end plate connection (unstiffened)	Sheet	2 of 23

DIMENSIONS AND SECTION PROPERTIES

Column

From data tables for 254 × 254 × 107 UKC: — P363

Depth	h_c	= 266.7 mm
Width	b_c	= 258.8 mm
Web thickness	t_{wc}	= 12.8 mm
Flange thickness	t_{fc}	= 20.5 mm
Root radius	r_c	= 12.7 mm
Depth between fillets	d_c	= 200.3 mm
Area	A_c	= 136 cm^2

Beams

From data tables for 533 × 210 × 92 UKB: — P363

Depth	h_b	= 533.1 mm
Width	b_b	= 209.3 mm
Web thickness	t_{wb}	= 10.1 mm
Flange thickness	t_{fb}	= 15.6 mm
Root radius	r_b	= 12.7 mm
Depth between fillets	d_b	= 476.5 mm
Area	A_b	= 117 cm^2

End plates

Depth	h_p	= 670 mm
Width	b_p	= 250 mm
Thickness	t_p	= 25 mm

Bolts

M24 non preloaded class 8.8 bolts

Diameter of bolt shank	d	= 24 mm
Diameter of hole	d_0	= 26 mm
Shear area	A_s	= 353 mm^2
Diameter of washer	d_w	= 41.6 mm

Bolt spacings

Column

End distance	(no end distance)	
Spacing (gauge)	w	= 100 mm
Edge distance	e_c	= 0.5 × (258.8-100) = 79.4 mm
Spacing row 1-2	p_{1-2}	= 100 mm
Spacing row 2-3	p_{2-3}	= 90 mm

End plate

End distance	e_x	= 50 mm
Spacing (gauge)	w	= 100 mm
Edge distance	e_p	= 75 mm
Spacing row 1 above beam flange	x	= 40 mm
Spacing row 1-2	p_{1-2}	= 100 mm
Spacing row 2-3	p_{2-3}	= 90 mm

Title	Example C.1 – Bolted end plate connection (unstiffened)	Sheet 3 of 23

MATERIAL STRENGTHS

Steel strength

For buildings that will be built in the UK the nominal values of the yield strength (f_y) and the ultimate strength (f_u) for structural steel should be those obtained from the product standard. Where a range is given, the lowest nominal value should be used. — BS EN 1993-1-1 NA.2.4

S275 steel — BS EN 10025-2 Table 7

For $t \leq 16$ mm	f_y	$= R_{eH} = 275$ N/mm^2
For 16 mm $< t \leq 40$ mm	f_y	$= R_{eH} = 265$ N/mm^2
For 3 mm $\leq t \leq 100$ mm	f_u	$= R_{eH} = 410$ N/mm^2

Hence:

Beam yield strength	$f_{y,b}$	= 275 N/mm^2
Column yield strength	$f_{y,c}$	= 265 N/mm^2
End plate yield strength	$f_{y,p}$	= 265 N/mm^2

Bolt strength

Table 3.1

Nominal yield strength	f_{yb}	= 640 N/mm^2
Nominal ultimate strength	f_{ub}	= 800 N/mm^2

PARTIAL FACTORS FOR RESISTANCE

Structural steel

BS EN 1993-1-1 NA.2.15

γ_{M0} = 1.0

γ_{M1} = 1.0

γ_{M2} = 1.1

Parts in connections

BS EN 1993-1-8 Table NA.1

γ_{M2} = 1.25 (bolts, welds, plates in bearing)

Title	Example C.1 – Bolted end plate connection (unstiffened)	Sheet 4 of 23

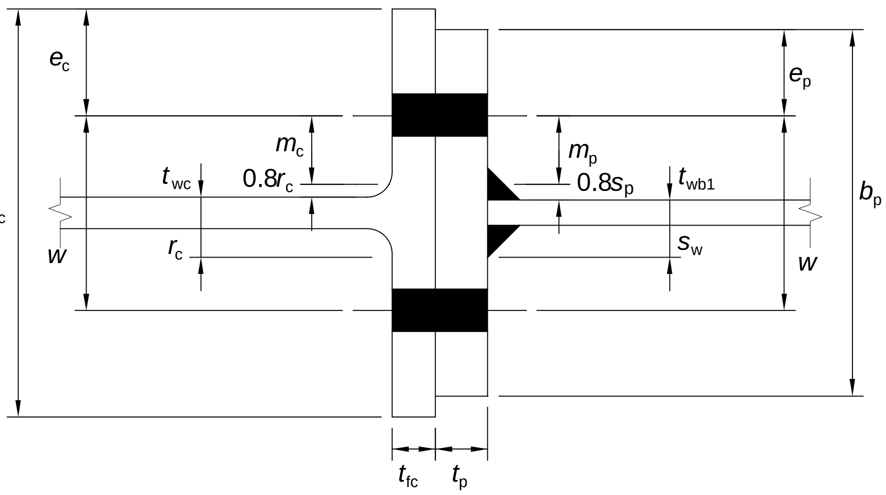
TENSION ZONE T-STUBS

When prying forces may develop, the design tension resistance ($F_{T,Rd}$) of a T-stub flange should be taken as the smallest value for the 3 possible failure modes in Table 6.2. — 6.2.4.1(6)

BOLT ROW 1

Column flange in bending (no backing plate) — STEP 1

Consider bolt row 1 to be acting alone. The key dimensions are shown below.



Determine e_{min}, m and ℓ_{eff} for the unstiffened column flange — 6.2.4.1(2))

$$m = m_c = \frac{w - t_{wc} - 2 \times 0.8\, r_c}{2} = \frac{100 - 12.8 - (1.6 \times 12.7)}{2} = 33.4 \text{ mm}$$

e_{min} = min(e_p ; e_c) = min(75 ; 79.4) = 75 mm — Figure 6.8

For Mode 1, $\ell_{eff,1}$ is the lesser of $\ell_{eff,nc}$ and $\ell_{eff,cp}$ — Table 6.6

$\ell_{eff,cp}$ = $2\pi m$ — Table 2.2(e) in STEP 1A

= $2\pi \times 33.4$ = 210 mm

$\ell_{eff,nc}$ = $4m + 1.25e$

= $4 \times 33.4 + 1.25 \times 79.4$ = 233 mm

As 210 < 233

$\ell_{eff,1}$ = $\ell_{eff,cp}$ = 210 mm

For failure Mode 2, $\ell_{eff,2} = \ell_{eff,nc}$ — STEP 1

Therefore $\ell_{eff,2}$ = 233 mm

<u>*Mode 1 resistance*</u>

For Mode 1, without backing plates, using Method 2:

$$F_{T,1,Rd} = \frac{(8n - 2e_w)M_{pl,1,Rd}}{2mn - e_w(m + n)}$$ — Table 6.2

where:

m = m_c = 33.4 mm

n = e_{min} but $\leq 1.25m$

$1.25m = 1.25 \times 33.4$ = 41.8 mm

As 41.8 < 75:

n = 41.8 mm

Title Example C.1 – Bolted end plate connection (unstiffened) Sheet 5 of 23

$$M_{pl,1,Rd} = \frac{0.25\, \Sigma\, \ell_{eff,1}\, t_f^2 f_y}{\gamma_{M0}}$$

f_y = $f_{y,c}$ = 265 N/mm^2

t_f = t_{fc} = 20.5 mm

$$M_{pl,1,Rd} = \frac{0.25 \times 210 \times 20.5^2 \times 265}{1.0} = 5850 \times 10^3 \text{ Nmm}$$

$$e_w = \frac{d_w}{4}$$

d_w is the diameter of the washer, or the width across points of the bolt head or nut, as relevant

Here, d_w = 39.55 mm (across the bolt head) — P358

Therefore, $e_w = \dfrac{39.55}{4}$ = 9.9 mm

Therefore, $F_{T,1,Rd} = \dfrac{(8 \times 41.8 - 2 \times 9.9) \times 5850 \times 10^3}{2 \times 33.4 \times 41.8 - 9.9 \times (33.4 + 41.8)} \times 10^{-3}$ = 898 kN

Mode 2 resistance

For Mode 2 — Table 6.2

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

where:

$$M_{pl,2,Rd} = \frac{0.25\, \Sigma\, \ell_{eff,2}\, t_f^2\, f_y}{\gamma_{M0}}$$

$$= \frac{0.25 \times 233 \times 20.5^2 \times 265}{1.0} = 6490 \times 10^3 \text{ Nmm}$$

$\Sigma\, F_{t,Rd}$ is the total value of $F_{t,Rd}$ for all the bolts in the row, where:

$F_{t,Rd}$ for a single bolt is: — Table 3.4

$$F_{t,Rd} = \frac{k_2\, f_{ub}\, A_s}{\gamma_{M2}} \qquad k_2 = 0.9$$

$$F_{t,Rd} = \frac{0.9 \times 800 \times 353}{1.25} = 203 \times 10^3 \text{ N}$$

For 2 bolts in the row, $\Sigma\, F_{t,Rd} = 2 \times 203 \times 10^3 = 406 \times 10^3$ N

Therefore, for Mode 2

$$F_{T,2,Rd} = \frac{2 \times 6490 \times 10^3 + 41.8 \times 406 \times 10^3}{33.4 + 41.8} \times 10^{-3} = 398 \text{ kN}$$

Mode 3 resistance (bolt failure)

$F_{T,3,Rd} = \Sigma\, F_{t,Rd}$ = 406 kN — Table 6.2

Resistance of column flange in bending

$F_{t,fc,Rd} = \min\{ F_{T,1,Rd}\, ;\, F_{T,2,Rd}\, ;\, F_{T,3,Rd} \}$ = 398 kN — 6.2.4.1(6)

Column web in transverse tension

STEP 1B

The design resistance of an unstiffened column web to transverse tension is determined from:

6.2.6.3(1) Eq (6.15)

$$F_{t,wc,Rd} = \frac{\omega\, b_{eff,t,wc}\, t_{wc}\, f_{y,wc}}{\gamma_{M0}}$$

ω is a reduction factor that allows for the interaction with shear in the column web panel

The transformation factor β is used to determine the expression to be used when calculating a value for ω. Here, with equal and opposite design moments from the two beams, $\beta = 0$

5.3
Table 2.5 in STEP 1B

Therefore $\omega = 1.0$

Table 6.3

For a bolted connection the effective width of the column web in tension ($b_{\mathrm{eff,t,wc}}$) should be taken as the effective length (ℓ_{eff}) of the equivalent T stub representing the column flange. Here, as the resistance of Mode 2 (398 kN) is less than that of Mode 1 (898 kN) the effective width of the column web is considered to be:

6.2.6.3(3)

$b_{\mathrm{eff,t,wc}} = \ell_{\mathrm{eff,2}} = 233$ mm

$f_{\mathrm{y,wc}} = f_{\mathrm{y,c}} = 265$ N/mm^2

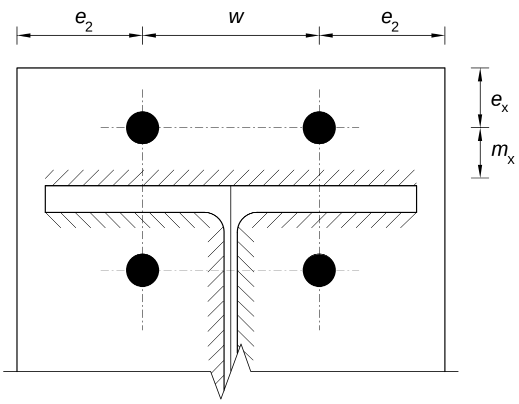
Thus,

$$F_{\mathrm{t,wc,Rd}} = \frac{1.0 \times 233 \times 12.8 \times 265}{1.0} \times 10^{-3} = 790 \text{ kN}$$

End plate in bending

STEP 1

Bolt row 1 is outside the tension flange of the beam. The key dimensions for the T-stub are shown below



The values of m_{x}, e_{x} and e for the T-stub are:

Figure 6.10

$e = e_{\mathrm{p}} = 75$ mm

$e_{\mathrm{x}} = 50$ mm

$m_{\mathrm{x}} = x - 0.8s_{\mathrm{f}} = 40 - 0.8 \times 12 = 30.4$ mm

For Mode 1, $\ell_{\mathrm{eff,1}}$ is the lesser of $\ell_{\mathrm{eff,nc}}$ and $\ell_{\mathrm{eff,cp}}$

Table 6.6

$\ell_{\mathrm{eff,cp}}$ is the smallest of:

Table 2.2(a) in STEP 1A

$2\pi m_{\mathrm{x}} = 2\pi \times 30.4 = 191$ mm

$\pi m_{\mathrm{x}} + w = (\pi \times 30.4) + 100 = 196$ mm

$\pi m_{\mathrm{x}} + 2e = (\pi \times 30.4) + (2 \times 75) = 246$ mm

As 191 < 196 < 246, $\ell_{\mathrm{eff,cp}} = 191$ mm

$\ell_{\mathrm{eff,nc}}$ is the smallest of:

Table 6.6
Table 2.2(a) in STEP 1A

$4m_{\mathrm{x}} + 1.25e_{\mathrm{x}} = (4 \times 30.4) + (1.25 \times 50)$m $= 184$ mm

$e + 2m_{\mathrm{x}} + 0.625e_{\mathrm{x}} = 75 + (2 \times 30.4) + (0.625 \times 50) = 167$ mm

$0.5b_{\mathrm{p}} = 0.5 \times 250 = 125$ mm

$0.5w + 2m_{\mathrm{x}} + 0.625e_{\mathrm{x}} = (0.5 \times 100) + (2 \times 30.4) + (0.625 \times 50) = 142$ mm

As 125 mm < 142 mm < 167 mm < 184 mm, $\ell_{\mathrm{eff,nc}} = 125$ mm

Title	Example C.1 – Bolted end plate connection (unstiffened)	Sheet 7 of 23

As 125 mm < 191 mm, $\ell_{eff,1}$ = 125 mm

For Mode 2, $\ell_{eff,2} = \ell_{eff,nc}$ = 125 mm

Mode 1 resistance

For Mode 1 failure, using Method 2: — Table 6.2

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

where:

n = e_{min} = e_x = 75 mm but $n \leq 1.25m$

$1.25m$ = 1.25 × 30.4 = 38.0 mm

Therefore, n = 38.0 mm

m = m_x = 30.4 mm — Sheet 6

e_w = 9.9 mm (based on width across the bolt head) — Sheet 5

$$M_{pl,1,Rd} = \frac{0.25 \Sigma \ell_{eff,1} t_p^2 f_y}{\gamma_{M0}}$$

f_y = $f_{y,p}$ = 265 N/mm^2

t_f = t_p = 25 mm

$$M_{pl,1,Rd} = \frac{0.25 \times 125 \times 25^2 \times 265}{1.0} = 5180 \times 10^3 \text{ Nmm}$$

$$F_{T,1,p,Rd} = \frac{(8 \times 38.0 - 2 \times 9.9) \times 5180 \times 10^3}{2 \times 30.4 \times 38.0 - 9.9 \times (30.4 + 38.0)} \times 10^{-3} = 901 \text{ kN}$$

Mode 2 resistance

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

where:

$$M_{pl,2,Rd} = \frac{0.25 \Sigma \ell_{eff,2} t_p^2 f_y}{\gamma_{M0}}$$ — Based on Table 6.2

As $\ell_{eff,2} = \ell_{eff,1}$, $M_{pl,2,Rd} = M_{pl,1,Rd}$ = 5180 × 10^3 Nmm

$\Sigma F_{t,Rd}$ = 406 × 10^3 N — Sheet 5

Therefore, $$F_{T,2,Rd} = \frac{2 \times 5180 \times 10^3 + 38.0 \times 406 \times 10^3}{30.4 + 38.0} \times 10^{-3} = 377 \text{ kN}$$

Mode 3 resistance (bolt failure)

$F_{T,3,Rd} = \Sigma F_{t,Rd}$ = 406 kN

Resistance of end plate in bending

$F_{t,ep,Rd}$ = min{ $F_{T,1,Rd}$; $F_{T,2,Rd}$; $F_{T,3,Rd}$ } = 377 kN — 6.2.4.1(6)

Beam web in tension

As bolt row 1 is in the extension of the end plate, the resistance of the beam web in tension is not applicable to this bolt row.

Summary: Resistance of T-stubs for bolt row 1

Resistance of bolt row 1 is the smallest value of:

Column flange in bending	$F_{t,fc,Rd}$	= 398 kN
Column web in tension	$F_{t,wc,Rd}$	= 790 kN
End plate in bending	$F_{t,ep,Rd}$	= 377 kN

Therefore, the resistance of bolt row 1 is $F_{t1,Rd}$ = 377 kN

BOLT ROW 2

Firstly, consider row 2 alone.

Column flange in bending — STEP 1

The resistance of the column flange in bending is as calculated for bolt row 1 (Mode 2)

$F_{t,fc,Rd}$ = 398 kN — Sheet 5

Column web in transverse tension — STEP 1B

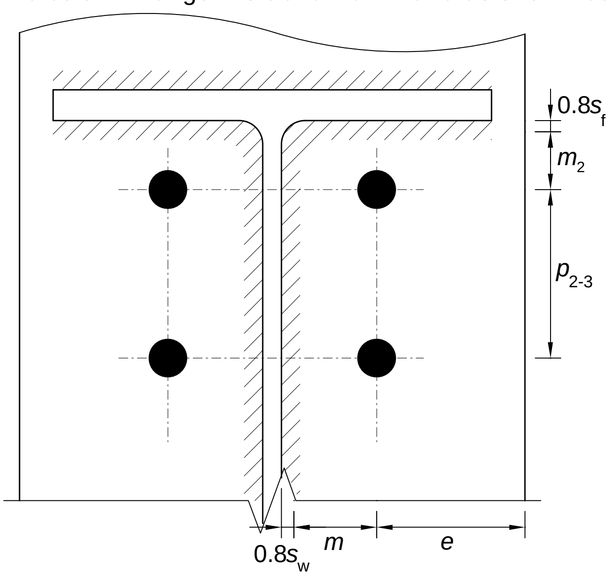
The column web resistance to transverse tension will also be as calculated for bolt row 1

Therefore:

$F_{t,wc,Rd}$ = 790 kN — Sheet 6

End plate in bending — STEP 1

Bolt row 2 is the first bolt row below the beam flange, considered as 'first bolt-row below tension flange of beam' in Table 6.6. The key dimensions for the T-stub are as shown for the column flange T-stub for row 1 and as shown below (in elevation) for row 2. — Sheet 4



Determine m, m_2, α, e and ℓ_{eff} — Table 6.6

$$m = m_p = \frac{w - t_{wb} - 2\times 0.8 s_w}{2} = \frac{100 - 10.1 - 1.6\times 8}{2} = 38.6 \text{ mm}$$

$e = e_p$ = 75 mm — Sheet 6

m_2 = 60 – t_{fb} – $0.8s_f$ = 60 – 15.6 – (0.8 × 12) = 34.8 mm

α is obtained from Figure 6.11 (reproduced in Appendix G as Figure G.1)

Parameters required to determine α are:

$$\lambda_1 = \frac{m}{m+e} \text{ and } \lambda_2 = \frac{m_2}{m+e}$$

Title Example C.1 – Bolted end plate connection (unstiffened) Sheet 9 of 23

$\lambda_1 = \frac{38.6}{38.6 + 75} = 0.34$

$\lambda_2 = \frac{34.8}{38.6 + 75} = 0.31$

Thus, by interpolation (or iterative use of equations in Appendix G), $\alpha = 7.5$

$\ell_{eff,cp} = 2\pi m = 2\pi \times 38.6 = 243$ mm — Table 6.6

$\ell_{eff,nc} = \alpha m = 7.5 \times 38.6 = 290$ mm — Table 2.2(c) in STEP 1A

$\ell_{eff,1}$ = is the lesser of $\ell_{eff,cp}$ and $\ell_{eff,nc}$

$\ell_{eff,1}$ = 243 mm

$\ell_{eff,2} = \ell_{eff,nc} = 290$ mm

Mode 1 resistance

For Mode 1, using Method 2: — Table 6.2

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

where:

$n = e_{min}$ but $n \leq 1.25m$

e_{min} = 75 mm — Sheet 4

$1.25m = 1.25 \times 38.6 = 48.3$ mm

Therefore, n = 48.3 mm

e_w = 9.9 mm (based on width across the bolt head) — Sheet 5

$$M_{pl,1,Rd} = \frac{0.25 \Sigma \ell_{eff,1} t_f^2 f_y}{\gamma_{M0}}$$

$t_f = t_p = 25$ mm

$$M_{pl,1,Rd} = \frac{0.25 \times 243 \times 25^2 \times 265}{1.0} = 10.1 \times 10^6 \text{ Nmm}$$

$$F_{T,1,Rd} = \frac{(8 \times 48.3 - 2 \times 9.9) \times 1.01 \times 10^7}{(2 \times 38.6 \times 48.3) - 9.9(38.6 + 48.3)} \times 10^{-3} = 1291 \text{ kN}$$

Mode 2 resistance

$$F_{T,Rd} = \frac{2M_{pl,2,Rd} + n\Sigma F_{t,Rd}}{m+n}$$ — Table 6.2

$F_{t,Rd}$ = 203 kN — Sheet 5

$$M_{pl,2,Rd} = \frac{0.25 \times 290 \times 25^2 \times 265}{1.0} = 12.0 \times 10^6 \text{ Nmm}$$

$\Sigma F_{t,Rd} = 2 \times 203 = 406$ kN

$$F_{T,2,Rd} = \frac{2 \times 12.0 \times 10^6 + 48.3 \times 406 \times 10^3}{38.6 + 48.3} \times 10^{-3} = 502 \text{ kN}$$

Mode 3 resistance (bolt failure)

$F_{T,3,Rd} = \Sigma F_{t,Rd} = \underline{406 \text{ kN}}$

<u>*Resistance of end plate in bending*</u>

$F_{t,ep,Rd} = \min\{ F_{T,1,Rd} \,;\, F_{T,2,Rd} \,;\, F_{T,3,Rd} \} = 406$ kN — 6.2.4.1(6)

Beam web in tension — STEP 1B

The design tension resistance of the web is determined from: — 6.2.6.8(1), Eq. (6.22)

$$F_{t,wb,Rd} = \frac{\omega\, b_{eff,t,wc}\, t_{wc}\, f_{y,b}}{\gamma_{M0}}$$

where:

$b_{eff,t,wb} = \ell_{eff}$ — 6.2.6.8(2)

Conservatively, consider the smallest ℓ_{eff} from earlier calculations. Therefore:

$b_{eff,t,wb} = \ell_{eff,cp} = 243$ mm — Sheet 9

$t_{wb} = 10.1$ mm

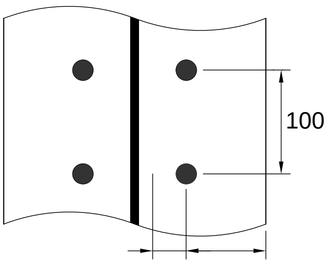
Therefore:

$$F_{t,wb,Rd} = \frac{1 \times 243 \times 10.1 \times 275}{1.0} \times 10^{-3} = 675 \text{ kN}$$

The above resistances for row 2 all consider the resistance of the row acting alone. However, on the column side, the resistance may be limited by the resistance of the group of rows 1 and 2. That group resistance is now considered.

ROWS 1 AND 2 COMBINED

Column flange in bending



For bolt row 1 combined with row 2 in the column flange, both rows are considered as ‘end bolt rows’ in Table 6.4.

For bolt row 1:

$\ell_{eff,nc}$ is the smaller of: — Table 2.3(a) in STEP 1A

$2m + 0.625e + 0.5p$

$e_1 + 0.5\,p$

Here, e_1 is large so it will not be critical.

