Joints in Steel Construction
Moment Connections

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FOREWORD

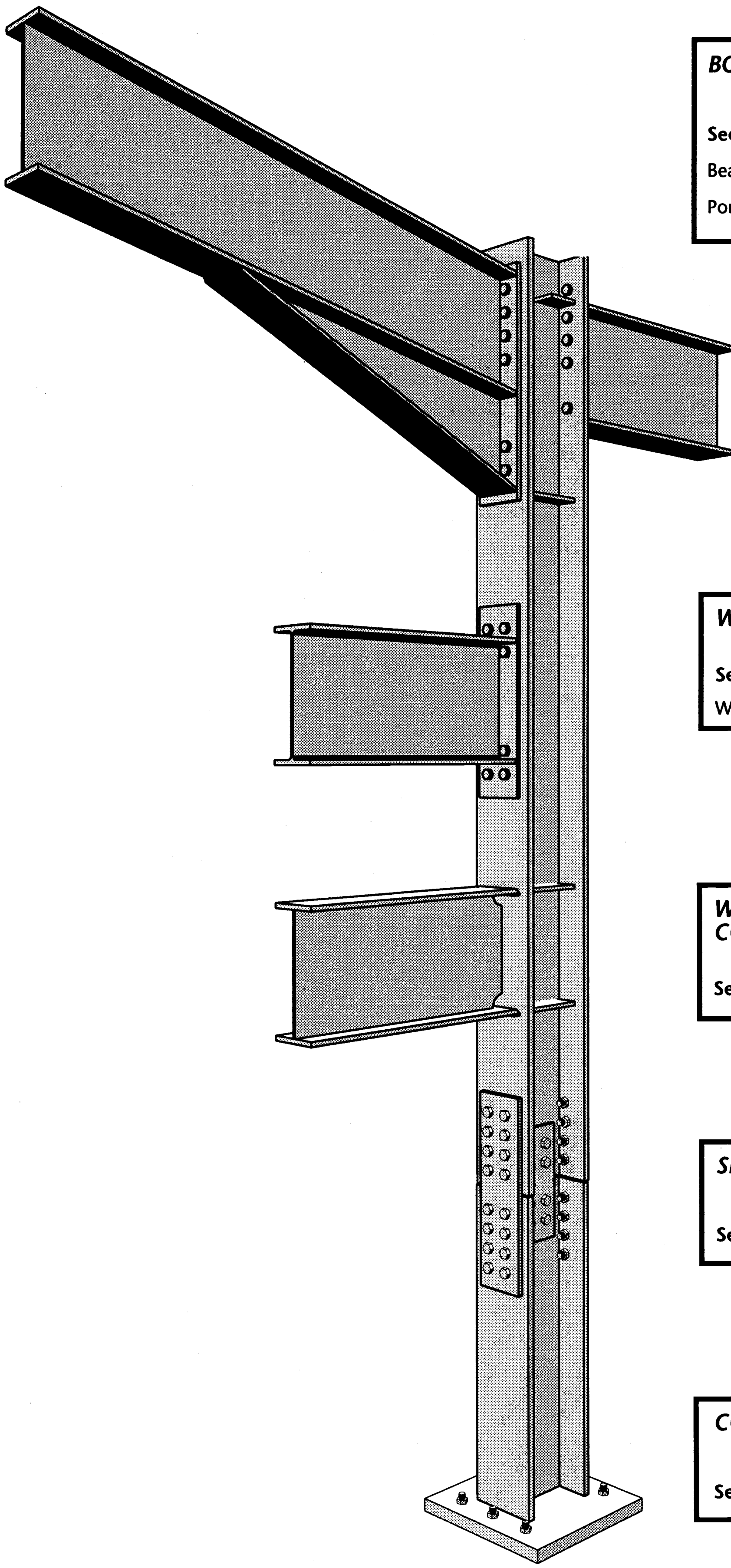
This publication is the third in a series of books which cover the range of structural steelwork connections. It provides a guide to the design of Moment Connections in Steelwork. The other books in the series are *Joints in Simple Construction, Volumes 1 and 2.*

Included in this guide are both bolted and welded connections suitable for use in continuous frame design, together with bolted wind-moment connections, which may be used in semi-continuous design.

The publication is produced by the SCI/BCSA Connections Group with sponsorship from the Building Research Establishment.

The Connections Group was established in 1987 to bring together academics, consultants and steelwork contractors to work on the development of authoritative design guides for structural steelwork connections.

PICTORIAL INDEX



MOMENT CONNECTIONS

CONTENTS

1. INTRODUCTION

1.1 ABOUT THIS DESIGN GUIDE

This publication provides methods for designing the following types of moment resisting connections in steel-framed structures:

Beam to column

- Bolted end plates
- Wind-moment connections
- Shop and site-welded connections

Beams

- Bolted splices
- Welded splices

Columns

- Bolted splices
- Welded splices
- Bases

Connections subject to seismic loading are **not** covered in this publication.

Although each Section of this publication describes connections between I-section members bending about their major axes, the general principles can be adapted for use with other section types and configurations.

Design procedures

The capacity checks on bolts, welds and sections are all based on BS 5950: Part 1 (1).

Other features in the design model are taken from a variety of sources. They include established methods used in the UK and overseas. (2 to 8)

Historically, moment connections have been designed for strength only with little regard to other characteristics i.e. stiffness and ductility. There is growing recognition that in certain situations this practice is questionable and so guidance is given to help designers.

Steel grades

Steel grades have been designated with the commonly used BS 5950: Part 1 notation (Amendment No. 1 1992). The equivalent designations in other specifications are given in Table 1.1.

Table 1.1 Steel grades

BS 5950 : Part 1	BS 4360	BS EN 10 025	
		1990	1993
Design Grade 43	Grade 43	Fe 430	S275
Design Grade 50	Grade 50	Fe 510	S355

Capacity tables

Without access to suitable software, designing efficient moment connections can be a long and tedious process. To help overcome this problem, capacity tables for standardised bolted beam to column connections are provided in the yellow pages of this publication.

The capacity tables have been arranged so that the designer can simply select a beam connection and with the minimum of calculation check whether the column it connects to needs to be stiffened.

The tables serve two other useful functions. Firstly, they can be an aid for frame designers to help with member selection, and secondly they can be used to provide a good 'first guess' in those cases where the standard geometry may not be appropriate.

A key aim during the production of the tables was to standardise the selection of bolts and fittings. This process continues the work on connection standardisation which was introduced in *Joints in Simple Construction* (9) and is widely recognised as being an important step towards improving the efficiency of the industry.

Design examples

Worked examples illustrating the design method are included in most Sections, with a further example of a bolted end plate connection in Appendix I. Examples showing use of the capacity tables precede each set of tables.

1.2 CLASSIFICATION OF CONNECTIONS

BS 5950: Part 1 requires that the connections in a steel structure should accord with the assumptions made in the design of the frame. It is not sufficient in all situations to assume that a moment connection is adequate simply because it is capable of resisting the design bending moment, shear and axial forces. It may also be necessary to consider the rotational stiffness and the rotation capacity.

The characteristics of a joint can be best understood by considering its rotation under load. Rotation is the actual change in angle which takes place as shown in Figure 1.1.

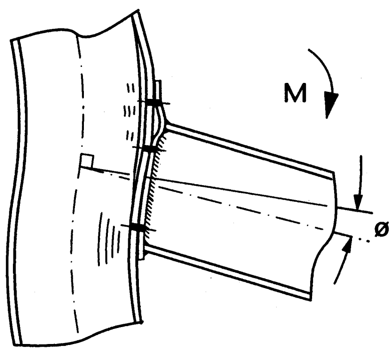


Figure 1.1 Moment - rotation of a connection

Connections can be classified in three ways as illustrated in Figure 1.2 on page 4. These are by:

- **Moment Resistance;**
  the connection may be either *full strength, partial strength, or nominally pinned (i.e. not moment resisting),*
- **Rotational Stiffness;**
  the connection may be *rigid, semi-rigid* or *nominally pinned (i.e. no rotational stiffness),*
- **Rotation Capacity;**
  connections may need to be *ductile.* This criterion is less familiar to most designers and introduces the concept that a connection may need to rotate plastically at some stage of the loading cycle without failure. *Joints in simple construction* (9) have to perform this way, and the principle also applies to some moment connections such as those in wind-moment frames which are the subject of Section 3.

Table 1.2 gives guidance on the properties that are needed for connections in frames designed by the more popular methods in use today. Definitions for some of the terms used are given in Section 1.5.

Stiffness and ductility

Calculating the stiffness of any connection is a tortuous process. Annex J of EC3 presents a method for bolted end plates although the evidence is that the results are far from satisfactory. A revised version is expected to be issued in 1995.

To compound the problem, the limits which are set in EC3 for rigid, semi-rigid and simple design are defined in various ways and may change depending on whether or not the frame is braced.

Checking for ductility is just as daunting. Assessing the connection is not an easy process and in principle the rotation capacity needed will depend on the arrangement of loading and whether the frame is braced or unbraced.

For these reasons, it is felt that the most realistic approach is for the designer to follow simple rule-of-thumb guidelines which will in most circumstances ensure that the frame design assumptions have not been invalidated. The use of 8mm and 10mm thick fittings with wide bolt spacing recommended in *Joints in Simple Construction* is an example of this approach.

Guidance to help ensure adequate levels of stiffness and ductility can be found in Section 2.5.

1.3 EXCHANGE OF INFORMATION

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

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

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

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

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

The *National Structural Steelwork Specification for Building Construction* (10) gives guidance on the transfer of information and there will be great benefits if this is observed. The following items should be considered a minimum:

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

Table 1.2 Methods of frame design

DESIGN		CONNECTIONS			NOTES
Type of framing	**Global Analysis**	**Properties**	**Fig 1.2 Example**	**Method**	
Simple	Pin Joints	Nominally Pinned	⑥	*Joints in Simple Construction (Note 2)*	Economic method for braced multi-storey frames. Connection design is made for shear strength only.
Continuous *(Note 1)*	Elastic	Rigid	①②③④	Section 2	Conventional elastic analysis.
	Plastic	Full strength	①②④	Section 2	Plastic hinges form in the adjacent member, **not** in the connections. Popular for portal frame designs.
	Elastic-Plastic	Full strength and Rigid	①②④	Section 2	
Semi-Continuous *(Note 1)*	Elastic	Semi-rigid	⑤⑥	Not Covered	Connections are modelled as rotational springs. Prediction of connection stiffness presents difficulties.
	Plastic	Partial strength and Ductile	⑤⑥	Section 3	Wind-moment design is a variant of this method
	Elastic-Plastic	Partial strength and/or Semi-rigid	Any	Not Covered	Full connection properties are modelled in the analysis. A research tool rather than a practical design method.

Note 1 *BS 5950 refers to these design methods as 'Rigid' and 'Semi-Rigid' respectively but this can be confusing because they encompass properties other than stiffness.*

Note 2 *See reference 9*

1.4 COSTS

Moment connections are inevitably more expensive to fabricate than simple ones, although the degree of extra workmanship can vary enormously.

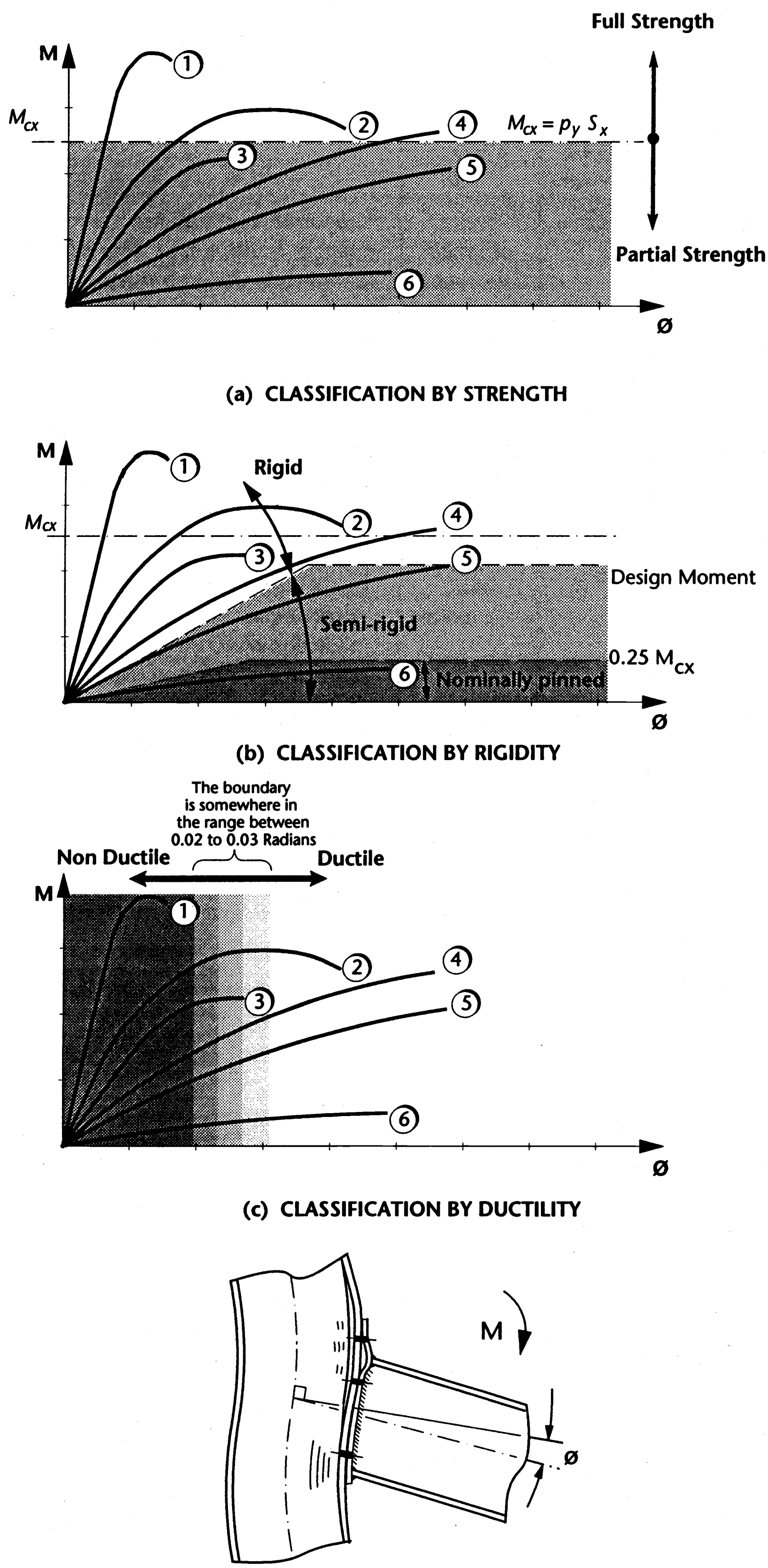
For partial strength connections, such as those in wind-moment frames, the difference can be slight. At the other end of the scale, full strength rigid connections with haunched beams and stiffened columns can be extremely expensive and here the fabrication costs of components can more than double.

For this reason, 'rigid' frame design is not popular in the multistorey building market, although it does have benefits such as permitting longer spans, shallower beams and elevations without bracing.

The single storey portal frame is a special case where the haunch is used to strengthen the rafter, leading to a significant reduction in the frame weight and an overall saving in cost.

Giving specific guidance on costs is difficult, as fabricators' workmanship rates can vary considerably, and are dependent upon the level of investment in plant and machinery. However, the designer's and the detailer's main objective must be to reduce the work content. The material costs for fittings and bolts are small compared with workmanship costs.

The real costs come from the time taken to design the connection, detail it, make the fittings, mark out the geometry, drill the holes and complete the welding and testing. In a fabrication shop, the disruption caused by having to weld one stiffener into a column that would otherwise have a clear passage through the works can be considerable.



NOTES:

The curves show the behaviour of a set of six connections with moment plotted against rotation at the joint. They are classified in three ways:

Strength = moment resistance

①② &④ are full strength. Connections in Capacity Tables on pages 150 - 181 are full strength when $M \geq M_{cx}$.

③& ⑤are partial strength. Connections in Capacity Tables pages 150 - 181 are partial strength when $M \leq M_{cx}$.

Rigidity = rotational stiffness

①②③ &④ are rigid connections.

⑤ is a semi-rigid connection. One interpretation of BS 5950 is that the slope of the dividing line between rigid and semi-rigid should be taken as 2EI/L.

Ductility = rotation capacity

② ④&⑤ are ductile connections.

① is not ductile, while ③ is a borderline case.
The wind-moment connections included in Capacity Tables on pages 205 - 218 are ductile connections.

⑥ is a 'nominally pinned' connection.
The connections found in *Joints in Simple Construction* illustrate this type of joint.

Figure 1.2 Classification of moment connections

When moment connections are used, the designer can minimise costs by adopting the following simple rules:

- Avoid complexity caused by eccentricities and skews – these can cause serious problems.
- Use standard connections wherever possible.
- Rationalise the sections used for fittings, adopting standard flats where possible. In general, design grade 43 steel is preferred because it is more readily available.
- Limit the range of bolt grades and sizes. Fully threaded M24 8.8 bolts should be the first choice for beam sections 400mm deep or greater, and M20 8.8 bolts for shallower beams.
- Use friction grip connections only as a last resort, e.g. where there is a possibility of fatigue or where joint slip is unacceptable.
- Consider increasing the beam depth or column weight to avoid excessive stiffening. Least weight solutions are rarely the most economical.

Most fabricators are happy to give advice about relative costs at the design stage without obligation and this can help to achieve an optimum design.

1.5 DEFINITIONS

Full strength connection

A connection with moment resistance at least equal to that of the member.

Partial strength connection

A connection with moment resistance which is less than that of the member.

Rigid connection

A connection which is stiff enough for the effect of its flexibility on the frame bending moment diagram to be neglected.

Semi-rigid connection

A connection which is too flexible to qualify as rigid but is not a pin.

Nominally pinned connection

A connection which is sufficiently flexible to be regarded as a pin for analysis purposes.

These connections are, by definition, not moment connections although partial strength connections able to resist less than 25% of M_{cx} may be regarded as nominally pinned.

Ductile connection

A connection which has sufficient rotation capacity to act as a plastic hinge.

Connection ductility should not to be confused with ductility of material (elongation to fracture).

Simple design

Method of frame design in which the connections are assumed not to develop moments that adversely affect either the members or the structure as a whole.

Continuous design

Method of frame design in which the connection properties are not modelled in the frame analysis. This covers either elastic analysis where the connections are rigid, or plastic analysis where the connections are full strength.

Semi-continuous design

Method of frame design in which the connection properties have to be modelled in the analysis. This covers elastic analysis where semi-rigid connections are modelled as rotational springs, or plastic analysis where partial strength connections are modelled as plastic hinges.

For ease of description within this manual, moment connections are generally illustrated with tension in the top flange and compression in the bottom flange.

1.6 MAJOR SYMBOLS

Note: Other symbols employed in particular Sections are described where used.

B	Width of section (Subscript c or b refers to column or beam)
b_p	Width of plate
C	Compression force
D	Depth of section (Subscript c or b refers to column or beam)
d	Depth of web between fillets *or* diameter of a bolt
e	End distance
g	Gauge (Transverse distance between bolt centrelines)
M	Bending moment
N	Axial force
P_c	Capacity in compression
P_t'	Enhanced tension capacity of a bolt when prying is considered
p	Bolt spacing ('pitch')
p_y	Design strength of steel
Q	Prying force associated with a bolt
s_w	Fillet weld leg length
S	Plastic modulus
T	Thickness of flange (Subscript c or b refers to column or beam) *or* tension force
t_p	Thickness of plate
t	Thickness of web (Subscript c or b refers to column or beam)
r	Root radius of section
V	Shear Force
Z	Elastic modulus

Lengths and thicknesses stated without units are in millimetres.

2. BOLTED END PLATE CONNECTIONS

2.1 SCOPE

This Section deals with the design of bolted end plate connections such as those shown in Figure 2.1.

Experienced designers will recognise a significant departure from traditional UK practice, since the design model includes a plastic distribution of bolt forces as in Eurocode 3. Although it is more complex, the model can provide a greater moment capacity and research has shown that it gives a more accurate prediction of the actual behaviour of a connection. Safeguards are built in to the method to prevent premature bolt failure.

Three design approaches are described:

(1) The rigorous design method

This method is comprehensive, and some of the steps are complicated. **For this reason the rigorous method is considered to be suitable primarily as a reference, and as a specification for the preparation of computer software.**

Procedures are given for each stage of the design in Section 2.8, and a worked example is given in Appendix I.

(2) Capacity tables

Moment and shear tables are provided for a standardised range of full strength and partial strength connections. A selected range of universal beams is included, connecting to most universal column sections.

Tables for standardised connections for portal frames with inclined rafters are also provided. They are for a typical selection of rafters with universal beams used as columns.

The application of the tables is discussed in Section 2.7. Detailed instructions and examples of their use precede each set of tables.

(3) An abridged method for manual design

Sections 2.9 and 2.10 show how the rigorous method can be abridged to enable quick results to be obtained by hand calculation. This can be useful for unusual connections which do not fit into the standard geometry of the capacity tables.

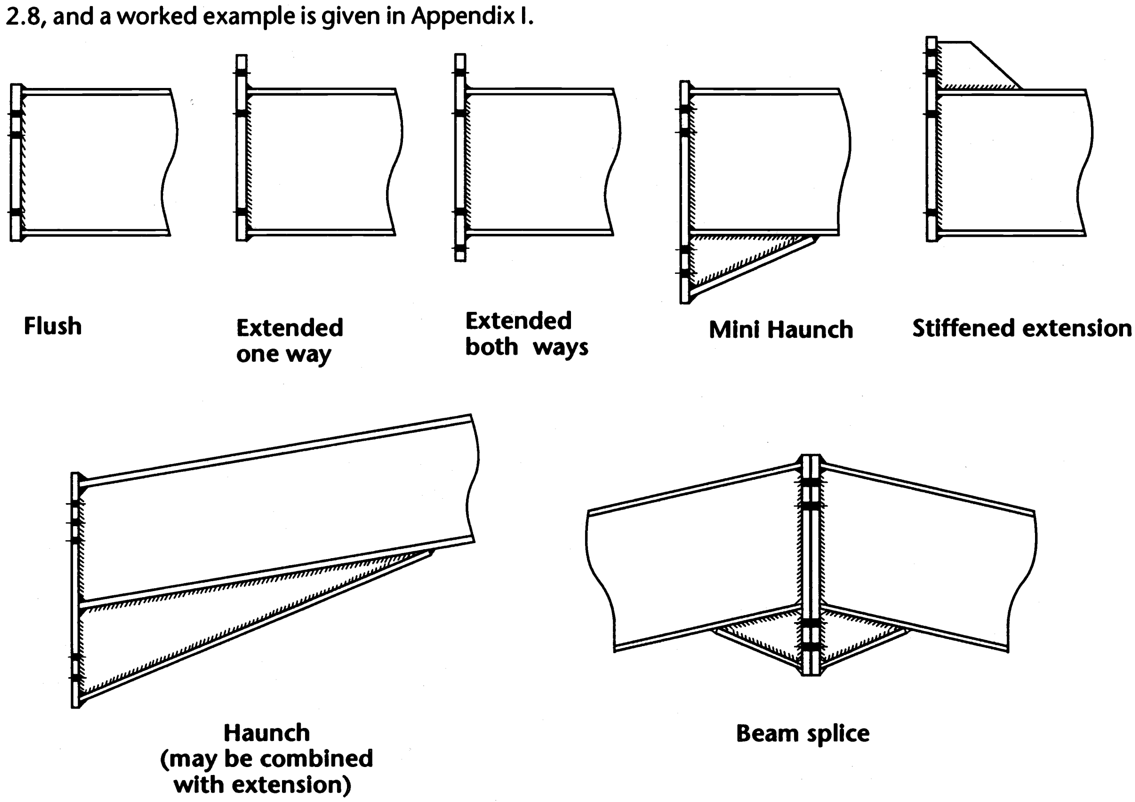


Figure 2.1 Typical end plate connections

2.2 DESIGN PHILOSOPHY

The design model used here is essentially that presented in Annex J of Eurocode 3: Part 1.1. It is based on a plastic distribution of bolt forces. The method is the result of extensive testing in Europe as well as a period of practical use in the Netherlands.

Although the design philosophy is taken directly from EC3, the strength checks on the bolts, welds and steel have been modified to suit BS 5950: Part 1.