$p = p_{1\text{-}2} = 100$ mm

$\ell_{eff,nc} = (2 \times 33.4) + (0.625 \times 79.4) + (0.5 \times 100) = 166$ mm

$\ell_{eff,cp}$ is the smaller of: — Table 2.3(a) in STEP 1A

$\pi m + p$

$2e_1 + p$

As above, e_1 is large so will not be critical.

$\ell_{eff,cp} = (\pi \times 33.4) + 100 = 205$ mm

The effective lengths for bolt row 2, as a bottom row of a group, are the same as for row 1

$\Sigma\, \ell_{eff,nc} = 2 \times 166 = 332$ mm

$\Sigma\, \ell_{eff,cp} = 2 \times 205 = 410$ mm

Title *Example C.1 – Bolted end plate connection (unstiffened)* *Sheet 11 of 23*

The effective lengths for the group of bolts is:

Mode 1: The smaller of $\Sigma\ell_{eff,nc}$ and $\Sigma\ell_{eff,cp}$ — Table 6.4

As 332 mm < 410 mm, $\Sigma\ell_{eff,1}$ = 332 mm

Mode 2: $\Sigma\ell_{eff,2} = \Sigma\ell_{eff,nc}$ = 332 mm — Table 6.4

Mode 1 resistance

$$F_{T,1,Rd} = \frac{(8n - 2e_w) M_{pl,1,Rd}}{2mn - e_w (m + n)}$$ — Table 6.2

where:

m = 33.4 mm — Sheet 4

n = 41.8 mm — Sheet 4

e_w = 9.9 mm — Sheet 5

$$M_{pl,1,Rd} = \frac{0.25 \sum \ell_{eff,1} t_f^2 f_y}{\gamma_{MO}}$$

$$= \frac{0.25 \times 332 \times 20.5^2 \times 265}{1.0} = 9.24 \times 10^6 \text{ Nmm}$$

$$F_{T,1,Rd} = \frac{((8 \times 41.8) - (2 \times 9.9)) \times 9.24 \times 10^6}{(2 \times 33.4 \times 41.8) - 9.9 \times (33.4 + 41.8)} \times 10^{-3} = 1420 \text{ kN}$$

Mode 2 resistance

$$F_{T,2,Rd} = \frac{2 M_{pl,2,Rd} + n \sum F_{t,Rd}}{m + n}$$ — Table 6.2

where:

$F_{t,Rd}$ = 203 kN — Sheet 5

$\Sigma F_{t,Rd} = 4 \times 203 = 812$ kN

$$M_{pl,2,Rd} = \frac{0.25 \sum \ell_{eff,2} t_f^2 f_y}{\gamma_{M0}}$$

Here, as $\Sigma \ell_{eff,2} = \Sigma \ell_{eff,1}$

$M_{pl,2,Rd} = M_{pl,1,Rd} = 9.24 \times 10^6$ Nmm

$$F_{T,2,Rd} = \frac{2 \times 9.24 \times 10^6 + 41.8 \times 812 \times 10^3}{33.4 + 41.8} \times 10^{-3} = 697 \text{ kN}$$

Mode 3 resistance (bolt failure)

$F_{T,3,Rd} = \Sigma F_{t,Rd} = 4 \times 203 = 812$ kN — Table 6.2

Resistance of column flange in bending

$F_{t,fc,Rd} = \min\{ F_{T,1,Rd} ; F_{T,2,Rd} ; F_{T,3,Rd} \} = 697$ kN — 6.2.4.1(6)

Column web in transverse tension

STEP 1B

The design resistance of an unstiffened column web in transverse tension is:

$$F_{t,wc,Rd} = \frac{\omega \, b_{eff,t,wc} \, t_{wc} \, f_{y,wc}}{\gamma_{M0}}$$ — 6.2.6.3(1) Eq (6.15)

where:

$b_{eff,t,wc}$ is the effective length of the equivalent T-stub representing the column flange from 6.2.6.4 — 6.2.6.3(3)

Conservatively use the lesser of the values of effective lengths for Mode 1 and Mode 2 — Sheet 10

$b_{eff,t,wc} = \Sigma\ell_{eff,2} = 332$ mm

The equation to use to calculate ω depends on β — Sheet 6

As before, $\beta = 0$ and therefore $\omega = 1.0$

$$F_{t,wc,Rd} = \frac{1.0 \times 332 \times 12.8 \times 265}{1.0} \times 10^{-3} = 1126 \text{ kN}$$

End plate in bending

There is no group mode for the end plate

Summary: resistance of bolt rows 1 and 2 combined

Resistance of bolt rows 1 and 2 combined, on the column side, is the smaller value of:

Column flange in bending	$F_{t,fc,Rd}$	= 697 kN
Column web in tension	$F_{T,wc,Rd}$	= 1126 kN

Therefore, the resistance of bolt rows 1 and 2 combined is $F_{t,1\text{-}2,Rd}$ = 697 kN

The resistance of bolt row 2 on the column side is therefore limited to:

$F_{t2,c,Rd} = F_{t,1\text{-}2,Rd} - F_{t1,Rd} = 697 - 377 = 320$ kN

Summary: resistance of bolt row 2

Resistance of bolt row 2 is the smallest value of:

Column flange in bending	$F_{t,fc,Rd}$	= 398 kN
Column web in tension	$F_{t,wc,Rd}$	= 790 kN
Beam web in tension	$F_{t,wb,Rd}$	= 675 kN
End plate in bending	$F_{t,ep,Rd}$	= 406 kN
Column side, as part of a group	$F_{t2,c,Rd}$	= 320 kN

Therefore, the resistance of bolt row 2 is $F_{t,2,Rd}$ = 320 kN

BOLT ROW 3

Firstly, consider row 3 alone

Column flange in bending

STEP 1

The column flange in bending resistance is the same as bolt rows 1 and 2 therefore:

$F_{t,fc,Rd} = 398$ kN — Sheet 5

Column web in transverse tension

STEP 1B

The column web resistance to transverse tension is as calculated for bolt rows 1 and 2. Therefore:

$F_{t,wc,Rd} = 790$ kN — Sheet 6

End plate in bending

STEP 1

Bolt row 3 is the second bolt row below the beam's tension flange, considered as an 'other end bolt-row' in Table 6.6. The key dimensions are as noted above for bolt row 2.

Determine m, e and ℓ_{eff}

$e = e_p = 75$ mm

$m = 38.6$ mm — Sheet 8

$\ell_{eff,cp} = 2\pi m = 2\pi \times 38.6 = 243$ mm — Table 2.2(c)

Title	Example C.1 – Bolted end plate connection (unstiffened)	Sheet 13 of 23

$\ell_{eff,nc}$ = $4m + 1.25e$ = $(4 \times 38.6) + (1.25 \times 75) = 248$ mm — in STEP 1A

$\ell_{eff,1}$ = min $\{\ell_{eff,cp} ; \ell_{eff,nc}\}$

= min {243 ; 248} = 243 mm

$\ell_{eff,2}$ = $\ell_{eff,nc}$ = 248 mm

Mode 1 resistance

For Mode 1, using Method 2:

$$F_{T,1,Rd} = \frac{(8n - 2e_w) M_{p\ell,1,Rd}}{2\,mn - e_w\,(m+n)} \times 10^{-3}$$ — Table 6.2

where:

n = 48.3 mm and m = 38.6 mm (as for row 2) — Sheets 9 & 8

e_w = 9.9 mm (based on width across the bolt head) — Sheet 5

$$M_{pl,1,Rd} = \frac{0.25\, \Sigma\, \ell_{eff,1}\, t_f^2\, f_y}{\gamma_{MO}}$$

t_f = t_p = 25 mm

$$M_{pl,1,Rd} = \frac{0.25 \times 243 \times 25^2 \times 265}{1.0} = 10.1 \times 10^6 \text{ Nmm}$$

$$F_{T,1,Rd} = \frac{(8 \times 48.32 \times 9.9) \times 1.0 \times 10^7}{(2 \times 38.6 \times 48.3) - 9.9 \times (38.6 + 48.3)} \times 10^{-3} = 1291 \text{ kN}$$

Mode 2 resistance

$$F_{T,2,Rd} = \frac{2\, M_{pl,2,Rd} + n\, \Sigma\, F_{t,Rd}}{m+n}$$ — Table 6.2

$$M_{pl,2,Rd} = \frac{0.25 \sum \ell_{eff,2}\, t_f^2 f_y}{\gamma_{M0}}$$

t_f = t_p = 25 mm

$$M_{pl,2,Rd} = \frac{0.25 \times 248 \times 25^2 \times 265}{1.0} = 10.3 \times 10^6 \text{ Nmm}$$

$\Sigma\, F_{t,Rd}$ = 2 × 203 = 406 kN

$$F_{T,2,Rd} = \frac{(2 \times 1.03 \times 10^7) + (48.3 \times 406 \times 10^3)}{38.6 + 48.3} \times 10^{-3} = 463 \text{ kN}$$

Mode 3 resistance (bolt failure)

$F_{T,3,Rd}$ = $\Sigma\, F_{t,Rd}$ = 406 kN

Resistance of end plate in bending

$F_{t,ep,Rd}$ = min{ $F_{T,1,Rd}$; $F_{T,2,Rd}$; $F_{T,3,Rd}$ } = 406 kN — 6.2.4.1(6)

Beam web in tension — STEP 1B

The design tension resistance of the beam web is determined from:

$$F_{t,wb,Rd} = \frac{b_{eff,t,wb}\; t_{wb}\; f_{y,b}}{\gamma_{M0}}$$ — 6.2.6.8(1) Eq (6.22)

where:

$b_{eff,t,wb}$ = ℓ_{eff}

Conservatively, consider minimum ℓ_{eff}

Therefore,

$b_{eff,t,wb} = \ell_{eff,1}$ = 243 mm — Sheet 13

t_{wb} = 10.1 mm

Therefore, $F_{t,wb,Rd} = \frac{243 \times 10.1 \times 275}{1.0} \times 10^{-3} = 675$ kN

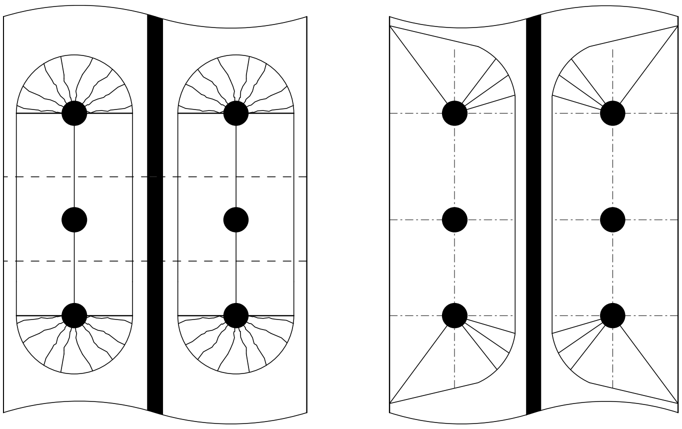
The above resistances for row 3 all consider the resistance of the row acting alone. However, on the column side the resistance may be limited by the resistance of the group of rows 1, 2, and 3 or by the group of rows 2 and 3. On the beam side, the resistance may be limited by the group of rows 2 and 3. Those group resistances are now considered.

ROWS 1, 2 AND 3 COMBINED

Column flange in bending

STEP 1

Circular and non-circular yield line patterns are:



The effective length for bolt row 1, as part of a group, is the same as that determined as part of the group of rows 1 and 2. Thus:

Row 1 $\ell_{eff,nc}$ = 166 mm

$\ell_{eff,cp}$ = 205 mm — Sheet 10

Row 3 is also an 'end bolt row', similar to row 1, but the value of bolt spacing p is different.

$p = p_{2\text{-}3}$ = 90 mm

Thus

$\ell_{eff,nc} = 2m + 0.625e + 0.5p = (2 \times 33.4) + (0.625 \times 79.4) + (0.5 \times 90) = 161$ mm — Table 2.3(c) in STEP 1A

$\ell_{eff,cp} = \pi m + p = (\pi \times 33.4) + 90 = 195$ mm

For this group, bolt row 2 is an 'other inner bolt row' in Table 6.6.

Therefore:

$\ell_{eff,cp} = 2p$ — Table 2.3(b) in STEP 1A

$\ell_{eff,nc} = p$

Here, the vertical spacing between bolts above and below row 2 is different, therefore use:

$$p = \frac{p_{1-2}}{2} + \frac{p_{2-3}}{2} = \frac{100}{2} + \frac{90}{2} = 95 \text{ mm}$$

$\ell_{eff,cp}$ = 2 × 95 = 190 mm

$\ell_{eff,nc}$ = 95 mm

Therefore, the total effective lengths for this group of rows are:

$\Sigma\, \ell_{eff,nc}$ = 166 + 95 + 161 = 422 mm

Title	Example C.1 – Bolted end plate connection (unstiffened)	Sheet 15 of 23

$\Sigma \ell_{eff,cp}$ = 205 + 190 + 195 = 590 mm	
Therefore, $\Sigma \ell_{eff,2} = \Sigma \ell_{eff,1}$ = 422 mm	Table 6.4
Mode 1 resistance	
For Mode 1, using Method 2:	
$F_{T,1,Rd} = \dfrac{(8n - 2e_w)M_{p\ell,1,Rd}}{2mn + e_w(m+n)}$	Table 6.2
where:	
m = 33.4 mm	Sheet 4
n = 41.8 mm	Sheet 4
e_w = 9.9 mm	Sheet 5
$M_{pl,1,Rd} = \dfrac{0.25 \sum \ell_{eff,1} t_f^2 f_{y,c}}{\gamma_{M0}}$	
$= \dfrac{0.25 \times 422 \times 20.5^2 \times 265}{1.0} = 11.7 \times 10^6$ Nmm	
$F_{T,1,Rd} = \dfrac{(8 \times 41.8 - 2 \times 9.9) \times 11.7 \times 10^6}{(2 \times 33.4 \times 41.8) - 9.9 \times (33.4 + 41.8)} \times 10^{-3} = 1797$ kN	
Mode 2 resistance	
$F_{T,2,Rd} = \dfrac{2 M_{p\ell,2,Rd} + n \Sigma F_{t,Rd}}{m+n}$	Table 6.2
where:	
$F_{t,Rd}$ = 203 kN	Sheet 5
$\Sigma F_{t,Rd} = 6 \times 203 = 1218$ kN	
$M_{pl,2,Rd} = \dfrac{0.25 \sum \ell_{eff,2,Rd} t_f^2 f_y}{\gamma_{M0}}$	
Here, as $\ell_{eff,2} = \ell_{eff,1}$	
$M_{pl,2,Rd} = M_{pl,1,Rd} = 11.7 \times 10^6$ Nmm	
$F_{T,2,Rd} = \dfrac{2 \times 11.7 \times 10^6 + 41.8 \times 1218 \times 10^3}{33.4 + 41.8} \times 10^{-3} = 988$ kN	
Mode 3 resistance (bolt failure)	
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 6 \times 203 = 1218$ kN	Table 6.2
Resistance of column flange in bending	
$F_{t,fc,Rd} = \min\{F_{T,1,Rd}; F_{T,2,Rd}; F_{T,3,Rd}\} = 988$ kN	6.2.4.1(6)
Column web in transverse tension	STEP 1B
The design resistance of an unstiffened column web in transverse tension is:	
$F_{t,wc,Rd} = \dfrac{\omega\, b_{eff,t,wc} t_{wc} f_{y,c}}{\gamma_{M0}}$	6.2.6.3(1) Eq (6.15)
where:	
$b_{eff,t,wc}$ is the effective length of the equivalent T-stub representing the column flange from 6.2.6.4. As the failure mode is Mode 2 (sheet 15) take	6.2.6.3(3)
$b_{eff,t,wc} = \Sigma \ell_{eff,2}$ = 422 mm	Sheet 15

The equation to use to calculate ω depends on β

As before, with equal and opposite moments from the beams, $\beta = 0$ and therefore $\omega = 1$

$$F_{\mathrm{t,wc,Rd}} = \frac{1 \times 422 \times 12.8 \times 265}{1.0} \times 10^{-3} = 1431 \text{ kN}$$

Summary: resistance of bolt rows 1, 2 and 3 combined

Resistance of bolt rows 1, 2 and 3 combined, on the column side, is the smaller value of:

Column flange in bending $F_{\mathrm{t,fc,Rd}}$ = 988 kN

Column web in tension $F_{\mathrm{t,wc,Rd}}$ = 1431 kN

Therefore, the resistance of bolt rows 1, 2 and 3 combined is $F_{\mathrm{t1\text{-}3,Rd}}$ = 988 kN

The resistance of bolt row 3 on the column side is therefore limited to:

$F_{\mathrm{t3,c,Rd}} = F_{\mathrm{t1\text{-}3,Rd}} - F_{\mathrm{t1\text{-}2,Rd}} = 988 - 697 = 291$ kN

ROWS 2 AND 3 COMBINED

Column side – flange in bending

Following the same process as for rows 1, 2 and 3 combined,

$\Sigma \ell_{\mathrm{eff,cp}} = 2\pi m + 2p$

$= 2 \times \pi \times 33.4 + 2 \times 90 = 390$ mm

$\Sigma \ell_{\mathrm{eff,nc}} = 4m + 1.25e + p$

$= 4 \times 33.4 + 1.25 \times 79.4 + 90 = 323$ mm

Therefore, $\Sigma \ell_{\mathrm{eff,2}} = \Sigma \ell_{\mathrm{eff,1}} = 323$ mm

<u>*Mode 1 resistance*</u>

Table 6.2

$$M_{\mathrm{pl,1,Rd}} = \frac{0.25 \sum \ell_{\mathrm{eff,1}} t_{\mathrm{f}}^2 f_{\mathrm{y,c}}}{\gamma_{\mathrm{M0}}} = \frac{0.25 \times 323 \times 20.5^2 \times 265}{1.0} = 9.0 \times 10^6 \text{ Nmm}$$

$$F_{\mathrm{T,1,Rd}} = \frac{(8 \times 41.8 - 2 \times 9.9) \times 9.0 \times 10^6}{(2 \times 33.4 \times 41.8) - 9.9 \times (33.4 + 41.8)} \times 10^{-3} = 1383 \text{ kN}$$

<u>*Mode 2 resistance*</u>

Table 6.2

$$F_{\mathrm{T,2,Rd}} = \frac{2 M_{\mathrm{pl,2,Rd}} + n \sum F_{\mathrm{t,Rd}}}{m + n} = \frac{2 \times 9.0 \times 10^6 + 41.8 \times 4 \times 203 \times 10^3}{33.4 + 41.8} \times 10^{-3} = 691 \text{ kN}$$

<u>*Mode 3 resistance (bolt failure)*</u>

Table 6.2

$F_{\mathrm{T,3,Rd}} = \Sigma F_{\mathrm{t,Rd}} = 6 \times 203 = 1218$ kN

Column web in transverse tension

$b_{\mathrm{eff,t,wc}} = 323$ mm 6.2.6.3(1)

As $\beta = 0$ and $\omega = 1$, then Eq. (6.15)

$$F_{\mathrm{t,wc,Rd}} = \frac{1 \times 323 \times 12.8 \times 265}{1.0} \times 10^{-3} = 1096 \text{ kN}$$

Title Example C.1 – Bolted end plate connection (unstiffened) Sheet 17 of 23

Beam side – end plate in bending

On the beam side, row 1 is not part of a group but the resistance of row 3 may be limited by the resistance of rows 2 and 3 as a group.

Determine the effective lengths for rows 2 and 3 combined:

Row 2 is a 'first bolt-row below tension flange of beam' in Table 6.6

$\ell_{eff,cp} = \pi m + p$

Here $p = p_{2\text{-}3}$ = 90 mm, n = 48.3 mm and m = 38.6 mm (as for row 2 alone)

$\ell_{eff,cp} = (\pi \times 38.6) + 90 = 211$ mm — Table 2.3(a) in STEP 1A

$\ell_{eff,nc} = 0.5\,p + \alpha m - (2m + 0.625e)$

Obtain α from Figure 6.11 (or Appendix G) using:

$$\lambda_1 = \frac{m}{m+e} \text{ and } \lambda_2 = \frac{m_2}{m+e}$$

$$\lambda_1 = \frac{38.6}{38.6+75} = 0.34$$

$$\lambda_2 = \frac{34.8}{38.6+75} = 0.31$$

From Figure 6.11 α = 7.5

$\ell_{eff,nc} = (0.5 \times 90) + (7.5 \times 38.6) - (2 \times 38.6 + (0.625 \times 75)) = 210$ mm

Row 3 is an 'other end bolt-row' in Table 6.6

$\ell_{eff,cp} = \pi m + p$

$= (\pi \times 38.6) + 90 = 211$ mm — Table 2.3(c) in STEP 1A

$\ell_{eff,nc} = 2m + 0.625e + 0.5p$

$= (2 \times 38.6) + (0.625 \times 75) + (0.5 \times 90) = 169$ mm

Therefore, the total effective lengths for this group of rows are:

$\Sigma\,\ell_{eff,nc} = 210 + 169 = 379$ mm

$\Sigma\,\ell_{eff,cp} = 211 + 211 = 422$ mm

As 379 mm < 422 mm, $\Sigma\,\ell_{eff,1} = \Sigma\,\ell_{eff,2} = 379$ mm

<u>*Mode 1 resistance (rows 2 + 3)*</u>

For Mode 1 failure, using Method 2: — Table 6.2

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

where:

n = 48.3 mm — Sheet 9

e_w = 9.9 mm — Sheet 5

$$M_{pl,1,Rd} = \frac{0.25\,\Sigma\ell_{eff,1}\,t_f^2\,f_y}{\gamma_{M0}}$$

$$= \frac{0.25 \times 379 \times 25^2 \times 265}{1.0} = 15.7 \times 10^6 \text{ Nmm}$$

m = 38.6 mm — Sheet 8

$$F_{T,1,Rd} = \frac{(8 \times 48.3 - 2 \times 9.9) \times 15.7 \times 10^6}{2 \times 38.6 \times 48.3 - 9.9 \times (38.6 + 48.3)} \times 10^{-3} = 2007 \text{ kN}$$

Mode 2 resistance (rows 2 + 3)

$$F_{T,2,Rd} = \frac{2\,M_{pl,2,Rd} + n\sum F_{t,Rd}}{m+n}$$ — Table 6.2

where:

$F_{t,Rd}$ = 203 kN — Sheet 5

$\Sigma F_{t,Rd}$ = 4 × 203 = 812 kN

$$M_{pl,2,Rd} = \frac{0.25\Sigma\ell_{eff,2,Rd}t_f^2 f_y}{\gamma_{M0}}$$

Here, as $\ell_{eff,2} = \ell_{eff,1}$

$M_{pl,2,Rd} = M_{pl,1,Rd} = 15.7 \times 10^6$ Nmm

$$F_{T,2,Rd} = \frac{2\times 15.7\times 10^6 + 48.3\times 812\times 10^3}{38.6+48.3}\times 10^{-3} = 813\text{ kN}$$

Mode 3 resistance (bolt failure) (rows 2 + 3)

$F_{T,3,Rd} = \Sigma F_{t,Rd}$ = 4 × 203 = 812 kN

Resistance of end plate in bending

$F_{t,ep,Rd} = \min\{F_{T,1,Rd}\ ;\ F_{T,2,Rd}\ ;\ F_{T,3,Rd}\}_{\text{Rows 2-3}}$ = 812 kN — 6.2.4.1(6)

Beam web in tension

This verification is not applicable as the beam flange (stiffener) is within the tension length

Summary: resistance of bolt rows 2 and 3 combined

Resistance of bolt rows 2 and 3 combined, on the beam side, is:

End plate in bending	$F_{t,ep,Rd}$	= 812 kN
Therefore, on the beam side	$F_{t2-3,Rd}$	= 812 kN

The resistance of bolt row 3 on the beam side is therefore limited to:

$F_{t3,b,Rd} = F_{t2-3,Rd} - F_{t2,Rd}$ = 812 − 320 = 492 kN — Sheet 12

Resistance of bolt rows 2 and 3 combined, on the column side, is:

Column flange in bending	$F_{t,fc,Rd}$	= 691 kN
Column web in tension	$F_{t,wc,Rd}$	= 1096 kN
Therefore, on the column side	$F_{t2-3,Rd}$	= 691 kN

The resistance of bolt row 3 on the column side is therefore limited to:

$F_{t3,b,Rd} = F_{t2-3,Rd} - F_{t2,Rd}$ = 691 − 320 = 371 kN — Sheet 12

Summary: resistance of bolt row 3

Resistance of bolt row 3 is the smallest value of:

Column flange in bending	$F_{t,fc,Rd}$	= 398 kN
Column web in tension	$F_{t,wc,Rd}$	= 790 kN
Beam web in tension	$F_{t,wb,Rd}$	= 675 kN
End plate in bending	$F_{t,ep,Rd}$	= 406 kN
Column side, as part of a group with 2 & 1	$F_{t3,c,Rd}$	= 291 kN
Column side, as part of a group with 2	$F_{t3,c,Rd}$	= 371 kN

Beam side, as part of a group with 2 $F_{t3,b,Rd}$ = 492 kN

Therefore, the resistance of bolt row 3 is $F_{t3,Rd}$ = 291 kN

SUMMARY OF TENSION RESISTANCES

The above derivation of effective resistances of the tension rows may be summarized in tabular form, as shown below.