Load paths

An end plate connection transmits moment by coupling tension in the bolts with compression at the opposite flange. Unless there is axial force in the beam, the two forces are equal and opposite. (See Figure 2.2.)

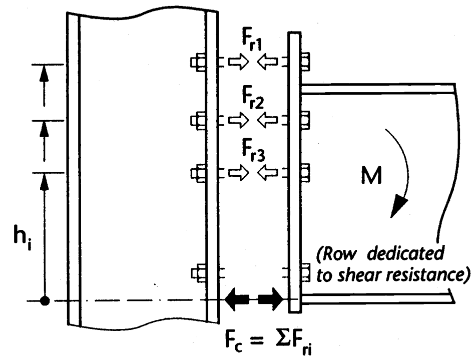


Figure 2.2 Forces in the connection

Tests show that, by the ultimate limit state, rotation has taken place with the centre of rotation at, or near, the compression flange which bears against the column. It is therefore reasonable to consider that compression is concentrated at the level of the centre of the flange.

The bolt row furthest from the compression flange will tend to attract the most tension, and traditional practice has been to assume a triangular distribution of forces. The method adopted here also gives greater priority to the outer bolts, but differs in that it allows a plastic distribution of bolt forces.

The force permitted in any bolt row is based on its potential resistance, and not just on its lever arm. Bolts near a point of stiffness, such as the beam flange or a stiffener, will therefore attract more load.

Rather than arbitrarily allocating force to each bolt row by a linear or 'triangular' distribution, the method considers each side of the connection separately, making a precise allocation based on the capacity of each part.

Surplus force in one row of bolts can be transferred to an adjacent row which has a reserve of capacity. This principle is closer to the way connections actually perform in practice.

A plastic distribution of bolt forces is only reasonable, however, if the necessary deformation can take place. An upper limit is therefore set on the thickness of the column flange, or end plate, relative to the bolt strength. Where this limit is exceeded on both sides of the connection, a modification to the bolt tension forces is made to ensure that they do not exceed a triangular distribution for rows below the beam flange. (This triangular limit to the plastic forces is at present under consideration for inclusion in EC3.)

Figure 2.3 compares the two plastic distributions with a more traditional triangular distribution.

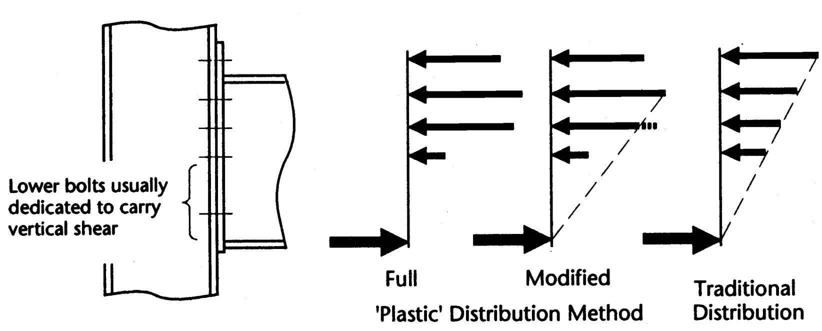
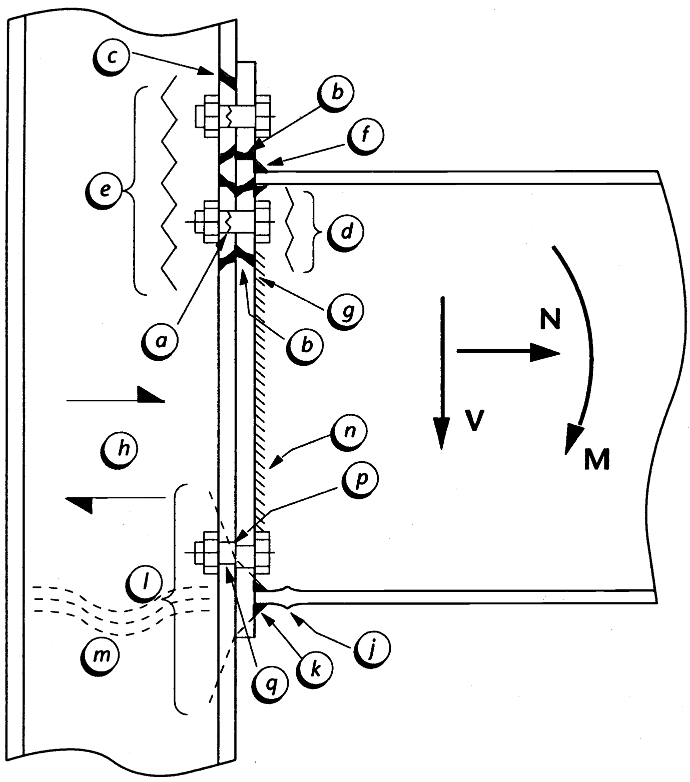


Figure 2.3 Distribution of bolt forces



ZONE	REF	CHECKLIST ITEM	See Procedure
TENSION	a	Bolt tension	STEP 1A
	b	End plate bending	STEP 1A
	c	Column flange bending	STEP 1A
	d	Beam web tension	STEP 1B
	e	Column web tension	STEP 1B
	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 crushing	STEP 2
	m	Column web buckling	STEP 2
VERTICAL SHEAR	n	Web to end plate weld	STEP 7
	p	Bolt shear	STEP 5
	q	Bolt bearing (plate or flange)	STEP 5

Fig 2.4 Component design checks

2.3 CAPACITY CHECKS

There are 15 principal checks to be made on the beam, the column, and on the bolts. These are shown, with a check list, in Figure 2.4.

Each of these checks is outlined in detail in the procedures later in this Section and a flow chart is included which leads the reader through the design process.

2.3.1 Tension zone

The resistance at each bolt row in the tension zone may be limited by:

- column flange bending and bolt strength
- end plate bending and bolt strength
- column web tension
- beam web tension.

For column flange or end plate bending the method uses the Eurocode 3 approach which converts the complex pattern of yield lines which occurs round the bolts into a simple 'equivalent tee-stub' as shown in Figure 2.5. The capacity of the tee-stub is then checked against three possible modes of failure illustrated in Figure 2.6.

One area of difficulty with bolted moment end plates has always been the treatment of the prying force 'Q'. Depending upon the geometry of the connection, this force can vary from 0% to upwards of 40% of the tension in the bolt.

For this reason, simple design methods make a blanket allowance for prying by assuming it is present, and has a value between 20% and 30% of the bolt capacity. This approach is adopted by BS 5950: Part 1 with the values for P_t given in Table 32 of that standard.

The calculations for modes 1, 2 and 3 do not determine 'Q' directly, but prying forces are implicit in the formulae. The enhanced tension capacities which are shown in table 2.1 for 8.8 bolts can therefore be used in the design method.

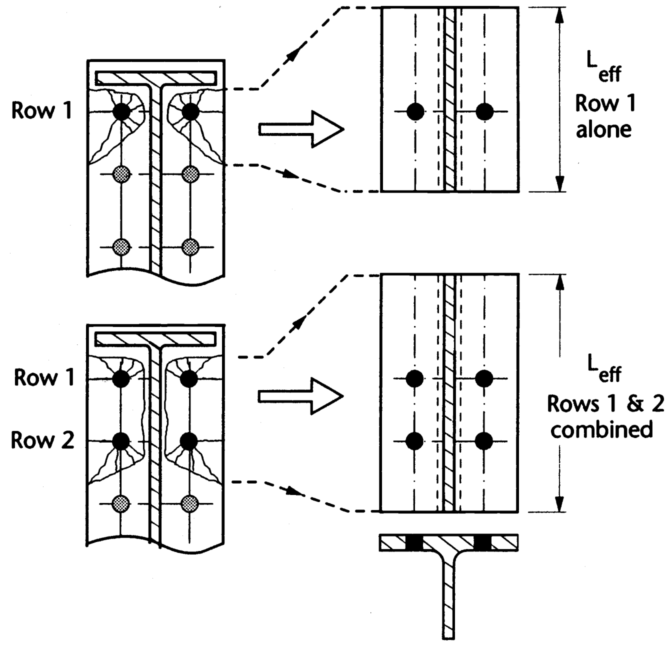


Figure 2.5 Equivalent T-Stubs

Table 2.1 Tensile capacity of a single 8.8 Bolt

Bolt Size	BS 5950: Part 1 Table 32, P_t	Enhanced value appropriate to the method, P_t'
	(450N/mm²)	*(560N/mm²)**
M20	110kN	137kN
M24	159kN	198kN
M30	252kN	314kN

** See Appendix IV*

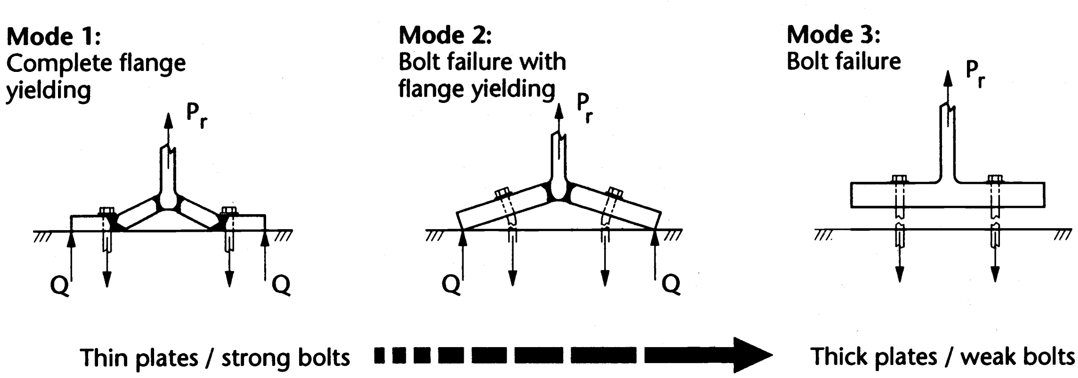


Figure 2.6 Column flange or end plate bending & bolt strength

Distribution of bolt forces

The resistance in each row, (P_{r1}, P_{r2}, P_{r3} ...), is calculated one row at a time, starting at the top and working down. In this way, priority is automatically given first to Row 1, then to Row 2 and so on.

At each stage, any bolts below the current row are ignored. The resistance of Row 1 is taken solely as the capacity for Row 1 acting alone.

Subsequent rows are checked both in isolation and also as part of a group in combination with successive rows above. The resistance of Row 2 is therefore taken as the lesser of:

- the capacity of Row 2 acting alone, and
- the capacity of Rows (2+1) acting as a group minus the tension already allocated to Row 1.

This process is illustrated in Figure 2.7.

A tension stiffener (or the beam flange) acts as a divider between bolt groups, so that no row below a stiffener need be considered in combination with any row above it for that side of the connection. For example, in Figure 2.7, Rows 2 and 1 are not considered together for group action on the beam side of the connection because the beam flange divides them, but they are considered together for the column side.

The limit on full plastic distribution depending on the ratio of minimum flange or plate thickness to bolt diameter must also be considered, as set out in Step 1C on page 25.

2.3.2 Compression zone

Checks in the compression zone are similar to those traditionally adopted for web bearing and buckling. It is reasonable to expect a properly sawn beam end to provide contact with the end plate, so that compression in the bottom flange is transferred in bearing. Guidance on allowable tolerance between bearing surfaces is given in the *National Structural Steelwork Specification for Building Construction.* [10]

It is common for the column web to be loaded in this region to a point where it controls the design of the connection. However it can be strengthened as shown in Figure 2.9 (page 13.)

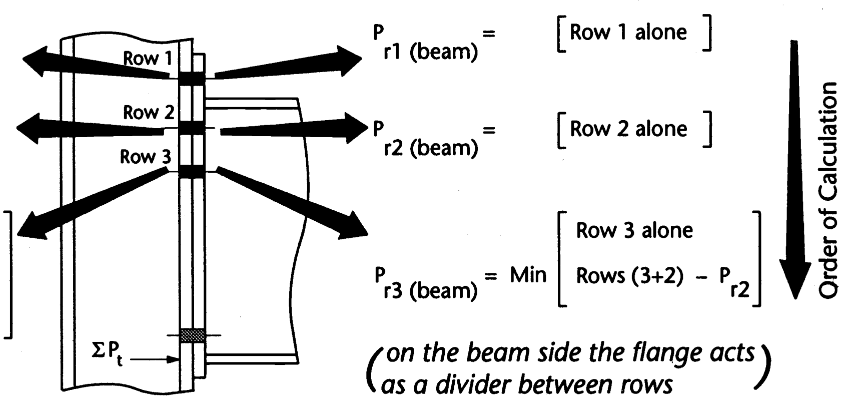
The column web must also be checked for buckling, but in this respect, it may be reasonable to consider whether in some cases buckling is prevented by other beam(s) connecting into the web at right angles to the connection under consideration.