Resistances of rows $F_{tr,Rd}$ (kN)

	Column flange	Column web	End plate	Beam web	Minimum	Effective resistance
Row 1, alone	398	790	377	N/A	377	377
Row 2,alone	398	790	406	675	398	
Row 2, with row 1	697	1126	N/A	N/A	697	
Row 2					697 − 377	320
Row 3, alone	398	790	406	675	309	
Row 3, with row 1 & 2	988	1431	N/A	N/A	988	
Row 3					988-697	291
Row 3, with row 2	691	1096	812	1052	691	
Row 3					691 − 320	

COMPRESSION ZONE

Column web in transverse compression

STEP 2

The design resistance of an unstiffened column web in transverse compression is determined from:

6.2.6.2(1) Eq. (6.9)

$$F_{c,wc,Rd} = \frac{\omega k_{wc} b_{eff,c,wc} t_{wc} f_{y,wc}}{\gamma_{M0}} \quad \text{(crushing resistance)}$$

but:

$$F_{c,wc,Rd} \leq \frac{\omega k_{wc} \rho\, b_{eff,c,wc} t_{wc} f_{y,wc}}{\gamma_{M1}} \quad \text{(buckling resistance)}$$

For a bolted end plate:

$b_{eff,c,wc} = t_{fb} + 2s_f + 5\ (t_{fc} + s) + s_p$

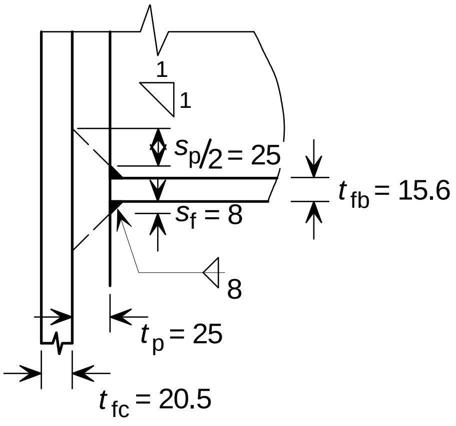
For rolled I and H column sections $s = r_c$

Thus:

$s = r_c$ = 12.7 mm

Sheet 2

s_p is the length obtained by dispersion at 45° through the end plate



$s_p = 2t_p = 2 \times 25 = 50$ mm

Verify that the depth of the end plate (h_p) is sufficient to allow the dispersion of the force. Minimum h_p required is:

$h_p \geq e_x + x + h_b + s_f + t_p$

$= 50 + 40 + 533.1 + 8 + 25 = 656$ mm

$h_p = 670$ mm — Sheet 2

As 670 mm > 656 mm, the depth of the end plate is sufficient.

Therefore:

$b_{eff,c,wc} = 15.6 + (2 \times 8) + 5\,(20.5 + 12.7) + 50$

$= 248$ mm

ρ is the reduction factor for plate buckling

If $\overline{\lambda}_p \leq 0.72$ $\rho = 1.0$ — Eq. (6.13a)

If $\overline{\lambda}_p > 0.72$ $\rho = \dfrac{\overline{\lambda}_p - 0.2}{\lambda_p^{\,2}}$ — Eq. (6.13b)

$\overline{\lambda}_p$ is the plate slenderness

$$\overline{\lambda}_p = 0.932\sqrt{\frac{b_{eff,c,wc}\, d_c\, f_{y.wc}}{E\, t_{wc}^2}}$$ — Eq. (6.13c)

$$= 0.932 \times \sqrt{\frac{248 \times 200.3 \times 265}{210 \times 10^3 \times 12.8^2}} = 0.59$$

As 0.59 < 0.72

$\rho = 1.0$

ω is determined from Table 6.3 based on β

As before, $\beta = 0$ therefore: $\omega = 1.0$

k_{wc} is a reduction factor that takes account of compression in the column web.

Here, it is assumed that $k_{wc} = 1.0$ — Note to 6.2.6.2(2)

$$\frac{\omega k_w b_{eff,c,wc} t_{wc} f_{y,wc}}{\gamma_{M0}} = \frac{1 \times 248 \times 12.8 \times 265}{10} \times 10^{-3} = 841 \text{ kN}$$

As the UK National Annex to BS EN 1993-1-1 gives $\gamma_{M1} = 1.0$ and $\gamma_{M0} = 1.0$

and in this example, $\omega = 1.0$, $\rho = 1.0$ and $k_w = 1.0$

$$\frac{\omega k_w\, \rho b_{eff,c,wc} t_{wc} f_{y,wc}}{\gamma_{M1}} = \frac{\omega k_w b_{eff,c,wc} t_{wc} f_{y,wc}}{\gamma_{M0}}$$

Therefore:

$F_{c,wc,Rd} = 841$ kN

Beam flange and web in compression

The resultant of the design resistance of a beam flange and adjacent compression zone of the web is determined using: — 6.2.6.7(1)

$$F_{c,fb,Rd} = \frac{M_{c,Rd}}{h - t_{fb}}$$ — Eq. (6.21)

Title	Example C.1 – Bolted end plate connection (unstiffened)	Sheet 21 of 23

where:

$M_{c,Rd}$ is the design resistance of the beam

At this stage, assume that the design shear force in the beam does not reduce $M_{c,Rd}$. Therefore, from P363

$M_{c,Rd}$ = 649 kNm — P363 Sheet 2

h = h_b = 533.1 mm

t_{fb} = 15.6 mm

$$F_{c,fb,Rd} = \frac{649}{(533.1-15.6)\times10^{-3}} = 1254 \text{ kN}$$

Summary: resistance of compression zone

Column web in transverse compression $F_{c,wc,Rd}$ = 841 kN

Beam flange and web in compression $F_{c,fb,Rd}$ = 1254 kN

Resistance of column web panel in shear — STEP 3

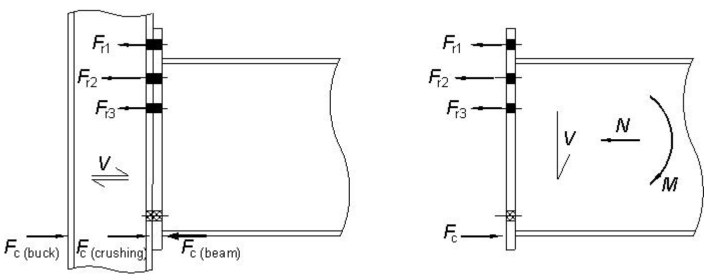
The plastic shear resistance of an unstiffened web is given by: — 6.2.6.1 Eq (6.7)

$$V_{wp,Rd} = \frac{0.9 f_{y,wc} A_{vc}}{\sqrt{3}\,\gamma_{M0}}$$

The resistance is not evaluated here, since there is no design shear in the web because the moments from the beams are equal and opposite.

MOMENT RESISTANCE

EFFECTIVE RESISTANCES OF BOLT ROWS — STEP 4



The effective resistances of each of the three bolt rows in the tension zone are:

$F_{t1,Rd}$ = 377 kN

$F_{t2,Rd}$ = 320 kN — Sheet 19

$F_{t3,Rd}$ = 291 kN

The effective resistances should be reduced if the resistance of one of the higher rows exceeds 1.9 $F_{t,Rd}$. — 6.2.7.2(9)

Here 1.9 $F_{t,Rd}$. = 1.9 × 203 = 386 kN

The resistances of both row 1 and row 2 are less than this value, so no reduction is necessary.

Note that the UK NA states that no reduction is necessary if:

$$t_p \le \frac{d}{1.9}\sqrt{\frac{f_{ub}}{f_{y,p}}} \quad \text{or} \quad t_{fc} \le \frac{d}{1.9}\sqrt{\frac{f_{ub}}{f_{y,fc}}}$$

Title	Example C.1 – Bolted end plate connection (unstiffened)	Sheet 22 of 23

In this case, the limiting thickness in both expressions = $\frac{24}{1.9}\sqrt{\frac{800}{265}} = 21.9$ mm

The column flange is 20.5 mm thick, so no reduction is necessary.

EQUILIBRIUM OF FORCES

The sum of the tensile forces, together with any axial compression in the beam, cannot exceed the resistance of the compression zone.

Similarly, the design shear cannot exceed the shear resistance of the column web panel; this is not relevant in this example as the moments in the identical beams are equal and opposite.

For horizontal equilibrium:

$\Sigma F_{tr,Rd} + N_{Ed} = F_{c,Rd}$

In this example there is no axial compression in the beam (N_{Ed} = 0)

Therefore, for equilibrium of forces in this example:

$\Sigma F_{tr,Rd} = F_{c,Rd}$

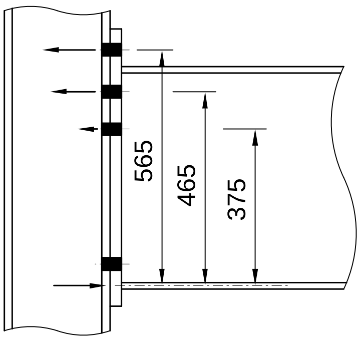
Here, the total effective tension resistance $\Sigma F_{tr,Rd}$ = 377 + 320 + 291 = 988 kN, which exceeds the compression resistance $F_{c,Rd}$ = 841 kN.

To achieve equilibrium, the effective resistances are reduced, starting at the lowest row and working upward, until equilibrium is achieved.

Reduction required = 988 − 841 = 147 kN

All of this reduction can be obtained by reducing the resistance of the bottom row.

Hence $F_{t3,Rd}$ = 291 − 147 = 144 kN



MOMENT RESISTANCE OF JOINT

The moment resistance of the beam to column joint ($M_{j,Rd}$) may be determined using: 6.2.7.2(1)

$M_{j,Rd} = \sum_r h_r F_{tr,Rd}$ (6.25)

Taking the centre of compression to be at the mid-thickness of the compression flange of the beam:

$h_{r1} = h_b - \left(\frac{t_{fb}}{2}\right) + x = 533.1 - \left(\frac{15.6}{2}\right) + 40 = 565$ mm

$h_{r2} = h_{r1} - 100 = 465$ mm

$h_{r3} = h_{r2} - 90 = 375$ mm

Thus, the moment resistance of the beam to column joint is:

$M_{j,Rd} = h_{r1}F_{t1,Rd} + h_{r2}F_{t2,Rd} + h_{r3}F_{t3,Rd}$

$= (565 \times 377 + 465 \times 320 + 375 \times 144) \times 10^{-3} = 416$ kNm

Title Example C.1 – Bolted end plate connection (unstiffened) Sheet 23 of 23

VERTICAL SHEAR RESISTANCE

RESISTANCE OF BOLT GROUP

From P363, the shear resistance of a non-preloaded M24 class 8.8 bolt in single shear is:

$F_{v,Rd}$ = 136 kN P363

$F_{b,Rd}$ = 200 kN (in 20 mm ply)

Hence $F_{v,Rd}$ governs

The shear resistance of the upper rows may be taken conservatively as 28% of the shear resistance without tension (this assumes that these bolts are fully utilized in tension) and thus the shear resistance of all 4 rows is: STEP 5

$(2 + 6 \times 0.28) \times 136 = 3.68 \times 136 = 500$ kN

WELD DESIGN

STEP 7

The simple approach requires that the welds to the tension flange and the web should be full strength and the weld to the compression flange is of nominal size only, assuming that it has been prepared with a sawn cut end.

BEAM TENSION FLANGE WELDS

A full strength weld is provided by symmetrical fillet welds with a total throat thickness at least equal to the flange thickness.

Required throat size = $t_{fb}/2 = 15.6/2 = 7.8$ mm

Weld throat provided $a_f = 12/\sqrt{2} = 8.5$ mm, which is adequate.

BEAM COMPRESSION FLANGE WELDS

Provide a nominal fillet weld either side of the beam flange.

An 8 mm leg length fillet weld will be satisfactory.

BEAM WEB WELDS

For convenience, a full strength weld is provided to the web.

Required throat size = $t_{fw}/2 = 10.2/2 = 5.1$ mm

Weld throat provided $a_p = 8/\sqrt{2} = 5.7$ mm, which is adequate.

SCI Steel Knowledge	Job No. *CDS 324*		Sheet *1* of *4*
CALCULATION SHEET	Title *Example C.2 - Column web compression stiffener*		
BCSA	Client		
	Calcs by *MEB*	Checked by *DGB*	Date *Nov 2012*

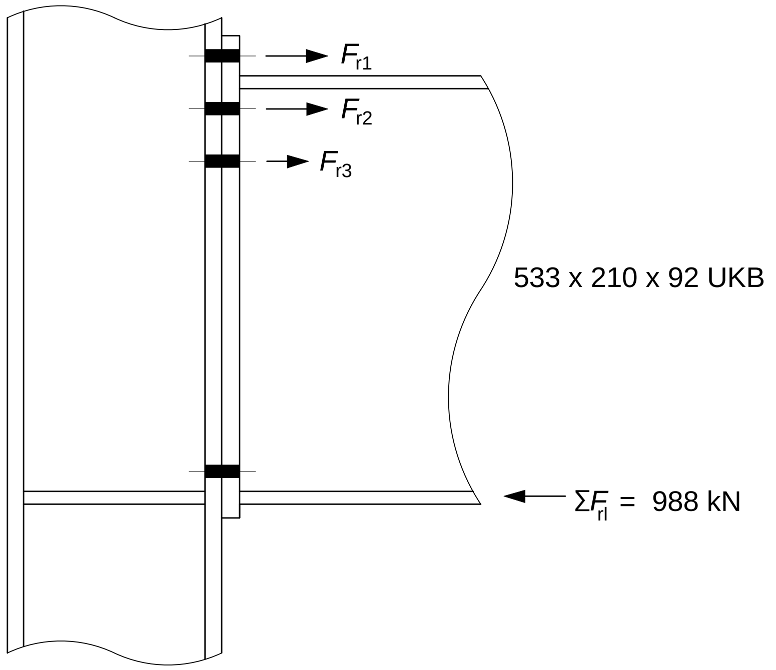
JOINT CONFIGURATION AND DIMENSIONS

This example shows how the column web compression resistance of connection in Example C.1 can be enhanced by adding stiffeners to the web.

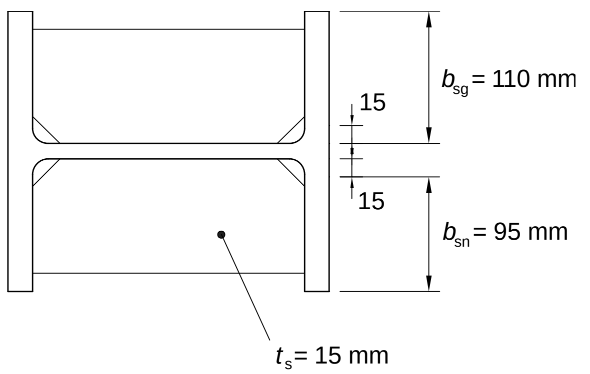
Full depth compression stiffeners will be designed for the column. The stiffened compression zone should have at least sufficient resistance to balance the total potential tension resistances of the upper three bolt rows, as determined in the previous example (i.e. to have a resistance of at least 988 kN).

References to clauses, etc. are to BS EN 1993-1-8: and its UK NA, unless otherwise stated.

254 x 254 x 107 UKC



The chosen stiffeners provide approximately the same overall width and thickness as the compression flange of the beam and are shaped as shown below



Title Example C.2 – Column web compression stiffener | Sheet 2 of 4

DIMENSIONS AND SECTION PROPERTIES

Column

From data tables for 254 × 254 × 107 UKC in S275

SCI P363

Depth	h_c	= 266.7 mm
Width	b_c	= 258.8 mm
Flange thickness	t_{fc}	= 20.5 mm
Web thickness	t_{wc}	= 12.8 mm
Depth between flanges	h_{wc}	= $h_c - 2\,t_{f,c}$
		= 266.7 – (2 × 20.5) = 226 mm

Web compression stiffeners

Depth	h_s	= 226 mm
Gross width	b_{sg}	= 110.0 mm
Net width (in contact with flange)	b_{sn}	= 95.0 mm
Thickness	t_s	= 15 mm

MATERIAL STRENGTHS

BS EN 1993-1-1 NA.2.4

The UK National Annex to BS EN 1993-1-1 refers to BS EN 10025-2 for values of nominal yield and ultimate strength. When ranges are given the lowest value should be adopted.

BS EN 10025-2 Table 7

As for Example 1:

Column yield strength $f_{y,c}$ = 265 N/mm^2

Stiffener yield strength $f_{y,s}$ = 275 N/mm^2 (assuming t_s not greater than 16 mm)

Conservatively, use the same strength for the stiffener as for the column, i.e.

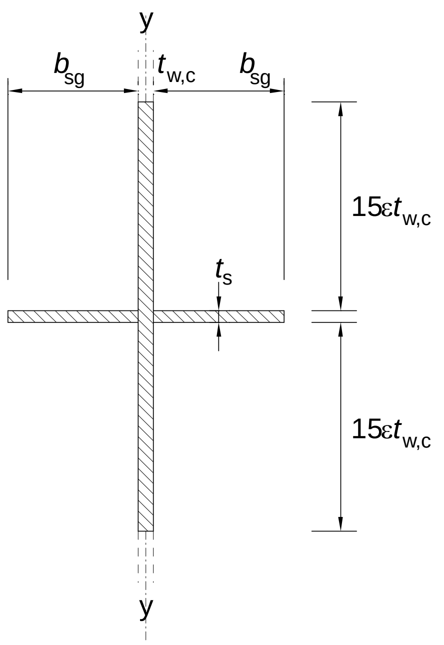
$f_{y,s}$ = 265 N/mm^2

COMPRESSION RESISTANCE OF EFFECTIVE STIFFENER SECTION

STEP 6B

Flexural buckling resistance

Determine the flexural buckling resistance of the cruciform stiffener section shown below



Title Example C.2 – Column web compression stiffener	Sheet 3 of 4

The width of web that may be considered as part of the stiffener section is given by BS EN 1993-1-5 as $15\varepsilon t_{wc}$ either side of the stiffener. — BS EN 1993-1-5, 9.1

The width/thickness ratio of the outstand should be limited to prevent torsional buckling but conservatively the Class 3 limit for compression flange outstands may be used.

Limiting value of c/t for Class 3 = 14ε — BS EN 1993-1-1, Table 5.2

Here,

$$\varepsilon = \sqrt{\frac{235}{265}} = 0.94$$

Hence limiting $c/t = 14 \times 0.94 = 13.2$

Actual ratio = 110/15 = 7.3 OK

Effective area of stiffener for buckling

$$A_{s,eff} = 2\,A_s + t_{wc}\,(30\,\varepsilon\,t_{wc} + t_s)$$

$$= 2 \times 110 \times 15 + 12.8 \times (2 \times 15 \times 0.941 \times 12.8 + 15) = 8110 \text{ mm}^2$$

The second moment of area of the stiffener section may be conservatively determined as:

$$I_s = \frac{\left(2b_{sg} + t_{wc}{}^3\right) t_s}{12}$$

$$= \frac{(2 \times 110 + 12.8)^3 \times 15}{12} = 15.8 \times 10^6 \text{ mm}^4$$

The radius of gyration of the stiffener section is given by:

$$i_s = \sqrt{\frac{I_s}{A_{s,eff}}} = \sqrt{\frac{15.8 \times 10^7}{8110}} = 44.1 \text{ mm}$$

Non-dimensional flexural slenderness:

$$\bar{\lambda} = \frac{\ell}{i_s\,\bar{\lambda}_1}$$ — BS EN 1993-1-1, 6.3.1.2

where

$$\lambda_1 = 93.9\,\varepsilon$$

Assume that the buckling length ℓ is equal to the length of the stiffener

$$\bar{\lambda} = \frac{226}{44.1 \times 93.9 \times 0.94} = 0.06$$

The reduction factor χ is given by buckling curve c according to the value of $\bar{\lambda}$ — BS EN 1993-1-5, 9.4

Since $\bar{\lambda} < 0.2$, the buckling effects may be ignored. Only the resistance of the cross section need be considered. — BS EN 1993-1-1, 6.3.1.2(4)

Resistance of cross section (crushing resistance)

The effective area of the stiffener comprises the area of the additional plates (making a deduction for corner snipes) together with a length of web. The length of web that may be considered depends on dispersal from the beam flange; its value was calculated as 248 mm in Example C.1.

The effective area for crushing is:

$$A_{s,eff} = 2 \times (110 - 15) \times 15 + 248 \times 12.8 = 6020 \text{ mm}^2$$

Thus:

$$N_{c,Rd} = \frac{A_{s,eff}\,f_{ys}}{\gamma_{M0}} = \frac{6020 \times 265}{1.0} \times 10^{-3} = 1595 \text{ kN}$$

With such a stiffener added to the connection in Example C.1, no reduction of bolt row forces in the tension zone would be needed. The moment resistance of the connection would then be:

$M_{j,Rd} = (565 \times 377 + 465 \times 320 + 375 \times 291) \times 10^{-3} = 471$ kNm

WELD DESIGN

Weld to flanges

It is usual for the stiffeners to be fitted for bearing.

Therefore, use 6 mm leg length fillet welds.

If the stiffeners are not fitted, use full strength welds.

Welds to web

In this double-sided connection, no force is transferred to the column web.

If the connection were one-sided, or was subject to unequal compression forces, the web welds would need to be designed to transfer the unbalanced force into the web.

SCI Steel Knowledge CALCULATION SHEET BCSA	Job No. CDS 324		Sheet 1 of 10
	Title Example C.3 – Column tension stiffener		
	Client		
	Calcs by MEB	Checked by DGB	Date Nov 2012

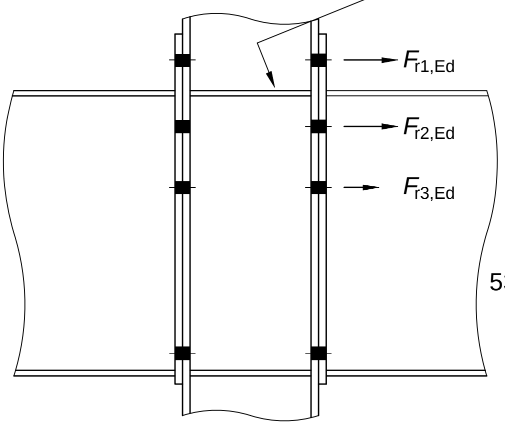
JOINT CONFIGURATION AND DIMENSIONS

The addition of tension stiffeners to the column has the potential to increase the tension resistance of the column web and to increase the tension resistance of bolt rows immediately above and below it.

Consider a beam to column connection similar to that in Example C.1 but with a lighter column section.

Design a tension stiffener to enhance the bending resistance of the column flange.

References to clauses, etc. are to BS EN 1993-1-8: and its UK NA, unless otherwise stated.



DIMENSIONS AND SECTION PROPERTIES

Column

From data tables for 254 × 254 × 73 UKC in S275:

P363

Depth	h_c	= 254.1 mm
Width	b_c	= 254.6 mm
Web thickness	t_{wc}	= 8.6 mm
Flange thickness	t_{fc}	= 14.2 mm
Root radius	r_c	= 12.7 mm
Depth between flange fillets	d_c	= 200.3 mm
Area	A_c	= 93.2 cm^2
Depth between flanges	h_w	$= h_c - 2\,t_{fc}$
		= 254.1 – (2 × 14.2) = 226 mm
Yield strength	$f_{y,c}$	= 275 N/mm^2 (since t_{fc} < 16 mm)

Beam and end plate

Dimensions as in Example C.1

Title	Example C.3 – Column tension stiffener	Sheet	2 of 10

Bolt spacings

Dimensions as in Example C.1, apart from the edge distance on the column side, which is:

Edge distance e_c = 0.5(254.6 − 100) = 77.3 mm

STIFFENER SIZE

Choose an initial size of stiffener using simple guidelines (see STEP 6A).

Gross width of the stiffener $b_{sg} \geq \frac{0.75(b_c - t_{wc})}{2} = \frac{0.75 \times (254.6 - 8.6)}{2} = 92.3$ mm

Take b_{sg} = 100 mm

Length of stiffener required is $h_s \geq 1.9 b_{sg}$ = 190 mm

(In a double-sided connection, a full depth stiffener is required; in a single-sided connection a shorter stiffener could be used.)

Allowing for a 15 × 15 mm corner snipe, the net width of each stiffener is:

b_{sn} = 100 – 15 = 85 mm

Assume a thickness of t_s = 10 mm (b_{sg} / t_s= 10)

For $t \leq 16$ mm and S275 — BS EN 10025-2 Table 7

Yield strength $f_{ys} = R_{eH}$ = 275 N/mm^2

RESISTANCES OF UNSTIFFENED CONNECTION

The resistances of the unstiffened connection have been calculated in the same manner as in Example C.1 (again assuming that the moments on either side of the column are equal and opposite). The resistances are given below.

Resistances of rows $F_{tr,Rd}$ (kN)

	Column flange	Column web	End plate	Beam web	Minimum	Effective resistance
Row 1, alone	309	565	377	N/A	309	309
Row 2,alone	309	565	406	675	309	
Row 2, with row 1	569	799	N/A	N/A	569	
Row 2					569 − 309	260
Row 3, alone	309	565	406	675	309	
Row 3, with row 1 & 2	825	1012	N/A	N/A	825	
Row 3					825 − 569	256
Row 3, with row 2	565	778	812	1052	565	
Row 3					565 − 260	

The sum of the effective resistances of the three bolt rows is 825 kN. The compression resistance of the unstiffened column web is only 473 kN so a compression stiffener would be provided; the moment resistance with an adequate compression stiffener but without a tension stiffener would be:

$$M_{j,Rd} = (565 \times 309 + 465 \times 260 + 375 \times 256) \times 10^{-3} = 392 \text{ kNm}$$

Title Example C.3 – Column tension stiffener | Sheet 3 of 10

STIFFENED COLUMN - TENSION ZONE T-STUBS

The following calculations are similar to those in Example C.1 but for the case where there is a tension stiffener in the column, below the top row of bolts. Only the calculations for the column side are shown; those for the beam side are as in Example C.1.

BOLT ROW 1

Column flange in bending

STEP 1

Bolt row 1 is a 'bolt row adjacent to a stiffener' according to Figure 6.9.

Determine e_{min} , m and ℓ_{eff} — 6.2.4.1(2))

$$m = m_c = \frac{w_{2c} - t_{wc} - 2 \times 0.8\, r_c}{2} = \frac{100 - 8.6 - 2 \times 0.8 \times 12.7}{2} = 35.5 \text{ mm}$$

$e = e_c = 77.3$ mm

$e_{min} = \min(e_c\,;\, e_b) = \min(77.3\,;\,75) = 75$ mm

Assume leg length of stiffener to flange weld = 8 mm.

Distance of bolt row above stiffener (assume top is level with beam flange top)

$m_2 = x - 0.8s_s = 40 - 0.8 \times 8 \times 5.6 = 33.6$ mm

Therefore:

$$\lambda_1 = \frac{m}{m+e} = \frac{35.5}{35.5+77.3} = 0.31$$

$$\lambda_2 = \frac{m_2}{m+e} = \frac{33.6}{35.5+77.3} = 0.30$$

For these values of λ_1 and λ_2, from the chart, $\alpha = 7.7$ — Appendix G

For Mode 1, $\ell_{eff,1} = \ell_{eff,nc}$ but $\ell_{eff,1} \leq \ell_{eff,cp}$ — Table 6.6

$\ell_{eff,cp} = 2\pi m$ — Table 2.2(c) in STEP 1A

$= 2\pi \times 35.5 = 223$ mm

$\ell_{eff,nc} = \alpha m$

$= 7.7 \times 35.5 = 273$ mm

As 223 < 273

$\ell_{eff,1} = \ell_{eff,cp} = 223$ mm

For Mode 2, $\ell_{eff,2} = \ell_{eff,nc}$

Therefore $\ell_{eff,2} = 273$ mm

Mode 1 resistance

$$F_{T,1,Rd} = \frac{(8n - 2e_w)M_{pl,1,Rd}}{2mn - e_w(m+n)}$$ — Table 6.2

where:

$m = m_c = 35.5$ mm

$n = e_{min}$ but $\leq 1.25\, m$

1.25 m = 1.25 × 35.5 = 44.4 mm

As 44.4 < 75:

$n = 44.4$ mm

$$M_{pl,1,Rd} = \frac{0.25\, \Sigma \ell_{eff,1}\, t_f^2 f_y}{\gamma_{M0}}$$

$f_y = f_{y,c} = 275$ N/mm^2

Title Example C.3 – Column tension stiffener	Sheet 4 of 10
$M_{pl,1,Rd} = \dfrac{0.25 \times 223 \times 14.2^2 \times 275}{1.0} = 3090 \times 10^3$ Nmm	
$e_w = \dfrac{d_w}{4}$	
d_w is the diameter of the washer, or the width across points of the bolt head or nut, as relevant	
Here, d_w = 39.55 mm (across the bolt head)	P358
Therefore, $e_w = \dfrac{39.55}{4}$ = 9.9 mm	
Therefore, $F_{T,1,Rd} = \dfrac{(8 \times 44.4 - 2 \times 9.9) \times 3090 \times 10^3}{2 \times 35.5 \times 44.4 - 9.9 \times (35.5 + 44.4)} \times 10^{-3}$ = 439 kN	
Mode 2 resistance	
$F_{T,2,Rd} = \dfrac{2 M_{pl,2,Rd} + n \Sigma F_{t,Rd}}{m + n}$	Table 6.2
where:	
$M_{pl,2,Rd} = \dfrac{0.25 \Sigma \ell_{eff,2} t_f^2 f_y}{\gamma_{M0}}$	
$= \dfrac{0.25 \times 273 \times 14.2^2 \times 275}{1.0} = 3790 \times 10^3$ Nmm	
$\Sigma F_{t,Rd}$ is the total value of $F_{t,Rd}$ for all the bolts in the row.	
For 2 bolts in the row, $\Sigma F_{t,Rd} = 2 \times 203 \times 10^{-3} = 406 \times 10^3$ N	
Therefore, for Mode 2	
$F_{T,2,Rd} = \dfrac{2 \times 3790 \times 10^3 + 44.4 \times 406 \times 10^3}{35.5 + 44.4} \times 10^{-3}$ = 321 kN	
Mode 3 resistance (bolt failure)	
$F_{T,3,Rd} = \Sigma F_{t,Rd}$ = 406 kN	
Resistance of column flange in bending	
$F_{t,fc,Rd} = \min\{ F_{T,1,Rd} ; F_{T,2,Rd} ; F_{T,3,Rd} \}$ = 321 kN	6.2.4.1(6)
Column web in tension	
No check is necessary for a row immediately adjacent to a tension stiffener.	
Because the stiffener is full depth no check is required at the end of the stiffener.	
BOLT ROW 2	
Column flange in bending	
Bolt row 2 is a 'bolt row adjacent to a stiffener' according to Figure 6.9.	
$m = m_c$ = 35.5 mm	
e_{min} = 75 mm and e = 77.3 (as for row 1)	
Distance of bolt row below stiffener (assume top is level with beam flange top)	
$m_2 = p_{1\text{-}2} - x - t_s - 0.8 s_s = 100 - 40 - 10 - 0.8 \times 8 = 43.6$ mm	
Therefore:	
$\lambda_1 = \dfrac{m}{m+e} = \dfrac{35.5}{35.5 + 77.3} = 0.31$	

Title Example C.3 – Column tension stiffener	Sheet 5 of 10
$\lambda_2 = \frac{m_2}{m+e} = \frac{43.6}{33.7+77.3} = 0.39$	
Interpolating in the chart (Figure 6.11 or Appendix G), α = 7.2	Appendix G
For Mode 1, $\ell_{eff,1} = \ell_{eff,nc}$ but $\ell_{eff,1} \le \ell_{eff,cp}$	
$\ell_{eff,cp} = 2\pi m$	
$= 2\pi \times 35.5 = 223$ mm	Table 2.2(c) in STEP 1A
$\ell_{eff,nc} = \alpha m$	
$= 7.2 \times 35.5 = 256$ mm	
As 223 < 256	
$\ell_{eff,1} = \ell_{eff,cp} = 223$ mm	
For Mode 2, $\ell_{eff,2} = \ell_{eff,nc}$	
Therefore $\ell_{eff,2} = 256$ mm	
Mode 1 resistance	
n = 44.4 mm (as for row 1)	
$M_{pl,1,Rd} = \frac{0.25 \Sigma \ell_{eff,1} t_f^2 f_y}{\gamma_{M0}}$	
$f_y = f_{y,c} = 275$ N/mm^2	
$M_{pl,1,Rd} = \frac{0.25 \times 223 \times 14.2^2 \times 275}{1.0} = 3090 \times 10^3$ Nmm	
As before, e_w = 9.9 mm	
Therefore:	
$F_{T,1,Rd} = \frac{(8n - 2e_w)M_{pl,1,Rd}}{2mn - e_w(m+n)}$	
$= \frac{(8 \times 44.4 - 2 \times 9.9) \times 3090 \times 10^3}{2 \times 35.5 \times 44.4 - 9.9 \times (35.5 + 44.4)} \times 10^{-3} = 439$ kN	
Mode 2 resistance	
$M_{pl,2,Rd} = \frac{0.25 \Sigma \ell_{eff,2} t_f^2 f_y}{\gamma_{M0}}$	
$= \frac{0.25 \times 256 \times 14.2^2 \times 275}{1.0} = 3550 \times 10^3$ Nmm	
Therefore, for Mode 2	
$F_{T,2,Rd} = \frac{2M_{pl,2,Rd} + n\Sigma F_{t,Rd}}{m+n} = \frac{2 \times 3550 \times 10^3 + 44.4 \times 406 \times 10^3}{35.5 + 44.4} \times 10^{-3} = 314$ kN	
Mode 3 resistance (bolt failure)	
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 406$ kN	
Resistance of column flange in bending	
$F_{t,fc,Rd} = \min\{F_{T,1,Rd}; F_{T,2,Rd}; F_{T,3,Rd}\} = 314$ kN	6.2.4.1(6)

Column web in tension

No check is necessary for a row immediately adjacent to a tension stiffener.
Because the stiffener is in full depth, no check is required at the end of the stiffener.

BOLT ROW 3

As for rows 1 and 2:

m = 35.5 mm

e_{min} = 75 mm

e = 77.3 mm

Bolt row is an 'inner bolt row' according to Figure 6.9.

For failure Mode 1, $\ell_{eff,1} = \ell_{eff,nc}$ but $\ell_{eff,1} \leq \ell_{eff,cp}$

$\ell_{eff,cp}$ = $2\pi m$ = 223 mm — Table 2.2(e) in STEP 1A

$\ell_{eff,nc}$ = $4m + 1.25e$

= $(4 \times 35.5) + (1.25 \times 77.3)$ = 239 mm

As 223 < 239

$\ell_{eff,1} = \ell_{eff,cp}$ = 223 mm

For failure Mode 2, $\ell_{eff,2} = \ell_{eff,nc}$

Therefore $\ell_{eff,2}$ = 239 mm

Mode 1 resistance

$$M_{pl,1,Rd} = \frac{0.25 \times 223 \times 14.2^2 \times 275}{1.0} = 3090 \times 10^3 \text{ Nmm}$$

$$\text{Therefore, } F_{T,1,Rd} = \frac{(8 \times 44.4 - 2 \times 9.9) \times 3090 \times 10^3}{2 \times 35.5 \times 44.4 - 9.9 \times (35.5 + 44.4)} \times 10^{-3} = 439 \text{ kN}$$

Mode 2 resistance

$$M_{pl,2,Rd} = \frac{0.25 \times 239 \times 14.2^2 \times 275}{1.0} = 3310 \times 10^3 \text{ Nmm}$$

For 2 bolts in the row, $\Sigma F_{t,Rd} = 2 \times 203 \times 10^{-3} = 406 \times 10^3$ N

Therefore, for Mode 2

$$F_{T,2,Rd} = \frac{2M_{pl,2,Rd} + n\,\Sigma F_{t,Rd}}{m+n} = \frac{2 \times 3310 \times 10^3 + 44.4 \times 406 \times 10^3}{35.5 + 44.4} \times 10^{-3} = 309 \text{ kN}$$

Mode 3 resistance (bolt failure)

$F_{T,3,Rd} = \Sigma F_{t,Rd}$ = 406 kN

Resistance of column flange in bending

$F_{t,fc,Rd} = \min\{F_{T,1,Rd}\,;\,F_{T,2,Rd}\,;\,F_{T,3,Rd}\}$ = 309 kN — 6.2.4.1(6)

Column web in transverse tension

STEP 1V

$$F_{t,wc,Rd} = \frac{\omega\, b_{eff,t,wc}\, t_{wc}\, f_{y,wc}}{\gamma_{M0}}$$

6.2.6.3(1) Eq (6.15)

As before, for a double-sided connection with equal moments, ω = 1.0 — Example C.1

$b_{eff,t,wc} = \ell_{eff,2}$ = 239 mm

$f_{y,wc} = f_{y,c}$ = 275 N/mm²

Title Example C.3 – Column tension stiffener	Sheet 7 of 10

Thus

$F_{t,wc,Rd} = \dfrac{1.0 \times 239 \times 8.6 \times 275}{1.0} \times 10^{-3}$ = 565 kN

BOLT ROW 3, AS PART OF A GROUP

Because of the presence of the tension stiffener between rows 1 and 2, the only group of rows to be considered is rows 2 and 3.

Column flange in bending

STEP 1

Row 2 is a 'bolt row adjacent to a stiffener', according to Figure 6.9. The effective lengths in Table 6.5, as part of a group are:

Table 2.3(a) in STEP 1A

$\ell_{eff,cp} = \pi m + p$

$\ell_{eff,nc} = 0.5p + \alpha m - (2m + 0.625e)$

Here $p = p_{2\text{-}3}$ = 90 mm, n = 44.4 mm and m = 35.5 mm (as for row 2 alone)

$m_2 = 60 - t_s - 0.8s_s = 60 - 15 - (0.8 \times 8) = 38.6$ mm

$\ell_{eff,cp} = (\pi \times 35.5) + 90 = 202$ mm

Obtain α from Figure 6.11 or Appendix G using:

$\lambda_1 = \dfrac{m}{m+e}$ *and* $\lambda_2 = \dfrac{m_2}{m+e}$

$\lambda_1 = \dfrac{35.5}{35.5+75} = 0.32$

$\lambda_2 = \dfrac{38.6}{35.5+75} = 0.35$

From Figure 6.11, α = 7.3

$\ell_{eff,nc} = (0.5 \times 90) + (7.3 \times 35.5) - (2 \times 35.5 + 0.625 \times 75) = 186$ mm

Row 3 is an 'other end bolt-row' in Table 6.5

Table 6.6

$\ell_{eff,cp} = \pi m + p$

$= (\pi \times 35.5) + 90 = 202$ mm

Table 2.3(c) in STEP 1A

$\ell_{eff,nc} = 2m + 0.625e + 0.5p$

$= (2 \times 35.5) + (0.625 \times 75) + (0.5 \times 90) = 163$ mm

Therefore, the total effective lengths for this group of rows are:

$\Sigma\ell_{eff,cp} = 202 + 202 = 404$ mm

$\Sigma\ell_{eff,nc} = 202 + 163 = 365$ mm

$\Sigma\ell_{eff,2} = \Sigma\, \ell_{eff,nc} = 365$ mm

As 365 mm < 404 mm, $\Sigma\ell_{eff,1} = 365$ mm

Mode 1 resistance

$M_{pl,1,Rd} = \dfrac{0.25 \times 365 \times 14.2^2 \times 275}{1.0} = 5060 \times 10^3$ Nmm

$F_{T,1,Rd} = \dfrac{(8 \times 44.4 - 2 \times 9.9) \times 5060 \times 10^3}{2 \times 35.5 \times 44.4 - 9.9 \times (35.5 + 44.4)} \times 10^{-3} = 719$ kN

Mode 2 resistance

$M_{pl,2,Rd} = \dfrac{0.25 \times 365 \times 14.2^2 \times 275}{1.0} = 5060 \times 10^3$ Nmm

For 2 bolts in each row, $\Sigma F_{t,Rd} = 2 \times 203 \times 10^3 = 406 \times 10^3$ N

Therefore, for Mode 2

$$F_{T,2,Rd} = \frac{2\,M_{pl,2,Rd} + n\,\Sigma\,F_{t,Rd}}{m+n} = \frac{2\times 5060\times 10^3 + 44.4\times 2\times 406\times 10^3}{35.5+44.4}\times 10^{-3} = 575\text{ kN}$$

Mode 3 resistance (bolt failure)

$F_{T,3,Rd} = \Sigma\, F_{t,Rd} = 812$ kN

Column web in transverse tension

STEP 1B

The resistance of the column web is given by:

6.2.6.3(1)
Eq (6.15)

$$F_{t,wc,Rd} = \frac{\omega\, b_{eff,t,wc}\; t_{wc}\; f_{y,wc}}{\gamma_{M0}}$$

As before, ω = 1.0 (for a double-sided connection with equal moments)

$b_{eff,t,wc} = \ell_{eff,2}$ = 365 mm

$f_{y,wc} = f_{y,c} = 275$ N/mm^2

Thus,

$$F_{t,wc,Rd} = \frac{1.0\times 365\times 8.6\times 275}{1.0}\times 10^{-3} = 863\text{ kN}$$

The least value of resistance of the group of rows 2 and 3, $F_{t,2\text{-}3,Rd}$, is thus 575 kN

The resistance of bolt row 3 on the column side is therefore limited to:

$F_{t3,c,Rd} = F_{t,2\text{-}3,Rd} - F_{t2,Rd} = 575 - 314 = 261$ kN

SUMMARY OF RESISTANCES OF STIFFENED CONNECTION

The resistances of the stiffened connection are summarised below. It is assumed that the moments on either side of the column are equal and opposite and thus the shear resistance of the column web does not affect the moment resistance. A compression stiffener is assumed to be provided in the column.

Resistances of rows $F_{tr,Rd}$ (kN)

	Column flange	Column web	End plate	Beam web	Minimum	Effective resistance
Row 1, alone	321	N/A	377	N/A	321	321
Row 2,alone	314	N/A	406	675	314	314
Row 3, alone	309	565	406	675	309	
Row 3, with row 2	575	863	812	1052	575	
					575 − 314	261

The moment resistance of the stiffened connection is therefore:

$M_{j,Rd}$ = (565 × 321 + 465 × 314 + 375 × 261) × 10^{-3} = 425 kNm

Title	Example C.3 – Column tension stiffener	Sheet 9 of 10

STEP 6A

RESISTANCE OF TENSION STIFFENER

The tension stiffener and its weld to the flange should be adequate to resist the larger of the forces given by the two alternative empirical relationships for load-sharing between web and stiffener.

Enhancement of tension resistance of column web

The resistance of the effective length of stiffened column web (above and below the stiffener) is taken as:

$$F_{t,wc,Rd} = \frac{L_{wt} t_{wc} f_y}{\gamma_{M0}}$$

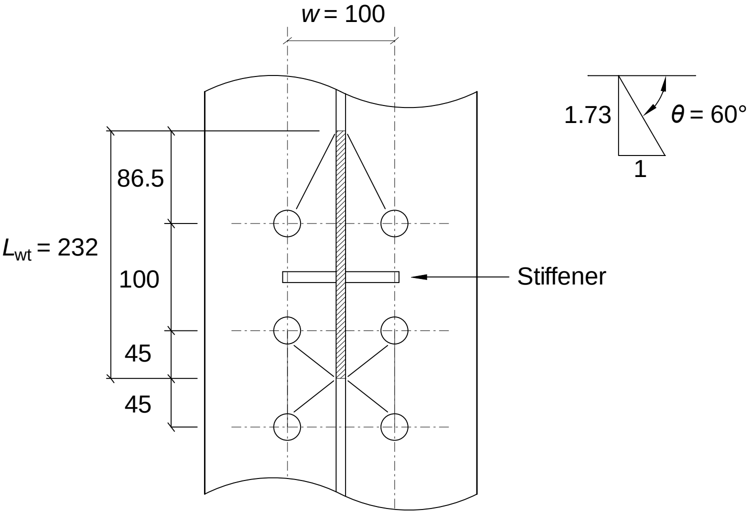
where

L_{wt} is the length of stiffened column web in tension (see diagram)

t_{wc} = 8.6 mm

f_y = 275 N/mm^2

Assuming a distribution of 60°, the length of column web in tension is as shown below.



$$L_{wt} = \left(1.73\frac{w}{2}\right) + p_{1,1-2} + \left(\frac{p_{1,2-3}}{2}\right)$$

$$= \left(1.73 \times \frac{100}{2}\right) + 100 + \left(\frac{90}{2}\right) = 232 \text{ mm}$$

Hence,

$$F_{t,wc,Rd} = \frac{232 \times 8.6 \times 275}{1.0} \times 10^{-3} = 549 \text{ kN}$$

Resistance of Rows 1 and 2 = 321 + 314 = 635 kN

So the stiffeners need to resist 635 − 549 = 86 kN

Support to column flange in bending

The forces in the four bolts located around the effective stiffener section are partly transferred to the web and partly to the stiffeners. It is assumed that the forces are shared in proportion to the distance of the bolts from the web and stiffener.