The compression on the beam side can usually be regarded as being carried entirely in the flange, and the centre of compression taken at the centre of the flange. However when large moments combine with axial load, the compression zone will spread up into the beam web with a corresponding movement of the centre of compression.

$$P_{r1\,(col)} = \left[\text{Row 1 alone}\right]$$

$$P_{r2\,(col)} = \text{Min}\begin{bmatrix}\text{Row 2 alone}\\ \text{Rows (2+1)} - P_{r1}\end{bmatrix}$$

$$P_{r3\,(col)} = \text{Min}\begin{bmatrix}\text{Row 3 alone}\\ \text{Rows (3+2)} - P_{r2}\\ \text{Rows (3+2+1)} - P_{r2} - P_{r1}\end{bmatrix}$$



$$P_{r1\,(beam)} = \left[\text{Row 1 alone}\right]$$

$$P_{r2\,(beam)} = \left[\text{Row 2 alone}\right]$$

$$P_{r3\,(beam)} = \text{Min}\begin{bmatrix}\text{Row 3 alone}\\ \text{Rows (3+2)} - P_{r2}\end{bmatrix}$$

(on the beam side the flange acts as a divider between rows)

Note: P_{ri} is the minimum of the column and beam values

Figure 2.7 Steps in calculating the distribution of bolt forces

2.3.3 Shear zone

The column web must also resist the horizontal panel shear forces. To carry out this check, any connection at the opposite flange of the column must also be taken into account, since it is the resultant of the shears which must be borne by the web.

In a one-sided connection with no axial force, the web panel shear F_v is equal to the compressive force 'C'. For a two-sided connection with balanced moments, the column web panel shear will be zero, and in the case of a connection with moments acting in the same sense, such as in a wind-moment frame, the shear will be additive. (See Figure 2.8.)

Table 2.2 Methods of strengthening columns

TYPE OF COLUMN STIFFENER	DEFICIENCY: Web in tension	Flange in bending	Web in bearing	Web in buckling	Web in shear
Flange backing plates		●			
Horizontal stiffeners: Full depth	●	●	●	●	
Horizontal stiffeners: Rib	●	●	●		
Supplementary web plates	●		●	●	●
Diagonal stiffeners					●
Morris stiffeners	●	●			●

The web of most UC section columns will fail in panel shear well before it fails in bearing or buckling and therefore, for one-sided connections, web shear is likely to govern. Where this is critical, the column web can be strengthened by using diagonal stiffeners, or by supplementary web plates as shown in Figure 2.9.

2.4 METHODS OF STRENGTHENING

Careful selection of the members during design will often avoid the need for strengthening at the connection, and will lead to a more cost-efficient structure. Sometimes however 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 2.9.

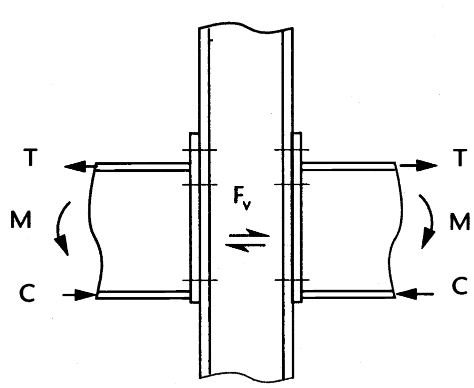
The type of strengthening must be chosen so that it does not clash with other components at the connection. This is often a problem with conventional stiffeners when secondary beams frame into the column web.

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 2.2.

2.5 CONNECTION ROTATIONAL STIFFNESS

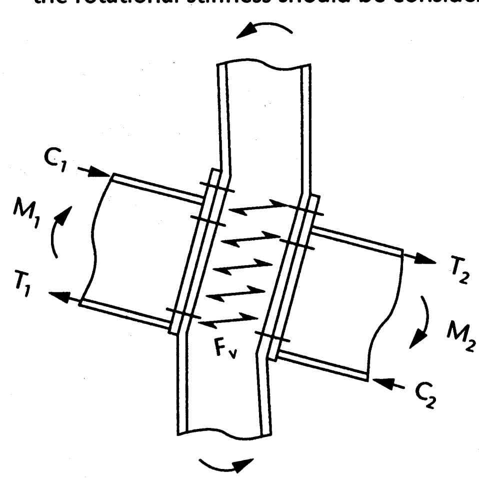
If a continuous frame is analysed elastically, the validity of the result depends upon the connection between the beam and the column having sufficient rotational stiffness. The connections are considered as 'rigid', because their flexibility is low enough to be ignored.

The importance of connection stiffness varies with the type of structure. The following guidance indicates when the rotational stiffness should be considered:



Web panel with no shear

$F_v = 0$



Web panel subject to shear force

$F_v = C_1 + T_2$

Fig 2.8 Column web panel shear

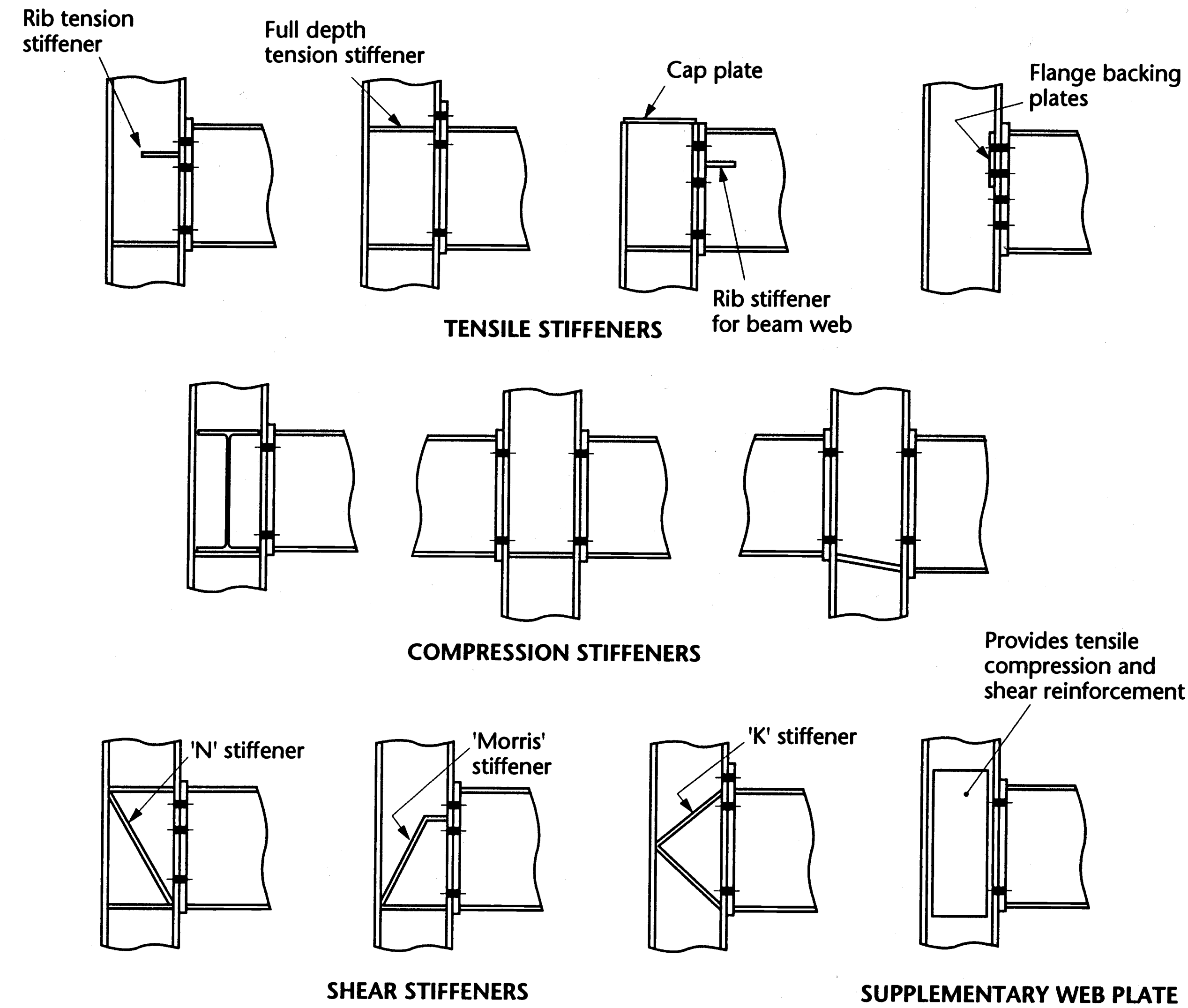


Figure 2.9 Methods of strengthening

Braced frames and single storey portals

Well proportioned connections designed for strength alone may be assumed to be Rigid. The standardised connections indicated in the tables on pages 150-181 and 190-201 are examples of this type of connection.

Wind-moment frames

Wind-moment connections, as described in Section 3, are not regarded as Rigid, and account must be taken of their flexibility in the design of the frame.

For guidance see *Wind-Moment Design for Unbraced Frames* (11)

Multi-storey unbraced frames

Connection rotational stiffness is inherent to the safety of this type of frame. Flexibility in the connection adversely affects frame stability and serviceability.

The connection details must therefore be the responsibility of the frame designer.

The designer may:

either

- estimate connection stiffness and consider this in evaluating λ_{cr} (BS 5950: Part 1, Clauses 5.6 and 5.7). *It is anticipated that a method for calculating connection stiffness will be presented in a revised Annex J of EC 3.*

or satisfy both of the following requirements:

- provide connection details which ensure that Mode 3 is the critical mode. This can be achieved on the beam side of the connection by making the end plate thickness not less than the bolt diameter - spaced within the range given in Section 2.6. The column side of the connection may have to be suitably stiffened with tension and compression stiffeners.

- limit column web panel shear to 80% capacity, failing which provide diagonal stiffeners or supplementary web plates.

2.6 STANDARDISATION

The principles of standardised connections are discussed more fully in *Joints in Simple Construction: Volume 2.*[9] Most of the benefits apply equally to bolted endplate moment connections. Some general recommendations are given below and summarised in Table 2.3. The capacity tables given in the yellow pages are based on these principles.

Bolts

M24 8.8 bolts in clearance holes should be adopted as the 'standard' bolt for moment connections. For some smaller connections - say beams up to 400mm deep and stanchions with thin or narrow flanges - M20 bolts are adequate.

For larger and more heavily loaded connections the designer may need to resort to M30 8.8 or possibly 10.9 bolts. However, care should be taken when using 10.9 bolts owing to their limited ductility. This will not be a problem where bolts are provided which have a minimum of five threads under the nut after tightening.

As with other types of steel structure, the objective should be to restrict the number of different bolts on any one contract. Variations in length can be kept to a minimum by the use of fully threaded bolts.

End plates

End plates, and other fittings, are commonly specified in design grade 43 steel which is usually readily available in small quantities. Design grade 43 can normally be used even when the parent member is in design grade 50 steel.

There should be a sensible relationship between the bolt spacing, bolt size and plate thickness. An efficient solution with design grade 43 steel is to make the plate thickness approximately equal to the bolt diameter, and select bolt spacing, both cross centres and pitch, within the range 80 to 100mm.

For the majority of sections, cross centres of 100mm with M24 bolts and 90mm with M20 bolts are recommended, although the centres must be increased for some Universal Column sections over 200 kg/m to allow sufficient clearance to the root radius.

Within this manual, standard wind-moment connections, portal eaves and portal apex connections use 90mm cross centres with both M24 and M20 bolts.

Haunches

Haunches can be either cut from Universal Beams or built up from flats or plate. The usual practice is to use section cuttings for long haunches, such as those in portal frames, and to size the haunch depth so that the section used can be split diagonally with a single cut.

Whichever method is used, it should be noted that the design procedures assume that the web and flange of the haunch section are at least as thick as the web and flange of the parent member. The design grade of the haunch may have been defined as part of the parent member design.