For bolt row 1, the force carried by the stiffeners is:

$$F_{t,s,1} = \frac{mF_{r1}}{(m + m_2)} = \frac{35.5 \times 321}{(35.5 + 33.6)} = 165 \text{ kN}$$

Title	Example C.3 – Column tension stiffener	Sheet 10 of 10

For row 2:

$$F_{t,s,2} = \frac{mF_{r2}}{(m + m_2)} = \frac{35.5 \times 314}{(35.5 + 43.6)} = 141 \text{ kN}$$

$$F_{t,s} = 165 + 141 = 306 \text{ kN}$$

Resistance of tension stiffeners

The area provided by the stiffeners is:

$A_{sn} = 2b_{sn}t_s$

$2 \times b_{sn} \times t_s = 2 \times 85 \times 10 = 1700 \text{ mm}^2$

Hence the resistance is:

$$F_{t,s,Rd} = \frac{A_{sn}f_{ys}}{\gamma_{M0}} = \frac{1700 \times 275}{1.0} \times 10^{-3} = 468 \text{ kN} \quad \text{Satisfactory}$$

WELD DESIGN

Use a full strength fillet weld between stiffener and flange.

In S275 steel, a full strength weld is provided by symmetrical fillet welds with a total throat thickness equal to that of the element.

Required throat = 10/2 = 5 mm

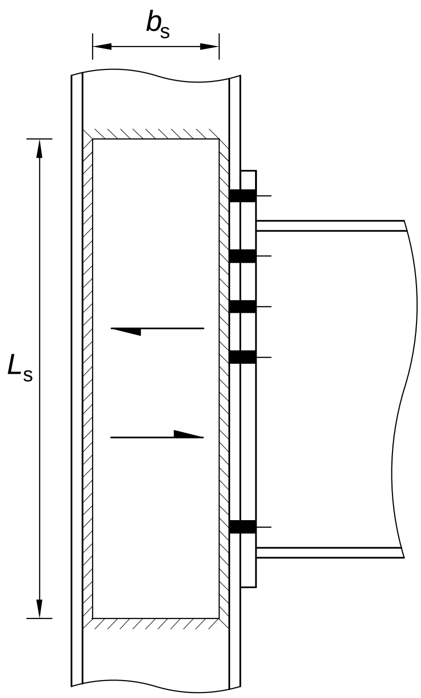
8 mm leg length weld provides a throat of $8/\sqrt{2} = 5.7$ mm, OK.

CALCULATION SHEET	Job No. *CDS 324*		*Sheet 1 of 4*
	Title *Example C.4 Supplementary column web plates*		
	Client		
	Calcs by *MEB*	Checked by *DGB*	Date *Nov 2012*

JOINT CONFIGURATION AND DIMENSIONS

Consider the beam to column connection in Example C.1 but with a beam connected on only one side of the column. To prevent the column web panel shear resistance limiting the resistance of the connection, a supplementary web plate is provided. The supplementary plate will increase column web tension and compression resistances as well as shear resistance.

References to clauses, etc. are to BS EN 1993-1-8: and its UK NA, unless otherwise stated.



SUPPLEMENTARY WEB PLATE PROPERTIES

Try a single supplementary web plate with the following details:

Steel grade S275 (as for the column)

Breadth b_s = 200 mm

Thickness t_s not less than column web thickness

Here, t_{wc} = 12.8 mm, therefore choose a plate with t_s = 15 mm

Minimum length required is the sum of three components: — STEP 6C

where:

L_1 = $b_{eff,t,wc}$ = 233/2= 117 mm (from row 1, column side, Example C.1) — Example C.1 Sheet 6

L_2 = $h_b - 60 - t_{fb}/2 = 533 - 60 - 15.6/2 = 465$ mm

L_3 = $b_{eff,c,wc}/2$ = 248/2= 124 mm (from column side, Example C.1) — Sheet 20

Therefore:

$L_s \geq 117 + 465 + 124 = 706$ mm – say 725 mm

Title	Example C.4 – Supplementary column web plate	Sheet 2 of 4

SHEAR RESISTANCE

The design plastic shear resistance of an unstiffened web is given by:

$$V_{wp,Rd} = \frac{0.9 A_{vc}(f_y/\sqrt{3})}{\gamma_{M0}}$$

BS EN 1993-1-1, 6.2.6.1(2)

Where

A_{vc} is the shear area given by BS EN 1993-1-1 and is determined as follows for rolled I and H sections with the load applied parallel to the web.

$A_{vc} = A - 2bt_f + t_f(t_w + 2r)$ But not less than $\eta h_w t_w$ — 6.2.6(3)

A_{vc} $9310 - (2 \times 254.6 \times 14.2) + 14.2(8.6 + 2 \times 12.7)$ = 2560 mm^2 — BS EN 1993-1-8 6.2.6.1(6)

$\eta h_w t_w = (1.0 \times 226 \times 8.6) = 1940$ mm^2

Where there is a single supplementary web plate, the shear area is increased by $b_s t_{wc}$

$b_s t_{wc} = 200 \times 8.6 = 1720$ mm^2

Therefore, for the stiffened web,

A_{vc} = 2560 + 1720 = 4280 mm^2

The plastic design shear resistance is:

$$V_{pl.Rd} = \frac{0.9 \times 4280 \times (275/\sqrt{3})}{1.0} \times 10^{-3} = 612 \text{ kN}$$

TENSION RESISTANCE

STEP 6C

Column web in transverse tension

STEP 1B

The effect of a single supplementary web plate that conforms to the requirements of STEP 6C and is connected with infill welds is to increase the effective web thickness in tension by 50%.

$$F_{t,wc,Rd} = \frac{\omega\, b_{eff,t,wc}\, t_{wc}\, f_{y,wc}}{\gamma_{M0}}$$

6.2.6.3(1)
Eq (6.15)

As the connection is single-sided, the transformation factor β = 1 and $\omega = \omega_1$, given by:

5.3, Table 6.3
Table 2.5 in STEP 1A

$$\omega_1 = \frac{1}{\sqrt{1 + 1.3(b_{eff,t,wc} t_{wc}/A_{vc})^2}}$$

<u>For row 1, alone</u>

$b_{eff,t,wc} = \ell_{eff,2}$ = 233 mm

t_{wc} = 1.5 × 12.8 – 19.2 mm

Thus,

$$\omega_1 = \frac{1}{\sqrt{1 + 1.3(233 \times 19.2/4280)^2}} = 0.64$$

$f_{y,wc} = f_{y,c}$ = 265 N/mm^2

$$F_{t,wc,Rd} = \frac{0.64 \times 233 \times 19.2 \times 265}{1.0} \times 10^{-3} = 759 \text{ kN}$$

<u>For rows 2 and 3, each row alone</u>

$b_{eff,t,wc} = \ell_{eff,2}$ = 243 mm

Thus,

$$\omega_1 = \frac{1}{\sqrt{1 + 1.3(243 \times 19.2/4280)^2}} = 0.63$$

$f_{y,wc} = f_{y,c}$ = 265 N/mm^2

Title	Example C.4 – Supplementary column web plate	Sheet 3 of 4

$$F_{t,wc,Rd} = \frac{0.63 \times 243 \times 19.2 \times 265}{1.0} \times 10^{-3} = 779 \text{ kN}$$

For row 1 and 2 combined

$b_{eff,t,wc} = \ell_{eff,2} = 332$ mm

$$\omega_1 = \frac{1}{\sqrt{1 + 1.3(332 \times 19.2 / 4280)^2}} = 0.51$$

$$F_{t,wc,Rd} = \frac{0.51 \times 332 \times 19.2 \times 265}{1.0} \times 10^{-3} = 862 \text{ kN}$$

For rows 1, 2 and 3 combined

$b_{eff,t,wc} = \ell_{eff,2} = 422$ mm

$$\omega_1 = \frac{1}{\sqrt{1 + 1.3(422 \times 19.2 / 4280)^2}} = 0.42$$

$$F_{t,wc,Rd} = \frac{0.42 \times 422 \times 19.2 \times 265}{1.0} \times 10^{-3} = 902 \text{ kN}$$

For rows 2 and 3 combined

$b_{eff,t,wc} = \ell_{eff,2} = 323$ mm

$$\omega_1 = \frac{1}{\sqrt{1 + 1.3(323 \times 19.2 / 4280)^2}} = 0.52$$

$$F_{t,wc,Rd} = \frac{0.52 \times 323 \times 19.2 \times 265}{1.0} \times 10^{-3} = 855 \text{ kN}$$

Column web in transverse compression

STEP 2

STEP 6C

The effect of a single supplementary web plate that conforms to the requirements of STEP 6C is to increase the effective web thickness in tension by 50%.

As the connection is single-sided, the transformation factor $\beta = 1$ (as above) and $\omega = \omega_1$

$$\omega_1 = \frac{1}{\sqrt{1 + 1.3(b_{eff,c,wc} t_{wc} / A_{vc})^2}}$$

$b_{eff,c,wc} = \ell_{eff,2} = 248$ mm

Thus

$$\omega_1 = \frac{1}{\sqrt{1 + 1.3(248 \times 19.2 / 4280)^2}} = 0.62$$

$$F_{t,wc,Rd} = \frac{0.62 \times 248 \times 19.2 \times 265}{1.0} \times 10^{-3} = 782 \text{ kN}$$

Title Example C.4 – Supplementary column web plate | Sheet 4 of 4

SUMMARY OF RESISTANCES

Resistances of rows $F_{tr,Rd}$ (kN)

	Column flange	Column web	End plate	Beam web	Minimum	Effective resistance
Row 1, alone	398	759	377	N/A	377	377
Row 2,alone	398	779	406	675	398	
Row 2, with row 1	697	862	N/A	N/A	697	
Row 2					697 − 377	320
Row 3, alone	398	779	406	675	398	
Row 3, with row 1 & 2	988	902	N/A	N/A	902	
Row 3					902-697	205
Row 3, with row 2	691	855	812	1052	691	
Row 3					691 − 320	

EQUILIBRIUM OF FORCES

STEP 4

The total effective tension resistance $\Sigma F_{tr,Rd}$ = 377 + 320 + 205 = 902 kN, which exceeds both the compression resistance $F_{c,Rd}$ = 782 kN and the shear resistance $V_{pl,Rd}$ = 612 kN. The forces in both rows 3 and 2 need to be reduced to maintain equilibrium.

The revised forces in the tension zone are:

$F_{t1,Rd}$ = 377 kN

$F_{t2,Rd}$ = 235 kN

$F_{t3,Rd}$ = 0 kN

MOMENT RESISTANCE OF JOINT

The moment resistance of the beam to column joint ($M_{j,Rd}$) is given by: 6.2.7.2(1)

$$M_{j,Rd} = h_{r1}F_{r1} + h_{r2}F_{r2} + h_{r3}F_{r3}$$ (6.25)

$$= (565 \times 377 + 465 \times 235 + 375 \times 0) \times 10^{-3} = 322 \text{ kNm}$$

The overall effect, relative to Example C.1, is a reduction in the moment resistance of the joint. However, this is due largely to the change from a balanced two-sided joint to a single sided joint and the resistance would have been even less without the supplementary web plate.

WELDS

Because the supplementary web plate is provided to increase web tension resistance, ‘fill in’ welds should be provided.

Horizontal welds

Fillet welds of leg length equal to the supplementary plate thickness should be used:

Leg length = 15 mm

Vertical welds

Fillet welds of leg length equal to the supplementary plate thickness should be used:

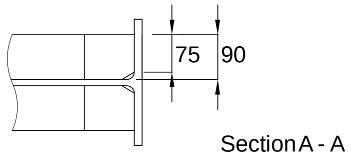
Leg length = 15 mm

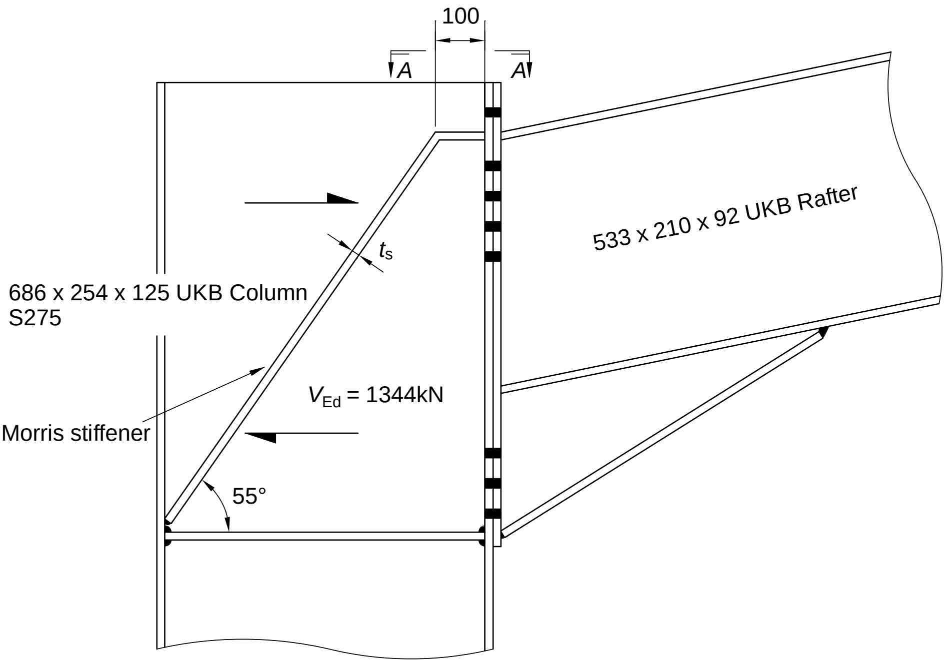
CALCULATION SHEET	Job No. CDS 324		Sheet 1 of 2
	Title Example C.5 Haunched connection with Morris stiffener		
	Client		
	Calcs by MEB	Checked by DGB	Date Nov 2012

JOINT CONFIGURATION AND DIMENSIONS

References to clauses, etc. are to BS EN 1993-1-8: and its UK NA, unless otherwise stated.

Morris stiffeners, or other diagonal stiffeners, can be provided to resist high web panel shear forces. In portal frame design the columns are usually a universal beam section and diagonal stiffeners are often found to be necessary, in addition to compression stiffeners.





DIMENSIONS AND PROPERTIES

Column

For a 686 × 254 × 125 UKB S275

Depth	h_c	= 677.9 mm
Width	b_c	= 253.0 mm
Thickness of web	t_{wc}	= 11.7 mm
Thickness of flange	t_{fc}	= 16.2 mm

For S275 and 16 mm < t_{fc} ≤ 40 mm — BS EN 10025-2 Table 7

Yield strength f_{yc} $= R_{eH} = 265$ N/mm^2

Morris stiffener in S275

Overall width b_{sg} = 100 mm

Title	Example C.5 Haunched connection with Morris stiffener	Sheet 2 of 2

For S275 and $t_s \leq 16$ mm — BS EN 10025-2 Table 7

Yield strength $f_{ys} = R_{eH} = 275$ N/mm^2

RESISTANCES OF STIFFENED JOINT

MOMENT RESISTANCE

With the Morris stiffener acting as a tension stiffener between bolt rows 1 and 2, the total effective tension resistance of all the bolt rows is 1500 kN.

However, the shear resistance of the unstiffened column web is only 1280 kN and this would limit the moment resistance.

The design requirement for the Morris stiffener is to increase the column web shear resistance to at least 1500 kN.

SHEAR RESISTANCE OF STIFFENED COLUMN WEB

To achieve a shear resistance at least equal to the total tension resistance: — STEP 6D

The gross areas of the stiffeners, A_{sg} must be such that:

$$A_{sg} \geq \frac{V_{Ed} - V_{Rd}}{f_y \cos\theta}$$

Where:

$A_{sg} = 2b_{sg}t_s$

b_{sg} is the gross width of stiffener on each side of the column web = 100 mm

t_s is the thickness of the stiffener.

V_{Ed} is the design shear force acting on the column, taken as the total tension resistance of the bolt rows = 1500 kN

V_{Rd} is the design shear resistance of the unstiffened column web

V_{Rd} =1280 kN

f_y is the weaker yield strength of the stiffener or column, assumed to be = 265 N/mm^2

θ is the angle of the stiffener to the horizontal.

$\theta = 55°$

$$A_{sg} \quad \frac{(1500 - 1280) \times 10^3}{265 \times \cos 55} = 1450 \text{ mm}^2$$

Therefore the minimum required thickness is given by:

$$t_s = \frac{A_{sg}}{2b_{sg}} = \frac{1450}{(2 \times 100)} = 7.25 \text{ mm}$$

Therefore, adopt a thickness for the Morris stiffener of t_s = 10 mm

WELD DESIGN

STIFFENER TO COLUMN FLANGE WELDS

Provide full strength fillet welds between the Morris stiffeners and column flanges.

The required weld throat thickness = t_s /2 = 10/2 = 5 mm

An 8 mm leg length fillet weld provides a throat of $a = 8/\sqrt{2} = 5.7$ mm, OK

STIFFENER TO COLUMN WEB WELDS

Provide a nominal fillet weld between the column web and Morris stiffener.

Therefore, adopt a weld with a leg length of 8 mm.

APPENDIX D WORKED EXAMPLE – BOLTED BEAM SPLICE

One worked example is presented in this Appendix:

Example D.1 Splice between UKB beam sections

The example follows the recommendations in the main text. Additionally, references to the relevant clauses, Figures and Tables in BS EN 1993-1-8 and its UK National Annex are given where appropriate; these are given simply as the clause, Figure or Table number. References to clauses etc. in other standards are given in full. References to Tables or Figures in the main text are noted accordingly; references to STEPS are to those in Section 4 of the main text.

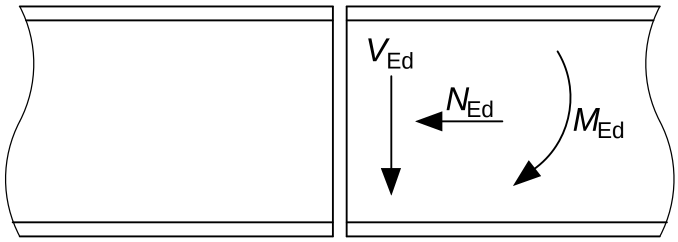
CALCULATION SHEET	Job No. *CDS 324*		Sheet *1* of *12*
	Title *Example D.1 - Beam Splice*		
	Client		
	Calcs by *MEB*	Checked by *DGB*	Date *Nov 2012*

References to clauses, etc. are to BS EN 1993-1-8: and its UK NA, unless otherwise stated.

JOINT CONFIGURATION AND DIMENSIONS

Design a bolted cover plate beam splice that connects two 457 × 191 × 67 UKB S275 sections. The splice carries a vertical shear, an axial force and bending moment and is to be non slip at serviceability (Category B connection).

The splice is located near to a restraint therefore it will not carry moments due to strut action.



DESIGN VALUES OF FORCES ON BEAM AT THE SPLICE

Values at ultimate limit state

V_{Ed} = 150 kN

N_{Ed} = 150 kN (compression)

M_{Ed} = 200 kNm

Values at serviceability limit state

$V_{Ed,ser}$ = 100 kN

$N_{Ed,ser}$ = 100 kN (compression)

$M_{Ed,ser}$ = 133 kNm

Title	Example D.1 Beam splice	Sheet 2 of 12

DIMENSIONS AND SECTION PROPERTIES

Beam

From data tables for 457 × 191 × 67 UKB S275:

Depth	h	= 453.4 mm	P363
Width	b	= 189.9 mm	
Web thickness	t_w	= 8.5 mm	
Flange thickness	t_f	= 12.7 mm	
Root radius	r	= 10.2 mm	
Depth between flange fillets	d_b	= 407.6 mm	
Second moment of area, y-y axis	I_y	= 29 400 cm^4	
Plastic modulus, y-y axis	$W_{pl,y,}$	= 1 470 cm^3	
Area	A	= 85.5 cm^2	

Cover plates

Assume, initially, 12 mm thick cover plates for the flanges and 10 mm thick cover plates for the web. Thickness and dimensions to be confirmed below.

Bolts

Two possible sizes will be considered:

M20 preloaded class 8.8 bolts

Diameter of bolt shank	d	= 20 mm
Diameter of hole	d_0	= 22 mm
Shear area	A_s	= 245 mm^2

M24 preloaded class 8.8 bolts

Diameter of bolt shank	d	= 24 mm
Diameter of hole	d_0	= 26 mm
Shear area	A_s	= 353 mm^2

MATERIAL STRENGTHS

Beam and cover plates

For buildings that will be built in the UK, the nominal values of the yield strength (f_y) and the ultimate strength (f_u) for structural steel should be those obtained from the product standard. Where a range is given, the lowest nominal value should be used. (BS EN 1993-1-1 NA.2.4)

S275 steel (BS EN 10025-2 ,Table 7)

For $t \leq 16$ mm	f_y	$= R_{eH}$ = 275 N/mm^2
For 3 mm $\leq t \leq$ 100 mm	f_u	$= R_m$ = 410 N/mm^2

Hence, for the beam, flange cover plates and web cover plates:

$f_{y,b} = f_{y,wp}$ = 275 N/mm^2

$f_{u,b} = f_{u,wp}$ = 410 N/mm^2

Bolts

Nominal yield strength	f_{yb}	= 640 N/mm^2	Table 3.1
Nominal ultimate strength	f_{ub}	= 800 N/mm^2	

Title	Example D.1 Beam splice	Sheet 3 of 12

PARTIAL FACTORS FOR RESISTANCE

Structural steel

BS EN 1993-1-1 NA.2.15

γ_{M0} = 1.0

γ_{M1} = 1.0

γ_{M2} = 1.1

Parts in connections

Table NA.1

γ_{M2} = 1.25 (bolts, welds, plates in bearing)

γ_{M3} = 1.25 (slip resistance at ULS)

$\gamma_{M3,ser}$ = 1.10 (slip resistance at SLS)

INTERNAL FORCES AT SPLICE

STEP 1

6.2.7.1(16)

For a splice in a flexural member, the parts subject to shear (the web cover plates) must carry, in addition to the shear force and the moment due to the eccentricity of the centroids of the bolt groups on each side, the proportion of moment carried by the web, without any shedding to the flanges

The second moment of area of the web is:

$$I_w = \frac{(h-2t_f)^3 t_w}{12} = \frac{428^3 \times 8.5}{12} \times 10^{-4} = 5550 \text{ cm}^4$$

Therefore, the web will carry 5550/29400 = 18.9% of the moment in the beam (assuming an elastic stress distribution). The flanges carry the remaining 81.1%

The area of the web is:

A_w = 428 × 8.5 × 10^{-2} = 36.4 cm^2

The web will therefore also carry 36.4/85.5 = 42.6% of the axial force in the beam. The flanges carry the remaining 57.4%.

FORCES AT ULS

The force in each flange due to bending is therefore given by:

$$F_{f,M,Ed} = 0.811 \frac{M_{Ed}}{(h-t_f)} = 0.811 \times \frac{200 \times 10^6}{453.4 - 12.7} \times 10^{-3} = 368 \text{ kN}$$

And the force in each flange due to axial force is given by:

$F_{f,N,Ed}$ = 0.574 × 150/2 = 43 kN

Thus:

$F_{tf,Ed}$ = 368 − 43 = 325 kN

$F_{bf,Ed}$ = 368 + 43 = 411 kN

The moment in the web = 0.189 × 200 = 37.8 kNm

The axial force in the web = 0.426 ×150 = 63.9 kN

The shear force in the web = 150 kN

FORCES AT SLS

The force in each flange due to bending is given by:

$$F_{f,M,Ed} = 0.811\frac{M_{Ed}}{(h-t_f)} = 0.811\times\frac{133\times10^6}{453.4-12.7}\times10^{-3} = 245 \text{ kN}$$

The force in each flange due to axial force is given by:

$F_{f,N,Ed}$ = 0.574 × 100/2 = 28.7 kN

Thus:

$F_{tf,Ed}$ = 245 − 29 = 216 kN

$F_{bf,Ed}$ = 245 + 29 = 274 kN

The moment in the web = 0.189 × 133 = 25.1 kNm

The axial force in the web = 0.426 ×100 = 42.6 kN

The shear force in the web = 100 kN

CHOICE OF BOLT NUMBER AND CONFIGURATION

RESISTANCES OF BOLTS

STEP 2

The shear resistance of bolts (at ULS) is given by P363: P363

For M20 bolts in single shear 94.1 kN

For M20 bolts in double shear 188 kN

The slip resistance of bolts (at SLS), assuming a class A friction surface is given by P363 as: P363

For M20 bolts in single shear 62.4 kN

For M20 bolts in double shear 125 kN

Assuming that the cover plate thicknesses and bolt spacings are such that the shear resistances of the bolts can be achieved, consider the number of bolts in the flanges and web.

FLANGE SPLICE

STEP 2

For the flanges, the force of 411 kN at ULS can be provided by 6 M20 bolts in single shear. The force of 274 kN at SLS can also be provided by 6 M20 bolts.

The full bearing resistance of an M20 bolt in a 12 mm cover plate (i.e. without reduction for spacing and end/edge distance) is:

$$F_{b,max,Rd} = \frac{2.5 f_u dt}{\gamma_{M2}} = \frac{2.5\times410\times20\times12}{1.25}\times10^{-3} = 197 \text{ kN}$$

Table 3.4

This is much greater than the resistance of the bolt in single shear and thus the spacings do not need to be such as to maximize the bearing resistance. Three lines of 2 bolts at a convenient spacing may be used.

WEB SPLICE

STEP 4

For the web splice, consider one or two lines of 3 bolts on either side of the centreline.

The full bearing resistance on the 8.5 mm web is:

$$F_{b,max,Rd} = \frac{2.5 f_u dt}{\gamma_{M2}} = \frac{2.5\times410\times20\times8.5}{1.25}\times10^{-3} = 139 \text{ kN}$$

This is less than the resistance in double shear and will therefore determine the resistance at ULS. To achieve this value, the spacings will need to be:

Table 3.4

e_1 $\geq 3d_0 = 3 \times 22 = 66$ mm

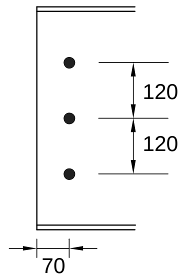
p_1 $\geq 15d_0/4 = 15 \times 22 / 4 = 83$ mm

e_2 $\geq 1.5d_0 = 1.5 \times 22 = 33$ mm

p_2 $\geq 3d_0 = 3 \times 22 = 66$ mm

The distinction between 'end' and 'edge' will depend on the direction of the dominant force on the bolt being considered.

Initially, try 3 bolts at a vertical spacing of 120 mm at a distance of 70 mm from the centreline of the splice. (If the cover plate is 340 mm deep, the end/edge distance at top and bottom is 50 mm.)



The additional moment due to the eccentricity of the bolt group is:

M_{add} = 150 × 0.07 = 10.5 kNm

Bolt forces at ULS

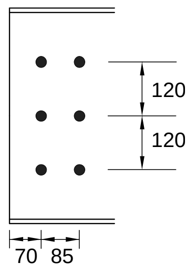
Force/bolt due to vertical shear = 150/3 = 50 kN

Force/bolt due to axial compression = 63.9/3 = 21.3 kN

Force/bolt due to moment = (37.8 + 10.5)/0.24 = 201 kN (top and bottom bolts only)

Thus, a single row is clearly inadequate.

Consider a second line of bolts at a horizontal pitch of 85 mm



Now:

Force/bolt due to vertical shear = 150/6 = 25 kN

Force/bolt due to axial compression = 63.9/6 = 10.7 kN

The additional moment due to the eccentricity of this bolt group is:

M_{add} = 150 × (0.07 + 0.085/2) = 16.6 kNm

The polar moment of inertia of the bolt group is given by:

$$I_{bolts} = 4 \times 120^2 + 6 \times (85/2)^2 = 68400 \text{ mm}^2$$

The horizontal component of the force on each top and bottom bolt is:

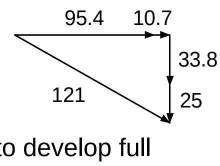
$$F_{M,horiz} = \frac{(37.8 + 16.6) \times 120}{68400} \times 10^3 = 95.4 \text{ kN}$$

The vertical component of the force on each bolt is:

$$F_{M,vert} = \frac{(37.8 + 16.6) \times 42.5}{68400} \times 10^3 = 33.8 \text{ kN}$$

Thus, the resultant force on the most highly loaded bolt is:

$$F_{v,Ed} = \sqrt{(25 + 33.8)^2 + (95.4 + 10.7)^2} = 121 \text{kN}$$



This is less than the full bearing resistance and is therefore satisfactory for such a bolt spacing.

Note: If a configuration with bolt spacings that are less than those needed to develop full bearing resistance is selected, a detailed evaluation of the bearing resistances of individual bolts, taking account of the direction of the force relative to an end and the edge, would need to be carried out. Similarly, if the joint were 'long', a reduction would be needed.

Bolt forces at SLS

For the 6 bolt configuration:

Force/bolt due to vertical shear = 100/6 = 16.7 kN

Force/bolt due to axial compression = 42.6/6 = 7.1 kN

The additional moment due to the eccentricity of this bolt group is:

M_{add} = 100 × 0.113 = 11.3 kNm

$$F_{M,horiz} = \frac{(25.1 + 11.3) \times 120}{68400} \times 10^3 = 63.9 \text{ kN}$$

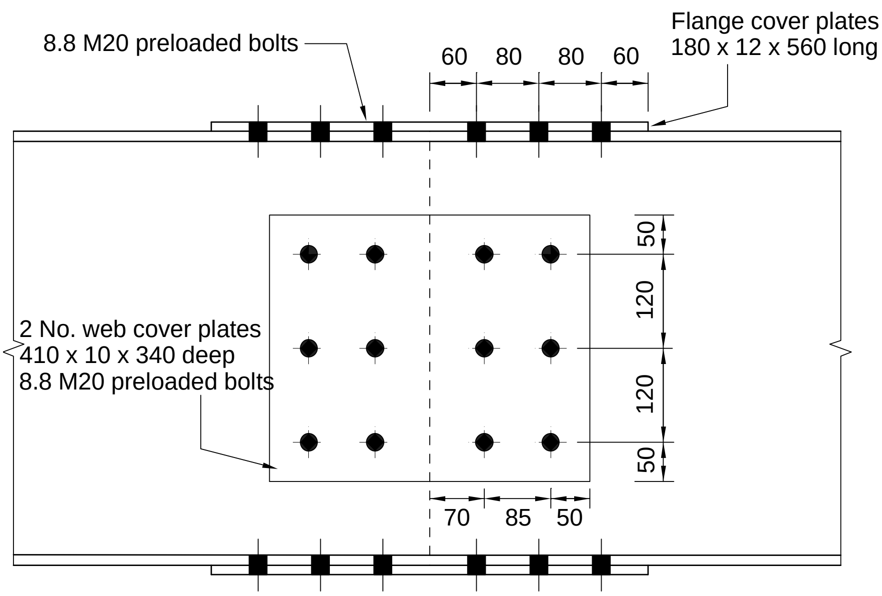
$$F_{M,vert} = \frac{(25.1 + 11.3) \times 42.5}{68400} \times 10^3 = 22.6 \text{ kN}$$

Thus, the resultant force on the most highly loaded bolt is:

$$F_{v,Ed} = \sqrt{(16.7 + 22.6)^2 + (63.9 + 7.1)^2} = 81 \text{kN}$$

This is less than the slip resistance in double shear.

CHOSEN JOINT CONFIGURATION



SUMMARY OF COVER PLATE DIMENSIONS AND BOLT SPACING

Flange cover plates

Thickness	t_{fp}	= 12 mm
Length	h_{fp}	= 560 mm
Width	b_{fp}	= 180 mm
End distance	$e_{1,fp}$	= 60 mm
Edge distance	$e_{2,fp}$	= 30 mm
Spacing:		
In the direction of the force	$p_{1,f}$	= 80 mm
Transverse to direction of force	$p_{2,f}$	= 120 mm
Across the joint in direction of force	$p_{1,f,j}$	= 120 mm

Note: The edge, end and spacing dimensions given above meet the requirements in Table 3.3. For brevity those verifications have not been shown.

Title	Example D.1 Beam splice	Sheet 7 of 12

Web cover plates ('1' direction taken as vertical)

Thickness	t_{wp}	= 10 mm
Height	h_{fp}	= 340 mm
Width	b_{fp}	= 410 mm
End distance	$e_{1,wp}$	= 50 mm
Edge distance	$e_{2,wp}$	= 50 mm
Spacing:		
Vertically	$p_{1,w}$	= 120 mm
Horizontally	$p_{2,w}$	= 85 mm
Horizontally, across the joint	$p_{1,w,j}$	= 140 mm

Note: The edge, end and spacing dimensions given above meet the requirements in Table 3.2 of BS EN 1993-1-8. For brevity those verifications have not been shown.

RESISTANCE OF FLANGE SPLICES

STEP 3

RESISTANCE OF BOLT GROUP

The above configuration provides edge, end and spacing distances that are larger than the values given in the first table of bearing resistances on page C-381 of P363. The bearing resistance in the 12 mm S275 cover plate is therefore at least 101 kN. This is greater than the resistance of the bolt in single shear (94.1 kN), so the shear resistance of the bolt will be critical.

The flange of the beam is 12.7 mm, (thicker than the cover plate) so will not be critical.

As the length of the bolt group is only 160 mm, there is no reduction for a 'long joint' (the length is less than 15*d* = 300 mm).

The shear resistance of the fasteners is 6 × 94.1 = 565 kN which is greater than the force in the compression flange (411 kN).

RESISTANCE OF COVER PLATE TO TENSION FLANGE

Resistance of net section

The resistance of a flange cover plate in tension ($N_{t,fp,Rd}$) is the lesser of $N_{pl,Rd}$ and $N_{u,Rd}$. Here,

BS EN 1993-1-1, 6.2.3.(2)

$$N_{u,Rd} = \frac{0.9\, A_{net,fp}\, f_{u,fp}}{\gamma_{M2}}$$

where:

$$A_{net,fp} = \left(b_{fp} - 2d_0\right)t_{fp} = \left(180 - \{2 \times 22\}\right) \times 12 = 1632 \text{ mm}$$

Therefore,

$$N_{u,Rd} = \frac{0.9 \times 1632 \times 410}{1.1} \times 10^{-3} = 547 \text{ kN}$$

$$N_{pl,Rd} = \frac{A_{fp}\, f_{y,fp}}{\gamma_{M0}} = \frac{180 \times 12 \times 275}{1.0} \times 10^{-3} = 594 \text{ kN}$$

As, 594 kN > 547 kN,

$N_{t,fp,Rd}$ = 547 kN

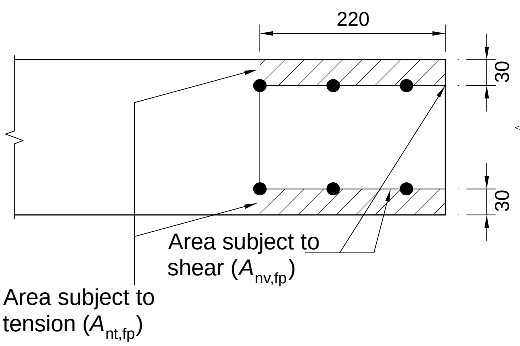
For the tension flange, N_{Ed} = 368 – 43 = 325 kN

Therefore the tension resistance of a flange cover plate is adequate.

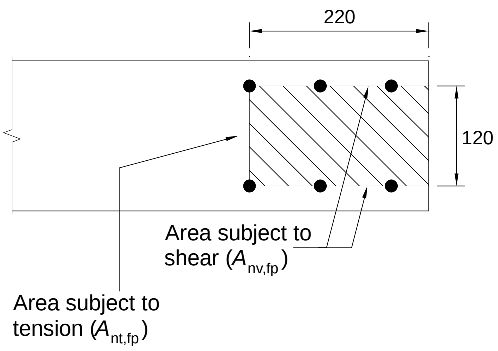
Block tearing resistance

3.10.2

a)



b)



$n_{1,fp}$ = 3 and $n_{2,fp}$ = 2

In this example, the edge distance (30 mm) is much less than the transverse spacing (120 mm) and therefore the block tearing failure area shown as a) above should be considered. However, if $p_{2,fp} < 2e_{2,fp}$ the block tearing failure area shown in b) above should be considered.

The resistance to block tearing ($N_{t,Rd,fp}$) is given by:

Eq. (3.9)

$$N_{t,fp,Rd} = \frac{f_{u,fp} A_{nt,fp}}{\gamma_{M2}} + \frac{A_{nv,fp}\left(f_{y,fp} / \sqrt{3}\right)}{\gamma_{M0}}$$

where:

$A_{nt,fp} = t_{fp}\left(2e_{2,fp} - d_0\right)$

$= 12 \times \left((2 \times 30) - 22\right) = 456 \text{ mm}^2$

$A_{nv,fp} = 2t_{fp}\left[(n_{1,fp} - 1)p_{1,fp} + e_{1,fp} - (n_{1,fp} - 0.5)d_0\right]$

$= 2 \times 12\left[(4-1) \times 80 + 60 - (4 - 0.5) \times 22\right] = 5352 \text{ mm}^2$

Therefore,

$$N_{t,fp,Rd} = \left(\frac{410 \times 456}{1.1} + \frac{5352 \times 275/\sqrt{3}}{1.0}\right) \times 10^{-3} = 1020 \text{ kN}$$

Note: γ_{M2} = 1.1 taken from BS EN 1993-1-1, as it is used with the ultimate strength.

Therefore, the resistance to block tearing of the flange cover plates is adequate.

As $t_f > t_{fp}$ and $b > b_{fp}$ the resistance of the beam flange to block tearing is adequate.

RESISTANCE OF COVER PLATE TO COMPRESSION FLANGE

Buckling resistance of the flange cover plate in compression.

Local buckling between the bolts need not be considered if:

Note 2 to Table 3.3

$$\frac{p_1}{t} \leq 9\varepsilon$$

where:

BS EN 1993-1-1 Table 5.2

$\varepsilon = \sqrt{\frac{235}{f_{y,fp}}} = \sqrt{\frac{235}{275}} = 0.92$

$9\varepsilon = 9 \times 0.92 = 8.28$

Here, the maximum bolt spacing is across the centreline of the splice, $p_{1,f,j}$ = 120 mm

Title *Example D.1 Beam splice*	*Sheet* *9 of 12*
$\dfrac{p_{1,f,j}}{t_{fp}} = \dfrac{120}{12} = 10$	
As 10 > 8.28 the local buckling verification is required for length $p_{1,f,j}$.	
The buckling resistance is given by:	
$N_{b,fp,Rd} = \dfrac{\chi A_{fp} f_{y,fp}}{\gamma_{M1}}$	BS EN 1993-1-1, 6.3.1.1(3)
$A_{fp} = b_{fp} t_{fp} = 180 \times 12 = 2160 \text{ mm}^2$	
$\chi = \dfrac{1}{\Phi + \sqrt{(\Phi^2 - \bar{\lambda}^2)}}$ but $\chi \leq 1.0$	
where:	
$\Phi = 0.5 + \left(1 + \alpha\left(\bar{\lambda} - 0.2\right) + \bar{\lambda}^2\right)$	
$\bar{\lambda}$ is the slenderness for flexural buckling	
$\bar{\lambda} = \sqrt{\dfrac{Af_y}{N_{cr}}} = \left(\dfrac{L_{cr}}{i}\right)\left(\dfrac{1}{\lambda_1}\right)$ (For Class 1, 2 and 3 cross-sections)	BS EN 1993-1-1, 6.3.1.3(1) Eq (6.50)
$L_{cr} = 0.6 p_{1,f,j}$	Note 2 to Table 3.3
$L_{cr} = 0.6 \times 120 = 72$ mm	
$\lambda_1 = 93.9\varepsilon$	
$\lambda_1 = 93.9 \times 0.92 = 86.39$	
Slenderness for buckling about the minor axis (z-z)	
$i_z = \dfrac{t_{fp}}{\sqrt{12}} = \dfrac{12}{\sqrt{12}} = 3.46 \text{ mm}$	
$\bar{\lambda}_z = \left(\dfrac{L_{cr}}{i_z}\right)\left(\dfrac{1}{\lambda_1}\right) = \left(\dfrac{72}{3.46}\right)\left(\dfrac{1}{86.39}\right) = 0.24$	BS EN 1993-1-1. Eq (6.50)
For a solid section in S275 steel use buckling curve *'c'*	Table 6.2
For buckling curve *'c'* the imperfection factor is, $\alpha = 0.49$	Table 6.1
$\Phi = 0.5\left(1 + \alpha\left(\bar{\lambda}_z - 0.2\right) + \bar{\lambda}_z^2\right)$	6.3.1.2(1)
$= 0.5 \times \left(1 + 0.49 \times \left(0.24 - 0.2\right) + 0.24^2\right) = 0.54$	
$\chi = \dfrac{1}{\Phi + \sqrt{(\Phi^2 - \bar{\lambda}_z^2)}}$	Eq. (6.49)
$= \dfrac{1}{0.54 + \sqrt{(0.54^2 - 0.24^2)}} = 0.98$	
As 0.98 < 1.0	
$\chi = 0.98$	
Thus,	
$N_{b,fp,Rd} = \dfrac{\chi A_{fp} f_{y,fp}}{\gamma_{M1}}$	
$= \dfrac{0.98 \times 2160 \times 275}{1.0} \times 10^{-3} = 582 \text{ kN}$	
For the compression flange, $N_{Ed} = 368 + 43 = 411$ kN	Sheet 3
Therefore the buckling resistance of the flange cover plate is adequate.	

RESISTANCE OF WEB SPLICE

STEP 4

RESISTANCE OF BOLT GROUP

The resistance of the most heavily loaded bolt was shown to be adequate if the edge and end distances are sufficiently large that they do not limit the bearing resistance. Those minimum distances have been achieved in the chosen configuration, hence the bolt resistance is adequate.

RESISTANCE OF WEB COVER PLATE IN SHEAR

STEP 4

The gross shear area is given by:

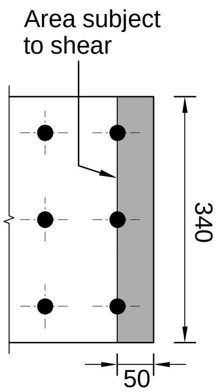
$$V_{\mathrm{wp,g,Rd}} = \frac{h_{\mathrm{wp}} t_{\mathrm{wp}}}{1.27} \frac{f_{\mathrm{y,wp}}}{\sqrt{3}\,\gamma_{\mathrm{M0}}}$$

For two web cover plates

$$V_{\mathrm{wp,g,Rd}} = 2 \times \frac{340 \times 10}{1.27} \times \frac{275}{\sqrt{3} \times 1} \times 10^{-3} = 850 \text{ kN}$$

V_{Ed} = 150 kN, therefore the shear resistance is adequate

The net shear area is given by:



$$V_{\mathrm{wp,net,Rd}} = \frac{A_{\mathrm{v,wp,net}} \left(f_{\mathrm{u,wp}} / \sqrt{3} \right)}{\gamma_{\mathrm{M2}}}$$

$$A_{\mathrm{v,net}} = \left(h_{\mathrm{wp}} - 3 d_0 \right) t_{wp}$$

$$= \left(340 - 3 \times 22 \right) \times 10 = 2740 \text{ mm}^2$$

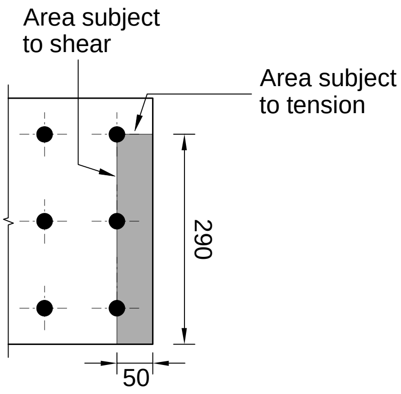
For two web cover plates:

$$V_{\mathrm{n,Rd}} = 2 \times \frac{2740 \times \left(410 / \sqrt{3} \right)}{1.1} \times 10^{-3} = 1180 \text{ kN}$$

V_{Ed} = 150 kN, therefore the shear resistance is adequate

Title	*Example D.1 Beam splice*	*Sheet 11 of 12*

Resistance to block tearing



$$V_{\mathrm{b,Rd}} = \frac{f_{\mathrm{u,wp}} A_{\mathrm{nt}}}{\gamma_{\mathrm{M2}}} + \frac{f_{\mathrm{y,wp}}}{\sqrt{3}} \frac{A_{\mathrm{nv}}}{\gamma_{\mathrm{M0}}}$$

For a single vertical line of bolts

$$A_{\mathrm{nt}} = t_{\mathrm{wp}} \left(e_2 - \frac{d_0}{2} \right)$$

$$= 10 \times \left(50 - \frac{22}{2} \right) = 390 \ \mathrm{mm}^2$$

$$A_{\mathrm{nv}} = t_{\mathrm{wp}} \ (h_{\mathrm{wp}} - e_1 - (n_1 - 0.5) d_0)$$

$$= 10 \times [340 - 50 - ((3 - 0.5) \times 22)] = 2350 \ \mathrm{mm}^2$$

For two web plates

$$V_{\mathrm{b,Rd}} = 2 \times \left[\left(\frac{410 \times 390}{1.1} \right) + \left(\frac{275}{\sqrt{3}} \times \frac{2350}{1.0} \right) \right] \times 10^{-3} = 1040 \ \mathrm{kN}$$

$$V_{\mathrm{Rd}} = \min \{850, 870, 1040\}$$

$$V_{\mathrm{Rd}} = 850 \ \mathrm{kN}$$

$$\frac{V_{\mathrm{Ed}}}{V_{\mathrm{Rd}}} = \frac{150}{850} = 0.18 < 1.0$$

Therefore the shear resistance of the web cover plates is adequate.

RESISTANCE OF BEAM WEB

The shear resistance of the beam (based on gross shear area) will have been verified in the design of the beam.

Resistance of net shear area

3.10.2

$$V_{\mathrm{n,w,Rd}} = \frac{A_{\mathrm{v,net}} \left(f_{\mathrm{u}} / \sqrt{3} \right)}{\gamma_{\mathrm{M2}}}$$

Where:

$$A_{\mathrm{v,net}} = A_{\mathrm{v}} - 3 d_0 t_{\mathrm{w}}$$

$A_{\mathrm{v}} = A - 2bt_{\mathrm{f}} + (t_{\mathrm{w}} + 2r) t_{\mathrm{f}}$ but not less than $\eta h_{\mathrm{w}} t_{\mathrm{w}}$

$$= 8552 - (2 \times 189.9 \times 12.7) + (8.5 + 2 \times 10.7) \times 12.7 = 4110 \ \mathrm{mm}^2$$

$$= 4110 - (3 \times 22 \times 8.5) = 3550 \ \mathrm{mm}^2$$

Title	Example D.1 Beam splice	Sheet 12 of 12

Thus,

$$V_{n,w,Rd} = \frac{3550 \times \left(410/\sqrt{3}\right)}{1.1} \times 10^{-3} = 764 \text{ kN}$$

Resistance to block tearing

Block shear resistance is applicable to a notched beam. Therefore it is not applicable for the connection considered here.

RESISTANCE OF WEB COVER PLATE TO COMBINED BENDING, SHEAR AND AXIAL FORCE

Following the principles of clauses 6.2.10 and 6.2.9.2, the web cover plates will be verified for the combination of bending moment and axial force. The design resistance of the cover plates will be reduced if $V_{Ed,} > V_{wp,Rd}$.

$V_{wp,Rd}$ = 850 kN

V_{Ed} = 150 kN < 850 kN — Sheet 1

Therefore, the effects of shear can be neglected.