Table 2.3 Standard components

ELEMENT	PREFERRED OPTION	NOTES
BOLTS	M24 8.8 in clearance holes	M20 8.8 bolts for smaller connections M30 8.8 bolts for larger connections
END PLATES	250 x 25 - M24 bolts 200 x 20 - M20 bolts (All plates in design grade 43 steel)	Plate width may need to increase to suit wider flange beams
HAUNCHES	Section cuttings	- For long (>2000mm) haunches
	Built up plates	- For smaller haunches
WELDS	Fillet welds - 6, 8, 10, 12mm	- For beam webs, stiffeners and most flanges
	Partial penetration butt welds	- When greater than 12FW required

Welds

Fillet welds are generally preferred to butt welds. The welds to the beam web and around stiffeners can almost always be fillets. The minimum recommended size is 6mm.

One exception is the tension flange welds for larger heavily loaded beams for which 12mm fillets may not suffice.

Because of the large volume of weld metal needed and the associated problems of distortion, many fabricators prefer to use a partial or full penetration butt weld rather than fillet welds greater than 12mm, as shown in Figure 2.10. This preference also applies to the compression flange where it is not possible to achieve a bearing fit against the end plate.

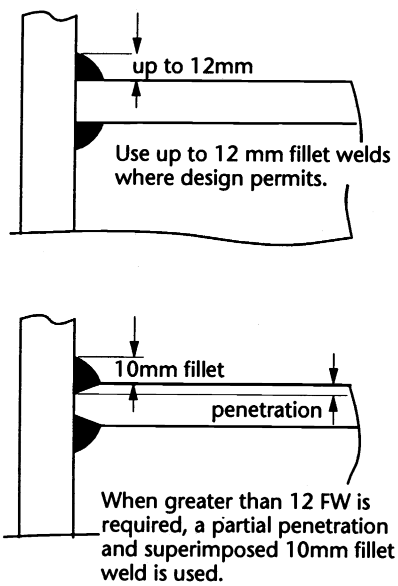


Figure 2.10 Standardised flange welds

Haunch welds

The shear along the haunch length is usually low enough to permit suitably designed intermittent fillet welds to connect the haunch web to the beam flange, although continuous fillets may be specified for aesthetic or corrosion reasons.

The weld between the haunch flange and beam flange is generally made a fillet with a leg length equal to the haunch flange thickness.

2.7 USING THE CAPACITY TABLES

The capacity tables presented in the yellow pages can be used for beam to column connections, and also for portal frame eaves and apex connections. The tables have three uses:

(1) Scheme design stage

When the framing arrangement and member sizes are being considered, the designer can refer to the tables to see if a reasonable connection can be made between the proposed beam and column sections. The necessity of a haunch and the need to stiffen can be investigated.

It will be noted that some connections listed in the tables are partial strength, and do not achieve the full plastic moment resistance of the beam. When it is not possible to achieve a connection capable of developing the full plastic moment of a beam with a flush or extended end plate, the tables will generally provide a haunched connection with a moment capacity greater than that of the beam.

(2) Detailed design stage

The tables may be used to arrive directly at a connection detail, including any stiffening that may be necessary, when the standard range of connections is being used.

(3) Preliminary sizing

When designing connections which are outside the range of the tables, the tables may be used as a guide for choosing a trial configuration for subsequent analysis by hand or computer.

2.8 DESIGN PROCEDURES - RIGOROUS METHOD

Introduction

The following procedures are **not** advocated for routine hand calculations. They are intended:

- as a source of reference for the full method
- for use in writing computer programs
- for use in checking output from computer programs.

The procedures present the method for assessing the moment resistance of bolted end plates. A beam to column flange connection with an extended end plate is used by way of illustration, although the method can easily be adapted to other similar connections such as those shown in Figure 2.1.

The full set of checks needed is shown in Figure 2.4. These are carried out in a logical sequence in the three zones as shown in Figures 2.11 and 2.12.

A worksheet is included on which can be set down in tabular form the process of calculation of the bolt row forces. (See page 26.)

A worked example showing the design of a bolted end plate connection using the rigorous method is given in Appendix I. Examples of stiffener design are included.

Section 2.9 demonstrates how the method can be abridged for manual use by experienced connection designers.

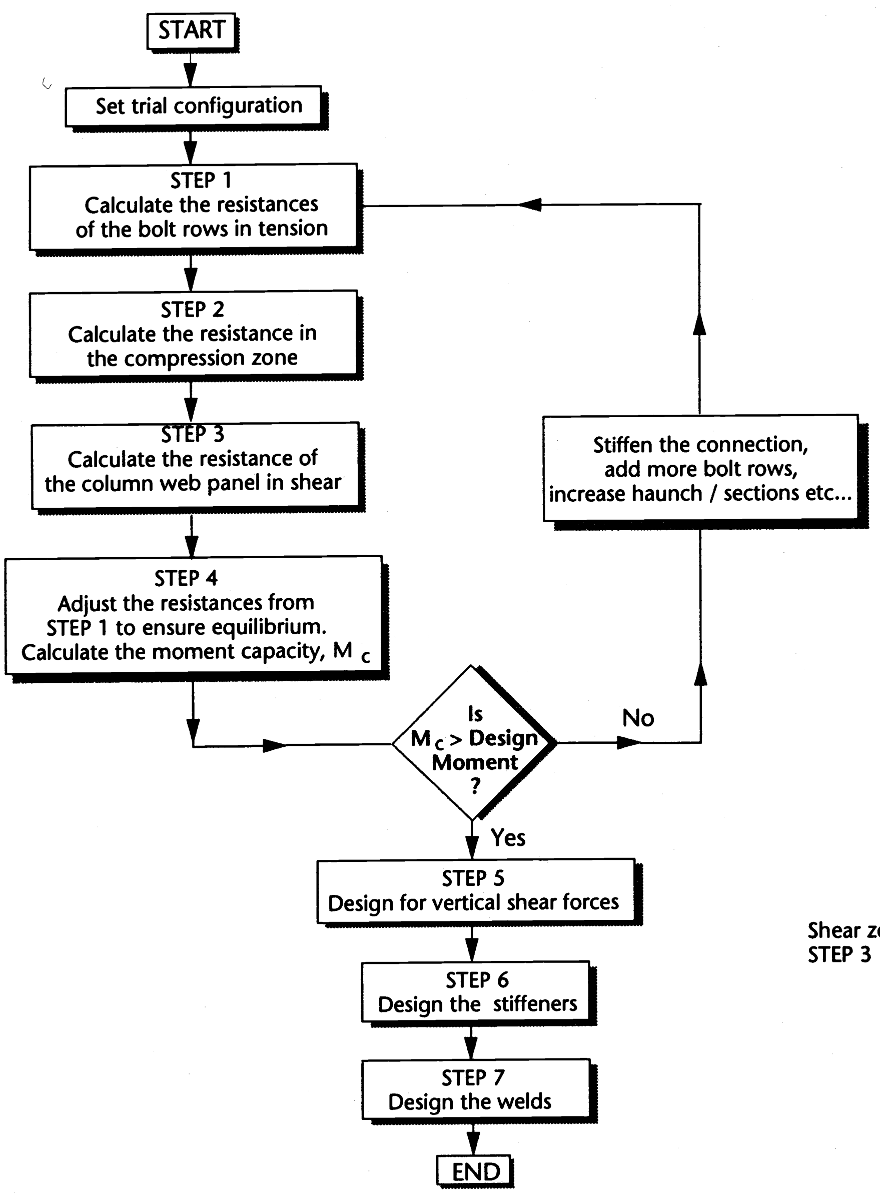


Figure 2.11
Flow diagram - design checks

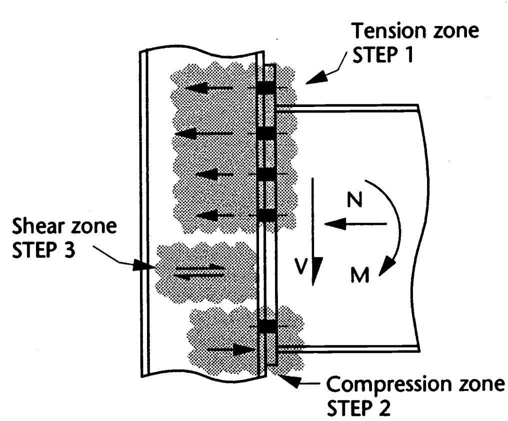


Figure 2.12
Check zones for an extended end plate connection

STEP 1 POTENTIAL RESISTANCES OF BOLT ROWS IN THE TENSION ZONE

General

The force in each row of bolts in the tension zone is limited by bending in the end plate or column flange, bolt failure, or tension failure in the beam or column web.

The procedure is to first calculate the *potential* resistance for each row i.e:

P_{ri} (Figure 2.13)

The values P_{r1}, P_{r2}, P_{r3} etc. are calculated in turn starting at the top row 1 and working down. Priority for load is given to row 1 and then row 2 and so on.

At every stage, **bolts below the current row are ignored.**

Each row is checked first in isolation and then in combination with successive rows above it, i.e.

$$P_{r1} = [\text{capacity of row 1 alone}]$$

$$P_{r2} = \text{Min. of:} \begin{bmatrix} \text{capacity of row 2 alone} \\ (\text{capacity of rows 2+1}) - P_{r1} \end{bmatrix}$$

$$P_{r3} = \text{Min. of:} \begin{bmatrix} \text{capacity of row 3 alone} \\ (\text{capacity of rows 3+2}) - P_{r2} \\ (\text{capacity of rows 3+2+1}) - P_{r2} - P_{r1} \end{bmatrix}$$

....and in a similar manner for subsequent rows.

For each of these checks the capacity of a bolt row or a group of bolt rows in the tension zone is taken as the least of the following four values:

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

In addition, the force in any bolt row may in some cases be limited by the connection's inability to achieve the plastic bolt force distribution without premature bolt failure. This additional check, and the required modification to the distribution, is given in STEP 1C.

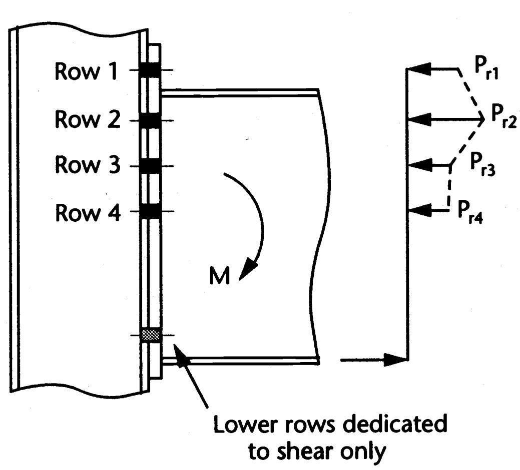


Figure 2.13 Potential resistance of bolt rows

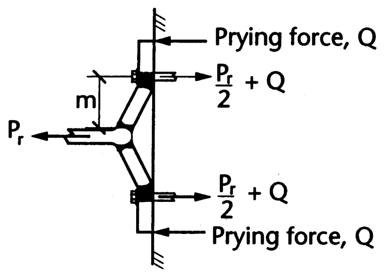
STEP 1A END PLATE OR COLUMN FLANGE BENDING OR BOLT YIELDING

This check is carried out separately for both the column flange and the end plate.

The potential resistance in tension of the column flange or end plate, P_r is taken as the minimum value obtained from the three equations *(2.1)*, *(2.2)* or *(2.3)*, below:

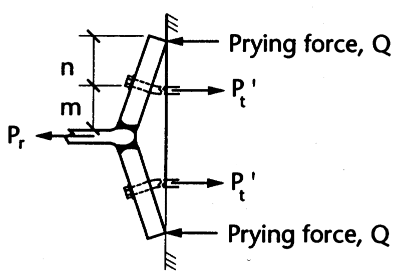
Mode 1 Complete flange yielding

$$P_r = \frac{4M_p}{m} \qquad (2.1)$$



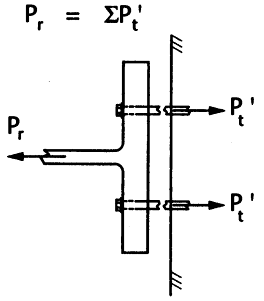
Mode 2 Bolt failure with flange yielding

$$P_r = \frac{2M_p + n(\Sigma P_t')}{m + n} \qquad (2.2)$$



Mode 3 Bolt failure

$$P_r = \Sigma P_t' \qquad (2.3)$$



where:

M_p = plastic moment capacity of the equivalent T-stub representing the column flange or end plate

$= \frac{L_{eff} \times t^2 \times p_y}{4}$

L_{eff} = effective length of yield line in equivalent T-stub (See Tables 2.4, 2.5, 2.6)

t = column flange or end plate thickness

p_y = design strength of column/end plate

P_r = potential resistance of the bolt row, or bolt group

P_t' = enhanced bolt tension capacity where prying is taken into account (See Table 2.1)

$\Sigma P_t'$ = total tension capacity for all the bolts in the group

m = distance from bolt centre to 20% distance into column root or end plate weld (See Figure 2.15 on page 22)

n = effective edge distance. (See Figure 2.15 on page 22)

*For an extended end plate, dimensions m_x and n_x are required, defined on page 22. (m_x and n_x are **only** used in the extension.)*

Backing plates

For small section columns with thin flanges, loose backing plates can increase the resistance of the column flange by preventing a Mode 1 type bending failure. Design rules for backing plates are given in STEP 6C.

Stiffeners

For end plate or column flange bending, bolt groups must be considered separately between stiffeners or the beam flange as shown in Figure 2.14. i.e. the yield pattern of any bolt row below a stiffener (or flange) cannot combine with any rows above it on the side where the stiffener (or flange) is.

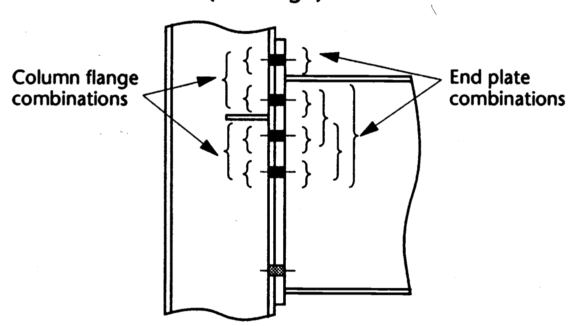
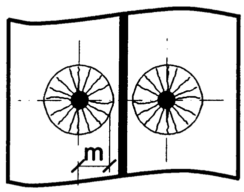
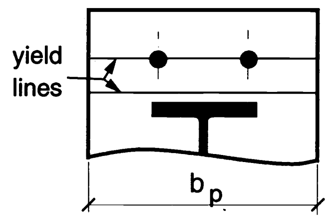
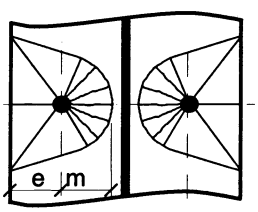
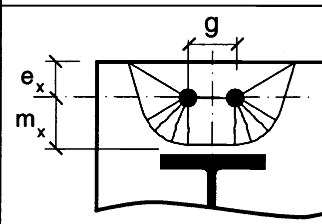
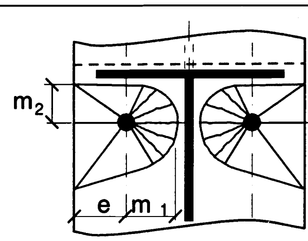
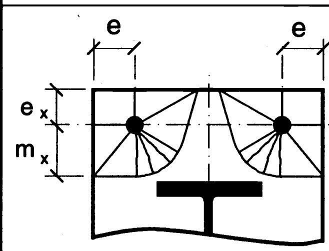
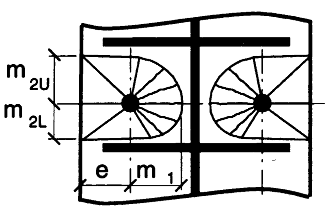
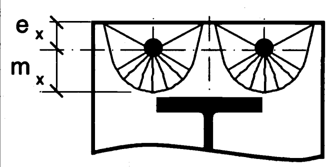
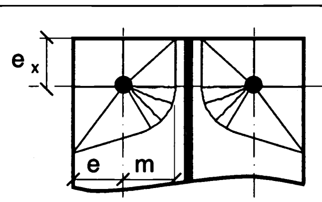
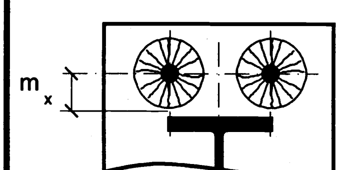
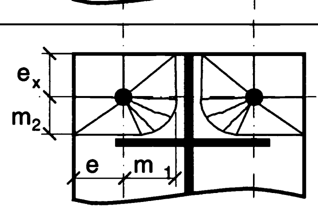
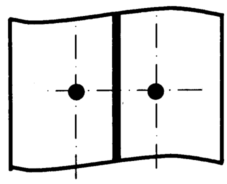
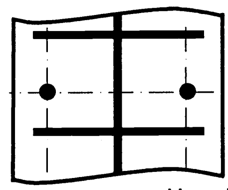
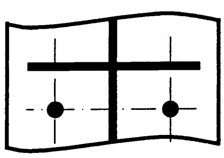
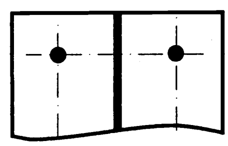
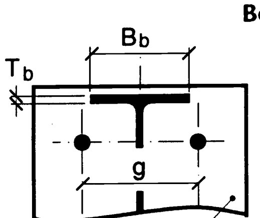
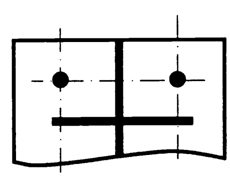
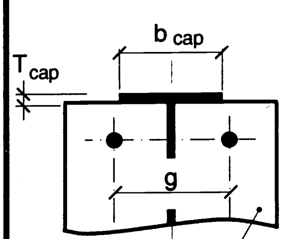
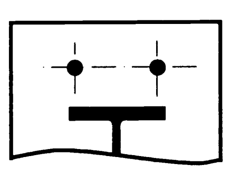


Figure 2.14 Influence of stiffeners

Table 2.4 L_{eff} for equivalent T-stubs for bolt row acting alone			
Pair of bolts separated by a web in a column flange or end plate		**Pair of bolts in a plate extension**	
	**Pattern (i)** Circular yielding $L_{eff} = 2\pi m$		**Pattern (vii)** Double curvature $L_{eff} = \frac{b_p}{2}$
	**Pattern (ii)** Side yielding $L_{eff} = 4m + 1.25e$		**Pattern (viii)** Group end yielding $L_{eff} =$ $2m_x + 0.625e_x + g/2$
	**Pattern (iii)** Side yielding near beam flange or a stiffener $L_{eff} = \alpha m_1$		**Pattern (ix)** Corner yielding $L_{eff} =$ $2m_x + 0.625e_x + e$
	**Pattern (iv)** Side yielding between two stiffeners $L_{eff} =$ $\alpha m_1 + \alpha' m_1 - (4m_1 + 1.25e)$ (α is calculated using m_{2U} α' is calculated using m_{2L})		**Pattern (x)** Individual end yielding $L_{eff} =$ $4m_x + 1.25e_x$
	**Pattern (v)** Corner yielding $L_{eff} = 2m + 0.625e + e_x$		**Pattern (xi)** Circular yielding $L_{eff} = 2\pi m_x$
	**Pattern (vi)** Corner yielding near a stiffener $L_{eff} =$ $\alpha m_1 - (2m_1 + 0.625e) + e_x$		

Notes:
- *Table 2.5 shows which of the above expressions have to be considered.*
- *Table 2.6 shows which parts of the above expressions have to be combined when bolt rows act as a group.*
- *Dimensions m, m_x, e, e_x are shown in Figure 2.15 (page 22).*
- *The value of α is determined from the chart in Figure 2.16 (page 23).*
- *L_{eff} is the length of the equivalent T-stub,* ***not*** *the length of the pattern shown.*

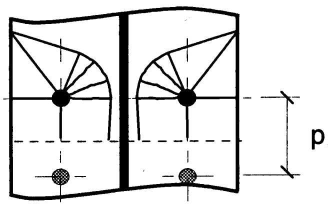
Table 2.5 L_{eff} to be considered for a bolt row acting alone (expressions to be used from Table 2.4)

Bolt row not influenced by a stiffener or a free end  Use: Min{ii,i}	**Bolt row between stiffeners**  Use: Min{Max{iv, iii (m_{2U}), iii(m_{2L}),ii},i} (m_{2U} and m_{2U} are as shown in pattern iv)
Bolt row next to a stiffener or below the beam flange of an extended end plate  Use: Min{Max{ii, iii},i}	**Bolt row next to a free end**  Use: Min{ v,ii,i}
Bolt row below the beam flange of a flush end plate  If $g > 0.7B_b$ or $T_b < 0.8t_p$ Use: Min$\left\{\text{Max}\{(\frac{ii+iii}{2}),ii\},i\right\}$ otherwise Use: Min{Max{ii, iii},i} (this is an interim rule)	**Bolt row between free end and stiffeners**  Use: Min{Max{v,vi},Max{ii,iii},i}
Bolt row next to a column cap plate  If $g > 0.7b_{cap}$ or $T_{cap} < 0.8T_c$ Use: Min$\left\{\text{Max}\{(\frac{ii+iii}{2}),ii\},i\right\}$ otherwise Use: Min{Max{ii, iii},i} (this is an interim rule)	**Bolt row in a plate extension**  Use: Min{ vii,viii,ix,x,xi}

Notes:

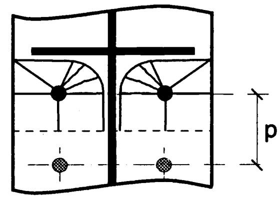
- *Effective length expressions are given in Table 2.4 on page 19.*
- *Any other bolts, above or below are ignored when considering a single bolt row.*
- *The expressions signified by the pattern numbers expressed automatically determine the correct effective length to be used. They take account of any benefit due to the proximity of a stiffener or adverse effect due to a free end.*
- **Min{Max{ii, iii},i}** *means: firstly determine the* ***maximum*** *from patterns (ii) and (iii), then take the* ***minimum*** *of this result and pattern (i).*
  e.g. If patterns (i), (ii) and (iii) gave lengths of 300mm,200mm,100mm respectively then the result would be 200mm .

Table 2.6 L_{eff} to be considered for bolt rows acting in combination (expressions to be used from Table 2.4)



Top or bottom row of a group along a clear length

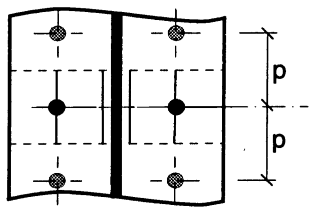
Use: $\frac{ii}{2} + \frac{p}{2}$



Top or bottom row of a group next to a stiffener

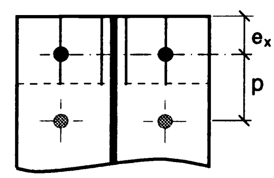
Use:

$\mathrm{Max}\left\{\frac{ii}{2}, \left(iii - \frac{ii}{2}\right)\right\} + \frac{p}{2}$



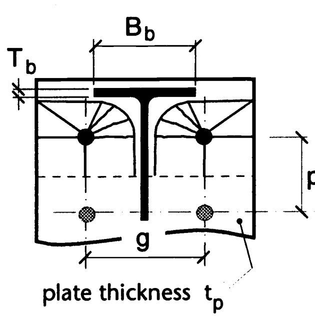
Intermediate row of a group

Use: p



Top or bottom row of a group next to a free edge

Use: $\mathrm{Min}\left\{e_x, \frac{ii}{2}\right\} + \frac{p}{2}$



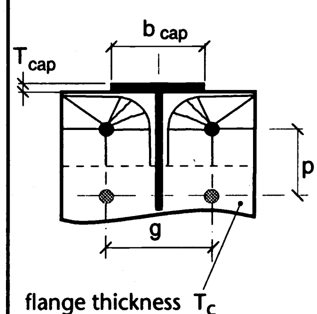
Bolt row of a group below the beam flange of a flush end plate

If $g > 0.7B_b$ or $T_b < 0.8t_p$

Use: $\mathrm{Max}\left\{\frac{ii}{2}, \frac{iii}{2}\right\} + \frac{p}{2}$

otherwise

Use: $\mathrm{Max}\left\{\frac{ii}{2}, \left(iii - \frac{ii}{2}\right)\right\} + \frac{p}{2}$