A_{wp} = 10 × 340 = 3400 mm^2

Modulus of the cover plate

$$= \frac{10 \times 340^2}{6} = 192.7 \times 10^3 \text{ mm}^3$$

$N_{wp,Rd}$ = 10 × 340 × 275 × 10^{-3} = 935 kN

Therefore, for two web cover plates

$$M_{c,wp,Rd} = \frac{2 \times 192.7 \times 10^3 \times 275}{1.0} \times 10^{-6} = 106 \text{ kNm}$$

For two web cover plates

$N_{pl,Rd}$ = 2 × 935 = 1870 kN

$M_{wp,Ed}$ = 37.8 + 16.6 = 54.2 kNm — Sheets 3 & 5

$N_{wp,Ed}$ = 63.9 kN

$$\frac{N_{wp,Ed}}{N_{wp,Rd}} + \frac{M_{wp,Ed}}{M_{c,wp,Rd}} = \frac{63.9}{1870} + \frac{54.2}{106} = 0.55 < 1.0$$

Therefore, the bending resistance of the web cover plates is adequate.

APPENDIX E WORKED EXAMPLE – BASE PLATE CONNECTION

One worked example is presented in this Appendix:

Example E.1 Base plate connection to UKC column section

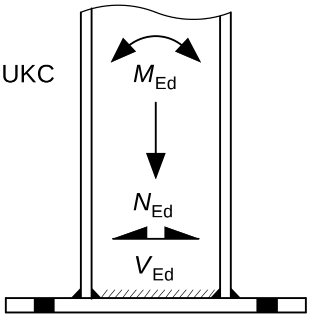
The example follows the recommendations in the main text. Additionally, references to the relevant clauses, Figures and Tables in BS EN 1993-1-8 and its UK National Annex are given where appropriate; these are given simply as the clause, Figure or Table number. References to clauses etc. in other standards are given in full. References to Tables or Figures in the main text are noted accordingly; references to STEPS are to those in Section 5 of the main text.

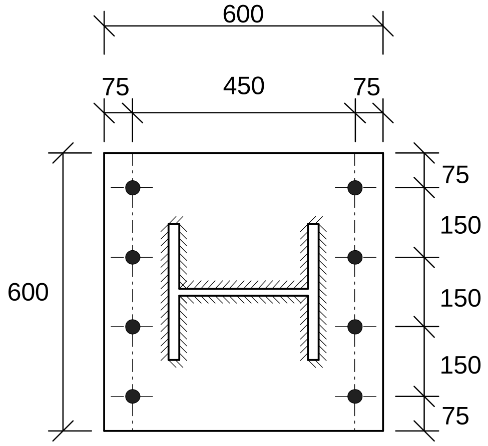
SCI Steel Knowledge	Job No. *CDS 324*		Sheet *1* of *8*
CALCULATION SHEET	Title *Example E.1 – Unstiffened column base plate*		
BCSA	Client		
	Calcs by *DCI*	Checked by *DGB*	Date *Nov 2012*

JOINT CONFIGURATION AND DIMENSIONS

Verify the resistance of the unstiffened column base plate shown below.

305 x 305 x 118 UKC





DESIGN VALUES OF FORCES AT ULS

	Situation 1	**Situation 2**
M_{Ed}	350 kNm	350 kNm
N_{Ed}	−2000 kN (compression)	−350 kN (compression)
V_{Ed}	75 kN	75 kN

Sign convention is:

Force: tension positive

Moment: clockwise positive (in above elevation)

Note: the example considers a design moment in only one direction and for such a situation the base could be asymmetric. However, a symmetric arrangement is chosen, although the requirements for welding on the two flanges are considered separately.

References to clauses, etc. are to BS EN 1993-1-8: and its UK NA, unless otherwise stated.

DIMENSIONS AND SECTION PROPERTIES

Column

From data tables for 305 × 305 × 118 UKC in S355 — P363

Depth	h_c	= 314.5 mm
Width	b_c	= 307.4 mm
Flange thickness	$t_{f,c}$	= 18.7 mm
Web thickness	$t_{w,c}$	= 12.0 mm
Root radius	r_c	= 15.2 mm
Elastic modulus (*y-y axis*)	$W_{el,y,c}$	= 1760000 mm^3
Plastic modulus (*y-y axis*)	$W_{pl,y,c}$	= 1960000 mm^3
Area of cross section	A_c	= 15000 mm^2

Depth between flanges $h_{w,c}$ $= h_c - 2\,t_{f,c}$

$= 314.5 - (2 \times 18.7) = 277.1$ mm

Base plate

Steel grade S275

Depth	h_{bp}	= 600 mm
Gross width	$b_{g,bp}$	= 600 mm
Thickness	t_{bp}	= 50 mm

Concrete

The concrete grade used for the base is C30/37

Bolts

M24 8.8 bolts

Diameter of bolt shank	d	= 24 mm
Diameter of hole	d_0	= 26 mm
Shear area (per bolt)	A_s	= 353 mm^2
Number of bolts either side	n	= 4

Title Example E.1 – Unstiffened column base plate	Sheet 3 of 8

MATERIAL STRENGTHS

Column and base plate

The National Annex to BS EN 1993-1-1 refers to BS EN 10025-2 for values of nominal yield and ultimate strength. When ranges are given the lowest value should be adopted. — BS EN 1993-1-1, NA.2.4

For S355 steel and 16 < $t_{f,c}$ < 40 mm — BS EN 10025-2 Table 7

Column yield strength	$f_{y,c}$	$= R_{eH}$	= 345 N/mm^2
Column ultimate strength	$f_{u,c}$	$= R_m$	= 470 N/mm^2

For S275 steel and 40 < t_{bp} < 63 mm

Base plate yield strength	$f_{y,bp}$	$= R_{eH}$	= 255 N/mm^2
Base plate ultimate strength	$f_{u,bp}$	$= R_m$	= 410 N/mm^2

Concrete

For concrete grade C30/37

Characteristic cylinder strength f_{ck} = 30 MPa = 30 N/mm^2 — BS EN 1992-1-1 Table 3.1

The design compressive strength of the concrete is determined from: — BS EN 1992-1-1, 3.1.6(1)

$$f_{cd} = \frac{\alpha_{cc} f_{ck}}{\gamma_c}$$

Where: — BS EN 1992-1-1, Table NA.1

α_{cc} = 0.85 (conservative, according to the NA)

γ_c = 1.5 (for the persistent and transient design situation)

Thus, — BS EN 1992-1-1, 3.1.6(1)

$$f_{cd} = \frac{0.85 \times 30}{1.5} = 17 \text{ N/mm}^2$$

For typical proportions of foundations (see the requirements of STEP 2), conservatively assume: — STEP 2

$f_{jd} = f_{cd}$ = 17 N/mm^2

Bolts

For 8.8 bolts

Nominal yield strength	f_{yb}	= 640 N/mm^2	Table 3.1
Nominal ultimate strength	f_{ub}	= 800 N/mm^2	

PARTIAL FACTORS FOR RESISTANCE

Structural steel

BS EN 1993-1-1 NA.2.15

γ_{M0} = 1.0

γ_{M1} = 1.0

γ_{M2} = 1.1

Parts in connections

Table NA.1

γ_{M2} = 1.25 (bolts, welds, plates in bearing)

DISTRIBUTION OF FORCES AT THE COLUMN BASE

STEP 1

The design moment resistance of the base plate depends on the resistances of two T-stubs, one for each flange of the column. Whether each T-stub is in tension or compression depends on the magnitudes of the axial force and bending moment. The design forces for each situation are therefore determined first.

Forces in column flanges

Forces at flange centroids, due to moment (situations 1 and 2):

$$N_{L,f} = \frac{M_{Ed}}{(h - t_f)} = \frac{350}{(314.5 - 18.7)} \times 10^3 = 1182 \text{ kN (tension)}$$

$$N_{R,f} = -\frac{M_{Ed}}{(h - t_f)} = -\frac{350}{(314.5 - 18.7)} \times 10^3 = -1182 \text{ (compression)}$$

Forces due to axial force:

Situation 1:

$N_{L,f} = N_{R,f} = N_{Ed}/2 = -2000/2 = -1000$ kN

Situation 2:

$N_{L,f} = N_{R,f} = N_{Ed}/2 = -350/2 = -175$ kN

Total force, situation 1:

$N_{L,f} = 1182 - 1000 = 182$ kN (tension)

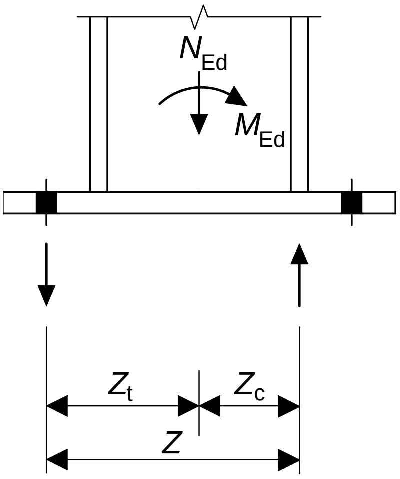
$N_{R,f} = -1182 - 1000 = -2182$ kN (compression)

Total force, situation 2:

$N_{L,f} = 1182 - 175 = 1007$ kN (tension)

$N_{R,f} = -1182 - 175 = -1357$ kN (compression)

In both cases, the left side is in tension and the right side is in compression. The resistances of the two T-stubs will therefore be centred along the lines shown below:



Forces in T-stubs of base plate

Assuming that tension is resisted on the line of the bolts and that compression is resisted concentrically under the flange in compression, the lever arms from the column centre can be calculated as follows:

$z_t = 450/2 = 225$ mm

$z_c = (314.5 - 18.7)/2 = 148$ mm

For both design situations, the left flange is in tension and the right in compression.

Therefore, $z_L = z_t$ and $z_R = z_c$

Title	Example E.1 – Unstiffened column base plate	Sheet 5 of 8

For situation 1, which gives the greater value of design compression force:

$$N_{L,T} = \frac{M_{Ed}}{(z_L + z_R)} + \frac{N_{Ed} \times z_R}{(z_L + z_R)} = \frac{350}{(225+148)} \times 10^3 + \frac{-2000 \times 148}{(225+148)} = 144 \text{ kN}$$

$$N_{R,T} = -\frac{M_{Ed}}{(z_L + z_R)} + \frac{N_{Ed} \times z_L}{(z_L + z_R)} = -\frac{350}{(225+148)} \times 10^3 + \frac{-2000 \times 225}{(225+148)} = -2144 \text{ kN}$$

For situation 2, which gives the greater value of tension on the left:

$$N_{L,T} = \frac{M_{Ed}}{(z_L + z_R)} + \frac{N_{Ed} \times z_R}{(z_L + z_R)} = \frac{350}{(225+148)} \times 10^3 + \frac{-350 \times 148}{(225+148)} = 799 \text{ kN}$$

$$N_{R,T} = -\frac{M_{Ed}}{(z_L + z_R)} + \frac{N_{Ed} \times z_L}{(z_L + z_R)} = -\frac{350}{(225+148)} \times 10^3 + \frac{-350 \times 225}{(225+148)} = -1149 \text{ kN}$$

RESISTANCE OF EQUIVALENT T-STUBS

RESISTANCE OF COMPRESSION T-STUB

STEP 2

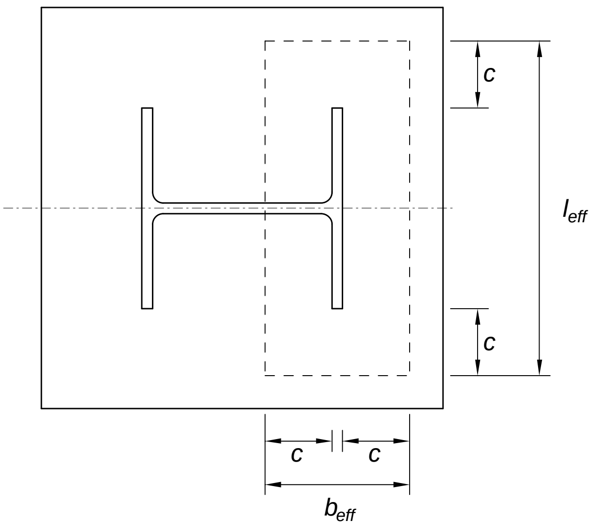
The resistance of a T-stub in compression is the lesser of:

The resistance of concrete in compression under the flange ($F_{c,pl,Rd}$)

The resistance of the column flange and web in compression ($F_{c,fc,Rd}$)

Compressive resistance of concrete below column flange

The effective bearing area of the joint depends on the additional bearing width, as shown below.



Determine the available additional bearing width (c), which depends on the plate thickness, plate strength and joint strength.

Eq(6.5)

$$c = t_{bp}\sqrt{\frac{f_{y,bp}}{3 f_{jd} \gamma_{M0}}}$$

$$= 50 \times \sqrt{\frac{255}{3 \times 17 \times 1.0}} = 112 \text{ mm}$$

The presence of welds is neglected

Thus the dimensions of the bearing area are,

b_{eff} = $2c + t_{fc}$ = $2 \times 112 + 18.7$ = 243 mm

l_{eff} = $2c + b_c$ = $2 \times 112 + 307.4$ = 531 mm

Area of bearing is,

A_{eff} = 531 × 243 = 129000 mm^2

Thus, the compression resistance of the foundation is,

$F_{c,pl,Rd} = A_{eff} f_{jd}$

= 129000 × 17 × 10^{-3} = 2193 kN, > $N_{R,T}$ = 2144 kN (maximum value, situation 1) — Sheet 5

Satisfactory

Resistance of the column flange and web in compression

The resistance of the column flange and web in compression is determined from: — 6.2.6.7(1) Eq.(6.21)

$$F_{c,fc,Rd} = \frac{M_{c,Rd}}{h_c - t_{f,c}}$$

$M_{c,Rd}$ is the design bending resistance of the column cross section

If $V_{Ed} > \frac{V_{c,Rd}}{2}$, the effect of shear should be allowed for.

$V_{c,Rd}$ = 856 kN — P363

V_{Ed} = 75 kN

By inspection:

$$V_{Ed} < \frac{V_{c,Rd}}{2}$$

Therefore, the effects of shear may be neglected and hence — P363

$M_{c,Rd}$ = 675 kNm

Therefore,

$$F_{c,fc,Rd} = \frac{675 \times 10^6}{(314.5 - 18.7)} \times 10^{-3} = 2282 \text{ kN}$$ — 6.2.6.7(1) Eq.(6.21

As, $F_{c,pl,Rd} < F_{c,fc,Rd}$, the compression resistance of the right hand T-stub is:

$F_{c,R,Rd}$ = 2282 kN

$F_{c,R,Rd} > N_{R,T}$ = 2144 kN (maximum value, situation 1) Satisfactory — Sheet 5

RESISTANCE OF TENSION T-STUB

The resistance of the T-stub in tension is the lesser of: — 6.2.8.3(2)

The base plate in bending under the left column flange, and

The column flange/web in tension.

Resistance of base plate in bending

The design resistance of the tension T-stub is given by: — 6.2.8.3, 6.2.6.11, 6.2.5

$$F_{t,pl,Rd} = F_{T,Rd} = \min\{F_{T,1\text{-}2,Rd}; F_{T,3,Rd}\}$$

Where $F_{T,1\text{-}2,Rd}$ is the 'Mode 1 / Mode 2' resistance in the absence of prying and $F_{T,3,Rd}$ is the Mode 3 resistance (bolt failure) — 6.2.4.1(7)

$$F_{T,1\text{-}2,Rd} = \frac{2M_{pl,1,Rd}}{m}$$ — Table 6.2

Title	*Example E.1 – Unstiffened column base plate*	*Sheet 7 of 8*

$$M_{\text{pl,1,Rd}} = \frac{0.25 \sum \ell_{\text{eff,1}} t_{\text{bp}}^{2} f_{\text{y,bp}}}{\gamma_{\text{M0}}}$$ Table 6.2

$\sum \ell_{\text{eff,1}}$ is the effective length of the T-stub, which is determined from Table 6.6.

Since there are four bolts in the row, the effective lengths are given by Table 5.3 in STEP 3.

In most cases, the effective length of T-stub can be judged by inspection to be that for a simple yield line across the width of the base plate but for illustration, the lengths for all possible yield line patterns are evaluated below.

$\ell_{\text{eff,1}}$ is the smallest of the following lengths (in which the number of bolts has been taken as n = 4):

Circular patterns:

$\ell_{\text{eff,cp}} = 2(2\pi m_{\text{x}})$

$\ell_{\text{eff,cp}} = 2(\pi m_{\text{x}} + 2e)$

Non-circular patterns:

$\ell_{\text{eff,nc}} = \frac{b_{\text{bp}}}{2}$

$\ell_{\text{eff,nc}} = 8m_{\text{x}} + 2.5e_{\text{x}}$

$\ell_{\text{eff,nc}} = 6m_{\text{x}} + e + 1.875e_{\text{x}}$

$\ell_{\text{eff,nc}} = 2m_{\text{x}} + 0.625e_{\text{x}} + 1.5w$;

In which m_{x} is as defined in Figure 6.10 and w is the gauge between the outermost bolts Figure 6.10

Evaluating each of the above gives:

$2(2\pi m_{\text{x}})$	$= 2 \times (2 \times \pi \times 60)$	= 754 mm
$2(\pi m_{\text{x}} + 2e)$	$= 2 \times (\pi \times 60 + 2 \times 75)$	= 677 mm
$0.5b_{\text{bp}}$	$= 0.5 \times 600$	= 300 mm
$8m_{\text{x}} + 2.5e_{\text{x}}$	$= (8 \times 60) + (2.5 \times 75)$	= 668 mm
$6m_{\text{x}} + e + 1.875e_{\text{x}}$	$= (6 \times 60) + 75 + (1.875 \times 75)$	= 576 mm
$2m_{\text{x}} + 0.625e_{\text{x}} + 1.5w$	$= (2 \times 60) + (0.625 \times 75) + (1.5 \times 150)$	= 392 mm

As expected, the minimum value is:

$\ell_{\text{eff,1}}$ = 300 mm

Therefore,

$$M_{\text{pl,1,Rd}} = \frac{0.25 \times 300 \times 50^2 \times 255}{1.0} \times 10^{-6} = 47.8 \text{ kNm}$$

$$F_{\text{T,1-2,Rd}} = \frac{2 \times 47.8}{60} \times 10^3 = 1593 \text{ kN}$$

$F_{\text{T,3,Rd}} = \sum F_{\text{t,Rd}}$ Table 6.2

For class 8.8. M24 bolts

$F_{\text{t,Rd}}$ =203 kN P363

$F_{\text{T,3}}$ = 4 × 203 = 812 kN

Title	Example E.1 – Unstiffened column base plate	Sheet 8 of 8

Hence the tension resistance of the T-stub is:

$F_{t,pl,Rd} = F_{T,Rd} = 812$ kN

$F_{t,pl,Rd} > N_{L,T} = 799$ kN (maximum value, situation 2) Satisfactory — Sheet 5

WELD DESIGN

STEP 5

WELDS TO THE TENSION FLANGE

The maximum tensile design force is significantly less than the resistance of the flange, so a full strength weld is not required.

The design force for the weld may be taken as that determined between column and base plate in STEP 1, i.e. 1182 kN ($N_{L,f}$ for situation 2) — Sheet 4

For a fillet weld with s = 12 mm, a = 8.4 mm

The design resistance due to transverse force is:

$$F_{nw,Rd} = K\frac{af_u/\sqrt{3}}{\beta_w \gamma_{M2}}$$

where K = 1.225, f_u = 410 N/mm^2 and β_w = 0.85 (using the properties of the material with the lower strength grade – the base plate)

$$F_{nw,Rd} = 1.225\frac{8.4 \times 410/\sqrt{3}}{0.85 \times 1.25} = 2.29 \text{ kN/mm}$$

Length of weld, assuming a fillet weld all round the flange:

For simplicity, two weld runs will be assumed, along each face of the column flange. Conservatively, the thickness of the web will be deducted from the weld inside the flange. An allowance equal to the leg length will be deducted from each end of each weld run.

L = 307.4 – 2 × 12 + 307.4 – 12 – 4 × 12 = 531 mm

$F_{t,weld,Rd}$ = 2.29 × 531 = 1216 kN, > 1182 kN - Satisfactory

WELDS TO THE COMPRESSION FLANGE

With a sawn end to the column, the compression force may be assumed to be transferred in bearing.

There is no design situation with moment in the opposite direction, so there should be no tension in the right hand flange. Only a nominal weld is required.

Commonly, both flanges would have the same size weld.

WELDS TO THE WEB

Although the web weld could be smaller, sufficient to resist the design shear, it would generally be convenient to continue the flange welds around the entire perimeter of the column.

APPENDIX F WORKED EXAMPLE – WELDED BEAM TO COLUMN CONNECTION

One worked example is presented in this Appendix:

Example F.1 Welded connection between UKB beam and UKC column sections

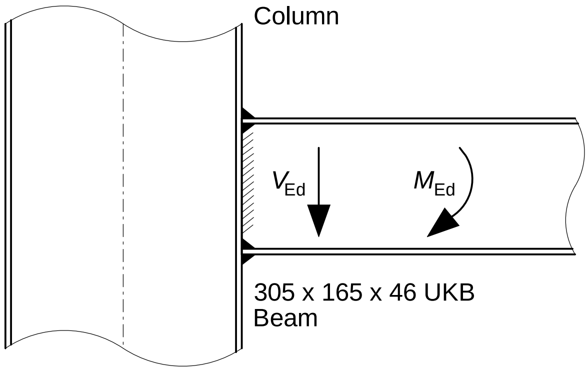
The example follows the recommendations in the main text. Additionally, references to the relevant clauses, Figures and Tables in BS EN 1993-1-8 and its UK National Annex are given where appropriate; these are given simply as the clause, Figure or Table number. References to clauses etc. in other standards are given in full. References to Tables or Figures in the main text are noted accordingly; references to STEPS are to those in Section 3 of the main text.

CALCULATION SHEET	Job No. *CDS 324*		Sheet *1* of *9*
	Title *Example F.1 – Welded beam to column connection*		
	Client		
	Calcs by *MEB*	Checked by *DGB*	Date *Nov 2012*

JOINT CONFIGURATION AND DIMENSIONS

Verify the resistance of the welded beam to column connection shown below. The column flanges are restrained in position by other steelwork, not shown.

References to clauses, etc. are to BS EN 1993-1-8: and its UK NA, unless otherwise stated.