Bolt row of a group next to a column cap plate

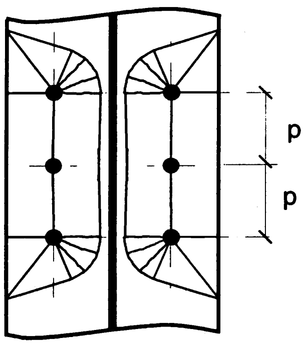
If $g > 0.7b_{cap}$ or $T_{cap} < 0.8T_c$

Use: $\mathrm{Max}\left\{\frac{ii}{2}, \frac{iii}{2}\right\} + \frac{p}{2}$

otherwise

Use: $\mathrm{Max}\left\{\frac{ii}{2}, \left(iii - \frac{ii}{2}\right)\right\} + \frac{p}{2}$

Typical Examples

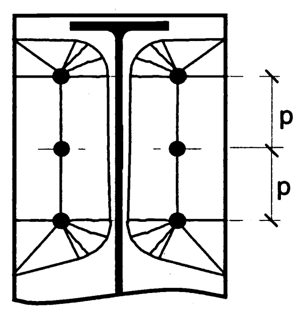


Group of three rows in a clear length

L_{eff} = pattern (ii) + 2p

$\therefore L_{eff}$ for group =

4m + 1.25e + 2p



Group of three rows in a flush end plate

where $T_b < 0.8t_p$

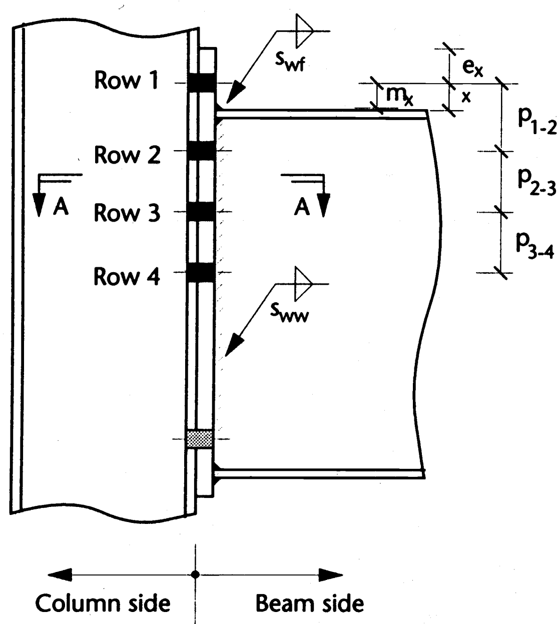
$L_{eff} = $ Max of: $\frac{ii}{2} + \frac{ii}{2} + 2p$ or $\frac{iii}{2} + \frac{ii}{2} + 2p$

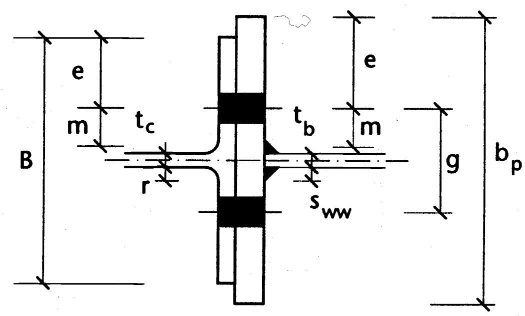
$\therefore L_{eff}$ for group = Max of:

4m + 1.25e + 2p **or** $0.5\alpha m_1 + 2m + 0.625 + 2p$

Note:
- *Effective length expressions are given in Table 2.4 on page 19.*
- *The total effective length of the equivalent T-stub for a group of bolts is the sum of the effective lengths for each row as given above.*

CONNECTION GEOMETRY





Section A-A

For the end plate:

$$m = \frac{g}{2} - \frac{t_b}{2} - 0.8s_{ww}$$

$$e = \frac{b_p}{2} - \frac{g}{2}$$

For the column flange:

$$m = \frac{g}{2} - \frac{t_c}{2} - 0.8r$$

$$e = \frac{B}{2} - \frac{g}{2}$$

Effective edge distance:

Dimension 'n' - used in Mode 2 formula, STEP 1A, is taken as:

for end plate, minimum of:

- 'e' for the column flange
- 'e' for the end plate
- 1.25m for the end plate

for column flange, minimum of:

- 'e' for the column flange
- 'e' for the end plate
- 1.25m for the column flange

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

where:

g	=	horizontal distance between bolt centrelines (gauge)
b_p	=	end plate width
B	=	column flange width
t_b	=	beam web thickness
t_c	=	column web thickness
s_{ww}	=	leg length of fillet weld to beam web
s_{wf}	=	leg length of fillet weld to beam flange

For the end plate extension only:

m_x = $x - 0.8s_{wf}$

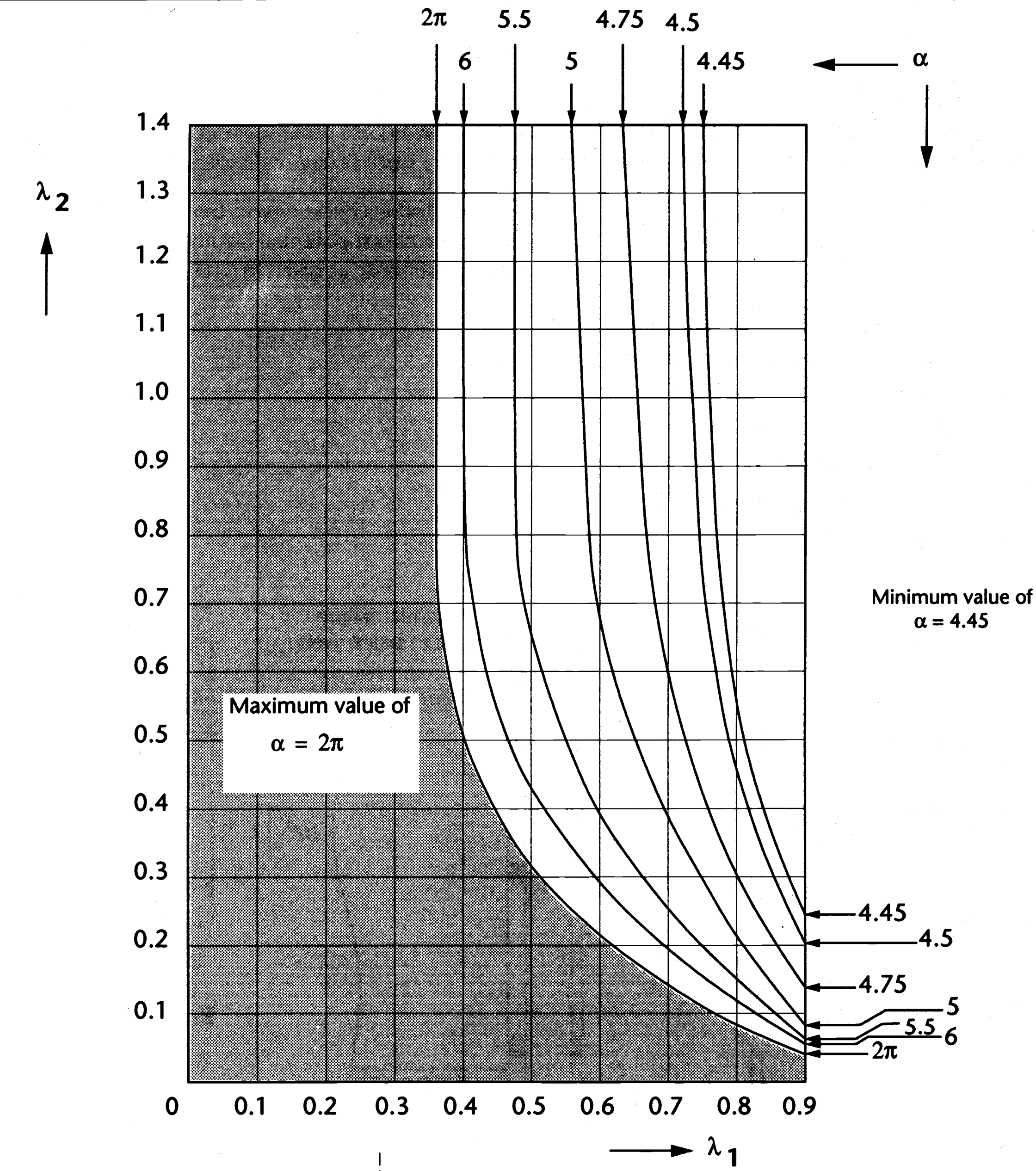
e_x = edge distance, as shown above

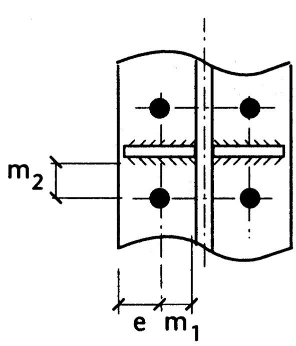
n_x = is the minimum of:

- e_x
- $1.25m_x$

Figure 2.15 Connection geometry

α CHART





$$\lambda_1 = \frac{m_1}{m_1 + e}$$

$$\lambda_2 = \frac{m_2}{m_1 + e}$$

Note: Mathematical expressions for α are given in Appendix III

Figure 2.16 Values of α for stiffened column flanges and endplates

STEP 1B WEB TENSION IN BEAM OR COLUMN

General

This check is carried out separately for both the beam web and the column web. The potential resistance in tension of the web for a row or a group of bolt rows is taken as:

$$P_t = L_t \times t_w \times p_y \qquad (2.4)$$

where:

L_t = effective tensile length of web assuming a maximum spread at 60° from the bolts to the centre of the web (Figure 2.17)

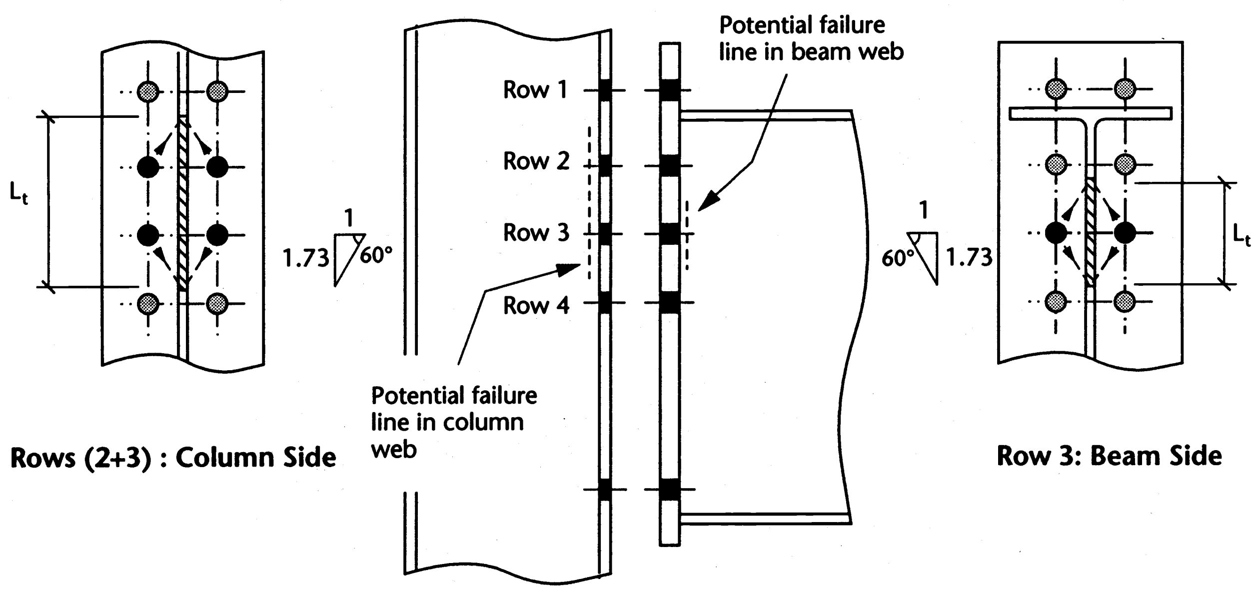
t_w = thickness of the column or beam web

p_y = design strength of the steel in the column or beam.

Stiffeners

Web tension will not govern for any row or group of bolts where stiffeners are present along the tensile length, L_t, which have been properly designed as in STEP 6C.