DIMENSIONS AND SECTION PROPERTIES

Column

From data tables for 533 × 210 × 82 UKB in S275 — P363

Depth	h_c	= 528.3 mm	P363
Width	b_c	= 208.8 mm	
Flange thickness	t_{fc}	= 13.2 mm	
Web thickness	t_{wc}	= 9.6 mm	
Root radius	r_c	= 12.7 mm	
Elastic modulus (*y-y axis*)	$W_{el,y,c}$	= 1800000 mm^3	
Plastic modulus (*y-y axis*)	$W_{pl,y,c}$	= 2060000 mm^3	
Area of cross section	A_c	= 10500 mm^2	
Depth between flanges	h_{wc}	$= h_c - 2\,t_{fc}$	
		= 528.3 – (2 × 13.2) = 501.9 mm	

Beam

From data tables for 305 × 165 × 46 UKB in S275

Depth	h_b	= 306.6 mm	P363
Width	b_b	= 165.7 mm	
Flange thickness	t_{fb}	= 11.8 mm	
Web thickness	t_{wb}	= 6.7 mm	
Root radius	r_b	= 8.9 mm	
Elastic modulus (*y-y axis*)	$W_{el,y,b}$	= 64640000 mm^3	
Plastic modulus (*y-y axis*)	$W_{pl,y,b}$	= 720000 mm^3	
Area of cross section	A_b	= 5870 mm^2	

Title	Example F.1 – Welded beam to column connection	Sheet 2 of 9

MATERIAL STRENGTHS

Column and beam

The National Annex to BS EN 1993-1-1 refers to BS EN 10025-2 for values of nominal yield and ultimate strength. When ranges are given the lowest value should be adopted. (BS EN 1993-1-1 NA.2.4)

For S275 steel and 16 < t_{fc} < 40 mm (BS EN 10025-2 Table 7)

Column yield strength $f_{y,c}$ = R_{eH} = 275 N/mm^2

Column ultimate strength $f_{u,c}$ = R_m = 410 N/mm^2

and

Beam yield strength $f_{y,b}$ = R_{eH} = 275 N/mm^2

Beam ultimate strength $f_{u,b}$ = R_m = 410 N/mm^2

PARTIAL FACTORS FOR RESISTANCE

Structural steel

(BS EN 1993-1-1 NA.2.15)

γ_{M0} = 1.0

γ_{M1} = 1.0

γ_{M2} = 1.1

Parts in connections

(Table NA.1)

γ_{M2} = 1.25 (bolts, welds, plates in bearing)

DESIGN VALUES OF FORCES AT ULS

Bending moment M_{Ed} = 170 kNm

Vertical shear force V_{Ed} = 57 kN

The force in the compression flange due to bending is given by: (6.2.6.7)

$$F_{c,Ed} = \frac{M_{Ed}}{(h_b - t_{fb})} = \frac{170 \times 10^3}{306.6 - 11.8} = 577 \text{ kN}$$

The force in the tension flange due to bending is taken as the same value:

$F_{t,Ed}$ = $F_{c,Ed}$ = 577 kN

RESISTANCE OF UNSTIFFENED CONNECTION

TENSION ZONE

Column stiffeners are not required if the effective width of the beam flange, governed by bending of the column flange, is adequate to carry the design force. (STEP 2)

Effective width of beam flange

The effective width of the beam must satisfy: (4.10(3) Based on Eq.(4.7))

$$b_{eff} \geq \left(\frac{f_{y,b}}{f_{u,b}}\right) b_b$$

Note: Reference to plate strengths and thickness in clause 4.10 are taken to mean the values for the beam flange in a welded connection.

b_{eff} = $t_{wc} + 2s + 7kt_{fc}$ (4.10(2))

s = r_c (for a rolled section)

Title	Example F.1 – Welded beam to column connection	Sheet 3 of 9

$k = \left(\frac{t_{fc}}{t_{fb}}\right)\left(\frac{f_{y,c}}{f_{y,b}}\right)$ but $k \leq 1$

$k = \left(\frac{13.2}{11.8}\right)\times\left(\frac{275}{275}\right) = 1.12$

1.12 > 1

Therefore,

$k = 1.0$

$b_{eff} = 9.6 + (2\times 12.7) + (7\times 1\times 13.2) = 127$ mm

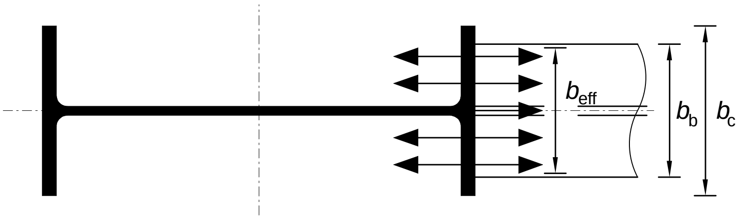
$\left(\frac{f_{y,b}}{f_{u,b}}\right)b_b = \left(\frac{275}{410}\right)\times 165.7 = 111$ mm

As 127 mm > 111 mm, the effective width is adequate.

Resistance of effective width of beam flange

The resistance of the unstiffened column flange is given by:

$F_{fc,Rd} = \frac{b_{eff,b,fc}\, t_{fb}\, f_{y,b}}{\gamma_{M0}}$ 6.2.6.4.3(1)



$b_{eff,b,fc} = b_{eff} = 127$ mm 4.10(2)

$F_{fc,Rd} = \frac{127\times 11.8\times 275\times 10^{-3}}{1.0} = 412$ kN

$F_{t,Ed} = 577$ kN

As 577 kN > 412 kN, tension stiffeners are required.

Column web in tension

Note: since stiffeners are required to strengthen the tension flange, this step could be omitted, but it is given here for completeness.

For an unstiffened column web

$F_{t,wc,Rd} = \frac{\omega b_{eff,t,wc} t_{wc} f_{y,wc}}{\gamma_{M0}}$ 6.2.6.3(1)

$b_{eff,t,wc} = t_{fb} + 2\sqrt{2}a_b + 5(t_{fc} + s)$ 6.2.6.3(2)

$= t_{fb} + 2s_f + 5(t_{fc} + s)$

s = r_c (for rolled sections)

a_b is the effective throat thickness of the flange weld

Assuming a 10 mm leg length weld s_{bf}= 10 mm

$b_{eff,t,wc} = 11.8 + 2\times 10 + 5\times(13.2 + 12.7) = 161$ mm

As the connection is single sided,

$\beta = 1.0$ Table 5.4

Thus,

$\omega = \omega_1$ — Table 6.3

$$\omega_1 = \frac{1}{\sqrt{1+1.3\left(b_{\text{eff,c,wc}} t_{\text{wc}} / A_{\text{vc}}\right)^2}}$$

where:

$A_{\text{vc}} = A - 2bt_{\text{f}} + t_{\text{f}}\left(t_{\text{w}} + 2r\right)$ but not less than $\eta h_{\text{w}} t_{\text{w}}$ — BS EN 1993-1-1 6.2.6(3)

$$A - 2bt_{\text{f}} + t_{\text{f}}\left(t_{\text{w}} + 2r\right) = 10500 - \left(2 \times 208.8 \times 13.2\right) + 13.2 \times \left(9.6 + 2 \times 12.7\right) = 5450 \text{ mm}^2$$

η = 1.0 (conservatively)

$\eta h_{\text{w}} t_{\text{w}}$ = 1 × 501.9 × 9.6 = 4818 mm^2

As 5450 > 4818

A_{vc} = 5450 mm^2

Therefore,

$$\omega_1 = \frac{1}{\sqrt{1+1.3 \times \left(161 \times 9.6 / 5450\right)^2}} = 0.95$$ — Table 6.3

$$F_{\text{t,wc,Rd}} = \frac{0.95 \times 161 \times 9.6 \times 275 \times 10^{-3}}{\gamma_{\text{M0}}} = 404 \text{ kN}$$

$F_{\text{t,Ed}}$ = 577 kN — Sheet 2

As 577 kN > 405 kN, the column web requires strengthening.

For the design of the stiffened tension zone, see Sheet 6.

COMPRESSION ZONE

Column web in compression

Verify that,

$F_{\text{c,wc,Rd}} \geq F_{\text{c,Ed}}$

$$F_{\text{c,wc,Rd}} = \frac{\omega k_{\text{wc}} b_{\text{eff,c,wc}} t_{\text{wc}} f_{\text{y,wc}}}{\gamma_{\text{M0}}} \text{ but } F_{\text{c,wc,Rd}} < \frac{\omega k_{\text{wc}} \rho b_{\text{eff,c,wc}} t_{\text{wc}} f_{\text{y,wc}}}{\gamma_{\text{M0}}}$$ — 6.2.6.2(1) Eq. (6.9)

Where,

ω = 0.95 (as above)

$b_{\text{eff,c,wc}} = b_{\text{eff,t,wc}}$ = 161 mm

In this example, no information is provided about the axial force and bending moment in the column. Therefore take, k_{wc} = 1.0 — Note to 6.2.6.2(2)

ρ is the reduction factor for plate buckling — 6.2.6.2(1)

If $\bar{\lambda}_{\text{p}} \leq 0.72$ then $\rho = 1.0$

If $\bar{\lambda}_{\text{p}} > 0.72$ then $\rho = \dfrac{\left(\bar{\lambda}_{\text{p}} - 0.2\right)}{\bar{\lambda}_{\text{p}}^{\,2}}$

$$\bar{\lambda}_{\text{p}} = 0.932\sqrt{\frac{b_{\text{eff,c,wc}} d_{\text{wc}} f_{\text{y,wc}}}{E t_{\text{wc}}^2}}$$ — 6.2.6.2(1), Eq (6.13c)

Where:

$$d_{\text{wc}} = h_{\text{c}} - 2\left(t_{\text{fc}} + r_{\text{c}}\right)$$

$$= 528.3 - 2 \times \left(13.2 + 12.7\right) = 476.5 \text{ mm}$$

Title	Example F.1 – Welded beam to column connection	Sheet 5 of 9

E = 210 000 N/mm^2 — BS EN 1993-1-1 3.2.6(1)

Therefore,

$$\bar{\lambda}_p = 0.932 \times \sqrt{\frac{161 \times 476.5 \times 275}{210000 \times 9.6^2}} \times 10^{-3} = 0.97$$

As 0.97 > 0.72

$$\rho = \frac{\left(\bar{\lambda}_p - 0.2\right)}{\bar{\lambda}_p^{\,2}}$$

$$= \frac{(0.97 - 0.2)}{0.97^2} = 0.82$$

$$\frac{\omega k_{wc} b_{eff,c,wc} t_{wc} f_{y,wc}}{\gamma_{M0}} = \frac{0.95 \times 1 \times 161 \times 9.6 \times 275 \times 10^{-3}}{1.0} = 404 \text{ kN}$$

$$\frac{\omega k_{wc} \rho\, b_{eff,c,wc} t_{wc} f_{y,wc}}{\gamma_{M0}} = \frac{0.95 \times 1 \times 0.82 \times 161 \times 9.6 \times 275 \times 10^{-3}}{1.0} = 331 \text{ kN}$$

$F_{c,wc,Rd}$ = min (404 ; 331) = 331 kN

$F_{c,Ed}$ = 577 kN — Sheet 2

As 577 kN > 331 kN, the column web requires compression stiffeners.

For the design of the stiffened compression zone, see Sheet 7.

COLUMN WEB PANEL IN SHEAR — STEP 4

Verify that,

$V_{wp,Rd} \geq F_{c,Ed}$

If, $\frac{d_c}{t_{wc}} \leq 69\varepsilon$ then $V_{wp,Rd} = \frac{0.9 f_{y,wc} A_{vc}}{\sqrt{3}\gamma_{M0}}$ — 6.2.6.1 (1)

$$\frac{d_c}{t_{wc}} = \frac{476.5}{9.6} = 49.64$$

$$\varepsilon = \sqrt{\frac{235}{f_{y,wc}}}$$ — BS EN 1993-1-1, Table 5.2

$$= \sqrt{\frac{235}{275}} = 0.92$$

$69\varepsilon = 69 \times 0.92 = 63.5$

As 49.6 < 63.5, the method given in 6.2.6.1 may be used to determine the shear resistance of the column web panel.

A_{vc} = 5450 mm^2 — Sheet 4

$$V_{wp,Rd} = \frac{0.9 \times 275 \times 5450}{\sqrt{3} \times 1} \times 10^{-3} = 779 \text{ kN}$$

$F_{c,Ed}$ = 577 kN

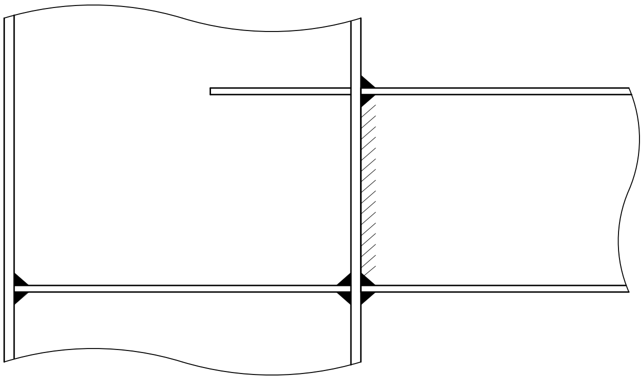
As 779 kN > 577 kN, the resistance of the column web panel in shear is adequate.

Title	Example F.1 – Welded beam to column connection	Sheet 6 of 9

RESISTANCE OF STIFFENED COLUMN

TENSION ZONE

STEP 2



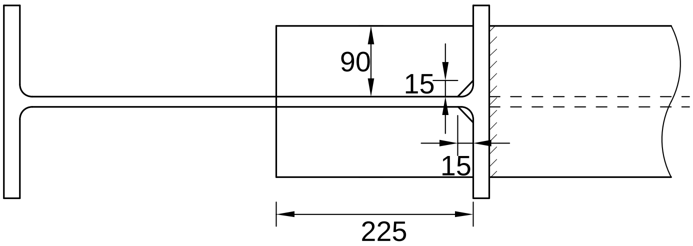
Stiffener size

Try tension stiffeners in S275 with the following dimensions:

b_{sg} = 90 mm

t_s = 15 mm

$f_{y,s}$ = 275 N/mm^2



The area provided by the stiffeners is:

$A_{sn} = 2b_{sn}t_s$

Assuming a 15 mm corner chamfer, b_{sn} = 75 mm

$2 \times b_{sn} \times t_s = 2 \times 75 \times 15 = 2250$ mm^2

Hence the resistance is:

$$F_{t,s,Rd} = \frac{A_{sn}f_{y,s}}{\gamma_{M0}} = \frac{2250 \times 275}{1.0} \times 10^{-3} = 619 \text{ kN}$$

Conservatively, for a welded connection, the design force may be taken as the force in the beam flange.

$F_{t,s,Ed} = F_{t,Ed}$ = 577 kN — Sheet 4

As 619 > 577, the stiffeners are satisfactory

The stiffeners only need to be partial depth. The stiffeners must be of sufficient length to transfer the applied force and to transfer that force to the web of the column.

The minimum length of stiffener is taken as $1.9b_s = 1.9 \times 90 = 171$ mm — STEP 6A in Section 2

Try 175 mm long stiffeners.

With two shear planes, the resistance of the column web, V_{pl} is given by:

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

For rectangular plane sections, the shear area is taken as 0.9 of the gross area

Title	Example F.1 – Welded beam to column connection	Sheet 7 of 9

The shear resistance of the two shear planes is therefore

$$V_{pl} = \frac{A_v f_{y,c}}{\sqrt{3}\gamma_{M0}} = \frac{2 \times 0.9 \times 175 \times 9.6 \times 275}{\sqrt{3} \times 1.0} \times 10^{-3} = 480 \text{ kN}$$

This is insufficient - the stiffeners must be lengthened; try 225 mm

Then

$$V_{pl} = \frac{A_v f_{y,c}}{\sqrt{3}\gamma_{M0}} = \frac{2 \times 0.9 \times 225 \times 9.6 \times 275}{\sqrt{3} \times 1.0} \times 10^{-3} = 617 \text{ kN}$$

617 > 577, so 225 mm long stiffeners are satisfactory

Because the connection is single sided no check of the web at the end of the stiffener is required.

COMPRESSION ZONE

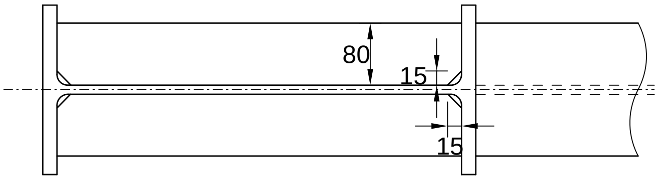
Try a pair of stiffeners in S275 steel with:

Gross width b_{sg} = 80 mm

Thickness t_s = 10 mm

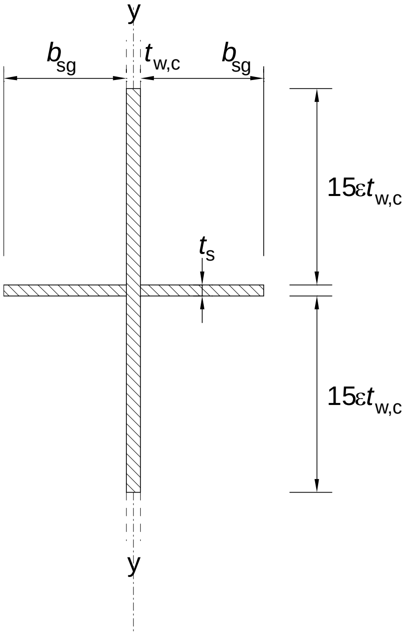
Length h_s $= h_{sc} - 2t_{fc}$

$= 528.3 - (2 \times 13.2)$ = 502 mm



Flexural buckling resistance

Determine the flexural buckling resistance of the cruciform stiffener section shown below



The width of web that may be considered as part of the stiffener section is $15\varepsilon t_{wc}$ either side of the stiffener. BS EN 1993-1-5, 9.1

The width/thickness ratio of the outstand should be limited to prevent torsional buckling but conservatively the Class 3 limit for compression flange outstands may be used. STEP 6B in Section 2

Title Example F.1 – Welded beam to column connection	Sheet 8 of 9
Limiting value of $\frac{c}{t}$ for Class 3 = 14ε	
Here,	
ε = 0.92	Sheet 6
Hence limiting $\frac{c}{t} = 14 \times 0.92 = 12.9$	
Actual ratio $\frac{c}{t_s} = \frac{80}{10} = 8 < 12.9$ OK	
Effective area of stiffener	
$A_{s,eff} = 2A_s + t_{wc}(30\varepsilon t_{wc} + t_s)$ $= (2 \times 80 \times 10) + 9.6 \times (30 \times 0.92 \times 9.6 + 10) = 4240$ mm^2	
The second moment of area of the stiffener section may be conservatively determined as:	
$I_s = \frac{(2b_{sg} + t_{wc})^3 t_s}{12}$ (excluding column web) $= \frac{((2\times 80)+9.6)^3 \times 10}{12} = 4.07 \times 10^6$ mm^4	
The radius of gyration of the stiffener section is given by:	
$i_s = \sqrt{\frac{I_s}{A_{s,eff}}} = \sqrt{\frac{4.07\times 10^6}{4240}} = 31.0$ mm	
Non-dimensional flexural slenderness:	
$\bar{\lambda} = \frac{\ell}{i_s \lambda_1}$	BS EN 1993-1-1, 6.3.1.2
Where $\lambda_1 = 93.9\varepsilon$	
$\ell \geq 0.75h_w$ Therefore, conservatively,	BS EN 1993-1-5, 9.4(2)
$\ell = h_s = 501.9$ mm	
$\bar{\lambda} = \frac{501.9}{31.0 \times 93.9 \times 0.92} = 0.19$	
The reduction factor χ is given by buckling curve c according to the value of $\bar{\lambda}$	BS EN 1993-1-5,9.4
Since $\bar{\lambda} < 0.2$, the buckling effects may be ignored. Only the resistance of the cross section of the stiffener need be considered.	BS EN 1993-1-1 6.3.1.2(4)
Resistance of cross section (crushing resistance)	
$N_{c,Rd} = \frac{A_{s,eff} f_{ys}}{\gamma_{M0}}$	
Where:	
$A_{s,eff} = 4240$ mm^2	Sheet 8
And thus	
$N_{c,Rd} = \frac{4240 \times 275 \times 10^{-3}}{1.0} = 1166$ kN	
$F_{c,Ed} = 577$ kN	Sheet 2
1166 kN > 577 kN	
Therefore the compression resistance of the compression stiffener is adequate.	

Title	*Example F.1 – Welded beam to column connection*	*Sheet 9 of 9*

WELD DESIGN

STEP 5

Beam to column welds

All welds will be designed as full strength

For the beam flange/column weld, the minimum required throat = $t_{fb}/2 = 11.8/2 = 5.9$ mm

A 10 mm leg length weld has a throat a_f = $10/\sqrt{2} = 7.1$ mm – satisfactory.

For the beam web/column weld, the minimum required throat = $t_{fw}/2 = 6.7/2 = 3.4$ mm

A 6 mm leg length weld has a throat a_w = $6/\sqrt{2} = 4.2$ mm – satisfactory.

Tension stiffener welds

For the stiffener to flange weld, the weld will be designed as full strength.

The minimum required throat = $t_{fs}/2 = 15/2 = 7.5$ mm

A 12 mm leg length weld has a throat a_s = $12/\sqrt{2} = 8.5$ mm – satisfactory.

For the stiffener to web welds, the force in each stiffener is $577/2 = 289$ kN

The effective length of weld to the web, for each stiffener, assuming a 6 mm fillet weld = 2 × (225 – 15 – 2× 6) = 396 mm

Force in the weld = $289/396 = 0.73$ kN/mm

A longitudinal 6 mm fillet weld provides 0.94 kN/mm – satisfactory. Reference 7

Compression stiffener welds

The stiffener will be fitted, so 6 mm fillet welds all round will be satisfactory.

APPENDIX G ALPHA CHART

The alpha chart given in BS EN 1993-1-8 Figure 6.1 which gives the values of α, dependent on λ_1 and λ_2, is very closely approximated by the following mathematical expressions.

For a given value of α, within the range α = 8 to α = 4.45, the straight part of the curve occurs at a value of λ_1 given by:

$$\lambda_{1,\text{lim}} = \frac{1.25}{(\alpha - 2.75)}$$

The lowest value of λ_2 on this straight part of the curve is given by:

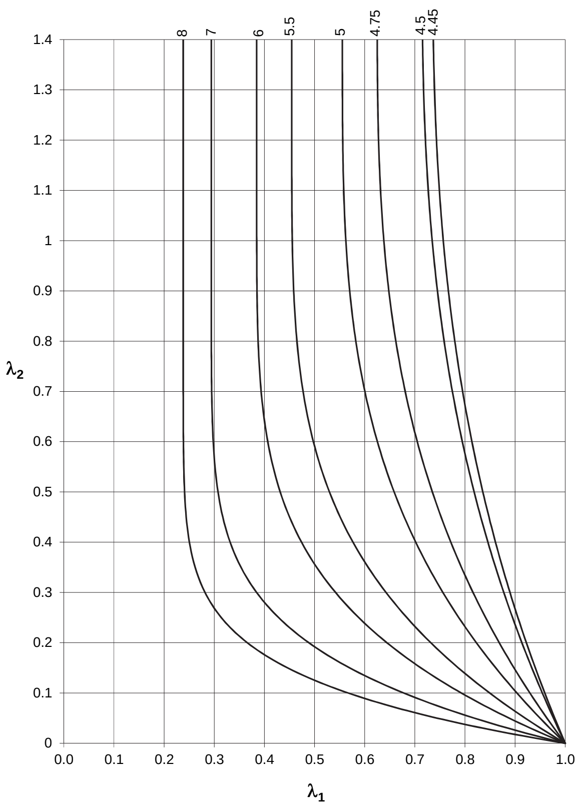
$$\lambda_{2,\text{lim}} = \frac{\alpha \lambda_{1,\text{lim}}}{2}$$

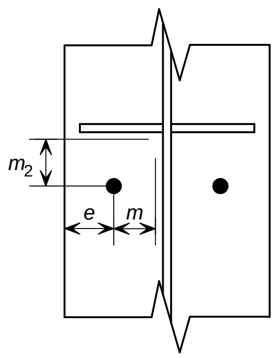
Although, when plotted, the graph for a particular value α may appear to be straight, down to a lower value of λ_2, it is actually only very close to the line of constant λ_1.

Below this limiting value of λ_2, the value of λ_1 is given by:

$$\lambda_1 = \lambda_{1,\text{lim}} + (1 - \lambda_{1,\text{lim}})\left(\frac{(\lambda_{2,\text{lim}} - \lambda_2)}{\lambda_{2,\text{lim}}}\right)^{0.185\alpha^{1.785}}$$

Curves produced using the above expressions are given below. Comparison with those in Figure 6.1 of the Standard will show close agreement.





$$\lambda_1 = \frac{m}{m+e}$$

$$\lambda_1 = \frac{m_2}{m+e}$$

m, m_2 and *e* are defined in Section 2.

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