However, further checks on web tension are also necessary along the weakest potential failure line beyond a partial depth rib stiffener as given in STEP 6C.



Note: Only two examples of web tension checks are illustrated.
Each row and combination of rows must be considered.

Figure 2.17 Typical web tension checks

STEP 1C MODIFICATION OF BOLT ROW FORCE DISTRIBUTION

The method given in STEPS 1A and 1B for assessing the forces in the tension zone produces a plastic distribution of bolt forces.

Often lower rows which are near a flange or stiffener have a greater resistance than higher rows, but some deformation needs to take place to permit them to develop their load.

Some connections with smaller bolts and relatively thick end plates have little deformation capacity. In such cases there is a danger that the upper bolts may fail before resistance is generated in lower rows. See Section 2.2

Plastic distribution limit

The **plastic distribution** must be modified unless:

Either

(a) on the beam side:

$$t_p < \frac{d}{1.9} \times \sqrt{\frac{U_f}{p_{yp}}} \qquad (2.5)$$

Or

(b) on the column side:

$$T_c < \frac{d}{1.9} \times \sqrt{\frac{U_f}{P_{yc}}} \qquad (2.6)$$

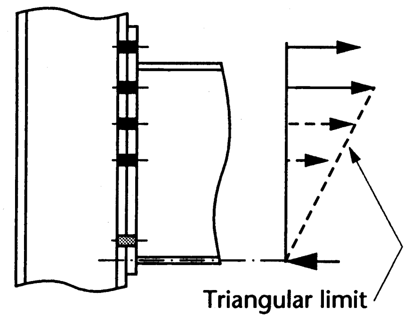
where:

t_p = end plate thickness
T_c = column flange thickness
d = bolt diameter
p_{yp} = design strength of the end plate
p_{yc} = design strength of the column
U_f = ultimate tensile strength of the bolt.

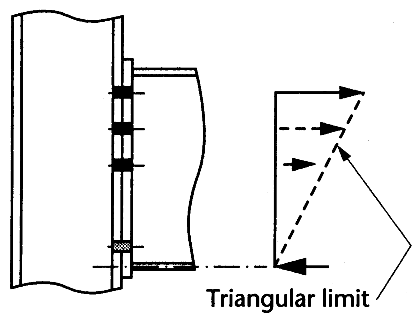
If the above condition is not satisfied, then the force assigned to any lower bolt row is restricted to the value resulting from a 'triangular' limit as shown in Figure 2.18

For this purpose, the centre of rotation is taken as the centre of the compression flange and the triangular limit line should generally be taken from the bolt row immediately below the tension flange. (Where an extended end plate has a vertical stiffener, the line is taken from the top bolt row.)

Where the potential resistance exceeds the triangular limit, it must be reduced, but surplus resistance can be redistributed to the rows below. This process is carried out row by row as potential resistances are calculated.



Extended End Plate



Flush End Plate

Figure 2.18 Triangular limits to bolt forces

Note that the triangular distribution limit line only needs to be imposed if **both** sides of the connection exceed their respective thickness limits.

When 8.8 bolts are used, the plastic distribution limit equations correspond to maximum thicknesses as shown in Table 2.7.

Table 2.7 Maximum thicknesses for unmodified plastic distribution of bolt row forces

8.8 Bolt Dia.	End Plate or Column Flange (mm) (Design Grade 43)	(Design Grade 50)
M20	18.3	16.0
M24	21.9	19.2
M30	27.5	24.0

STEP 1	WORKSHEET: TENSION ZONE					
	Column Side		Beam Side			
Row	STEP 1A Flange Bending	STEP 1 B Web Tension	STEP 1A Plate Bending	STEP 1B Web Tension	STEP 1C Triangular Limit	Potential Resistance
1	*Resistance of Row 1*					least of boxes 1 to 4 gives
	1	2	3	4	N/A 5	P_{r1} = 6
2	*Resistance of row 2 alone:*					
	7	8	9	10		
	Resistance of rows (1 + 2) combined:					least of boxes:
	11	12	13	14		7 to 10 and
	Deduct box 6:					15 to 19 gives
	15	16	17	18	19	P_{r2} = 20
3	*Resistance of row 3 alone:*					
	21	22	23	24		
	Resistance of combined rows (2+3):					
	25	26	27	28		
	Deduct box 20:					least of boxes
	29	30	31	32		21 to 24 and
	Resistance of combined rows (1+2+3):					29 to 32 and
	33	34	35	36		37 to 41 gives
	Deduct sum of boxes 6 & 20:					
	37	38	39	40	41	P_{r3} = 42
4	*Resistance of row 4 alone:*					
	43	44	45	46		
	Resistance of combined rows (3+4):					
	47	48	49	50		
	Deduct box 42:					
	51	52	53	54		
	Resistance of combined rows (2+3+4):					
	55	56	57	58		
	Deduct sum of boxes 20 & 42:					least of boxes
	59	60	61	62		43 to 46 and
	Resistance of combined rows (1+2+3+4):					51 to 54 and
	63	64	65	66		59 to 62 and
	Deduct sum of boxes 6, 20 & 42:					67 to 71 gives
	67	68	69	70	71	P_{r4} = 72

See the worked example using the worksheet in Appendix I

STEP 2A — COMPRESSION CHECK - COLUMN

RESISTANCE OF THE COLUMN WEB IN THE COMPRESSION ZONE

The resistance in the compression zone, P_c is the lesser of *(2.7)* or *(2.8)* below.

For the resistance of stiffened columns, reference should be made to STEP 6A.

Column web crushing (bearing)

An area of web providing resistance to crushing is calculated on the force dispersion length taken from Figure 2.19. (BS 5950: Part 1 Cl 4.5.3)

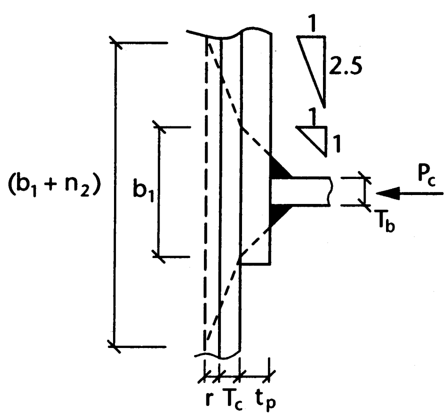


Figure 2.19 Force dispersion for web crushing

$$P_c = (b_1 + n_2) \times t_c \times p_y \qquad (2.7)$$

where:

- b_1 = stiff bearing length based on a 45° dispersion through the end plate from the edge of the welds
- n_2 = length obtained by a 1:2.5 dispersion through the column flange and root radius
- t_c = column web thickness
- p_{yc} = design strength of the column
- t_p = end plate thickness
- T_c = column flange thickness
- r = column root radius.

Column web buckling

An area of web providing resistance to buckling is calculated on a web length taken from Figure 2.20. (BS 5950: Part 1 Cl 4.5.2.1)

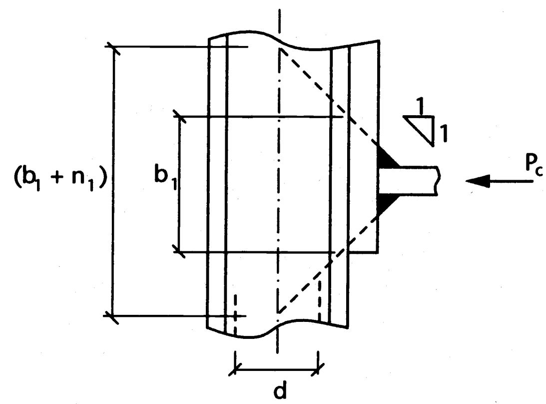


Figure 2.20 Length for web buckling

$$P_c = (b_1 + n_1) \times t_c \times p_c \qquad (2.8)$$

where,

- b_1 = stiff bearing length as above
- n_1 = length obtained by a 45° dispersion through half the depth of the column
  = column depth (D_c)
- t_c = column web thickness
- p_c = compressive strength of the column web from BS 5950: Part 1 Table 27(c) with $\lambda = 2.5d/t_c$
- d = depth of web between fillets

The above expression assumes that the column flanges are laterally restrained relative to one another. (BS 5950: Part 1 clause 4.5.2.1). If this is not the case, further reference should be made to BS 5950: Part 1 clause 4.5.1.5 and 4.5.2.1.

Note: b_1, n_1, n_2, must be reduced if:

- *the end plate projection is insufficient for full dispersal.*
- *the column projection is insufficient for full dispersal.*

STEP 2B — COMPRESSION CHECK - BEAM

RESISTANCE OF THE BEAM FLANGE AND WEB IN THE COMPRESSION ZONE

Beam flange crushing (bearing)

The potential resistance of the flange in compression is taken as:

$$P_c = 1.4 \times p_{yb} \times T_b \times B_b \qquad (2.9)$$

where:

p_{yb} = design strength of the beam
T_b = the beam flange thickness
B_b = the beam flange breadth.

The centre of compression is taken as coinciding with the centre of the beam compression flange as shown in Figure 2.21. This accords with the behaviour of connections under test. (12)

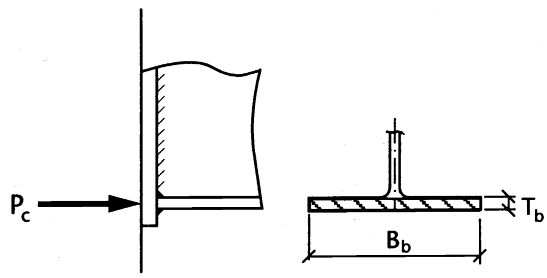


Figure 2.21 Compression in beam flange only

Allowing the flange bearing stress to exceed the yield stress by a factor of 1.4 is justified by two localised effects. It is a combination of strain-hardening and dispersion into the web at the root of the section. Typically (for UB sections) each of these effects will account for around 20% 'overstress', so that an effective $1.4p_{yb}$ can be taken when the flange area is assumed to act alone.

For most moment connections, this simplified check will establish that compression flange crushing does not govern. However, there will be circumstances in which this limit is exceeded, notably when axial compression is present.

In such cases a ⊥ shaped compression zone should be taken extending some distance up the web, as in Figure 2.22.

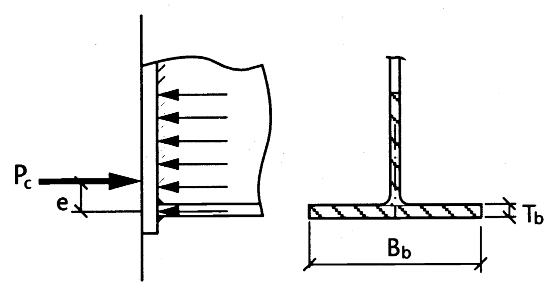


Figure 2.22 Compression in beam flange and portion of web

But note that:

- The stress in this ⊥ section is limited to $1.2p_y$, since the contribution of the web is now being taken into account.
- The centre of compression is redefined as the centroid of the ⊥ section needed to resist F_c, and the lever arm of the bolts is reduced accordingly.
- An iterative calculation process becomes necessary.

Haunched connection

When a haunched connection is adopted, the haunch flange resists compressive forces. See also STEP 8 for the haunch design.

STEP 3 DESIGN FOR COLUMN PANEL SHEAR

RESISTANCE OF THE COLUMN WEB PANEL IN SHEAR

The resistance of an unstiffened column web panel in shear (Figure 2.23) is:

$$P_v = 0.6 \times p_{yc} \times t_c \times D_c \qquad (2.10)$$

where:

p_{yc} = design strength of the column

t_c = column web thickness

D_c = column section depth.

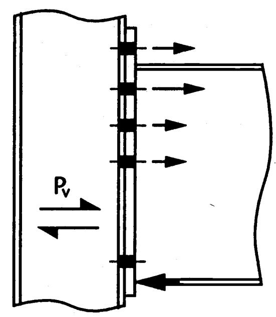


Figure 2.23 Local shear in web

It is the resultant panel shear from connections to both column flanges which must be taken into account when checking the web as shown in Figure 2.24.

In a one-sided connection with no axial force, the shear in the column web will be equal to the compressive force F_c.

For a two-sided connection with balanced moments, the shear is zero, but in the case of a connection with moments acting in the same direction, such as in a wind moment frame, the shear is additive.

The resistance of stiffened columns can be determined with reference to STEPS 6D and 6E.

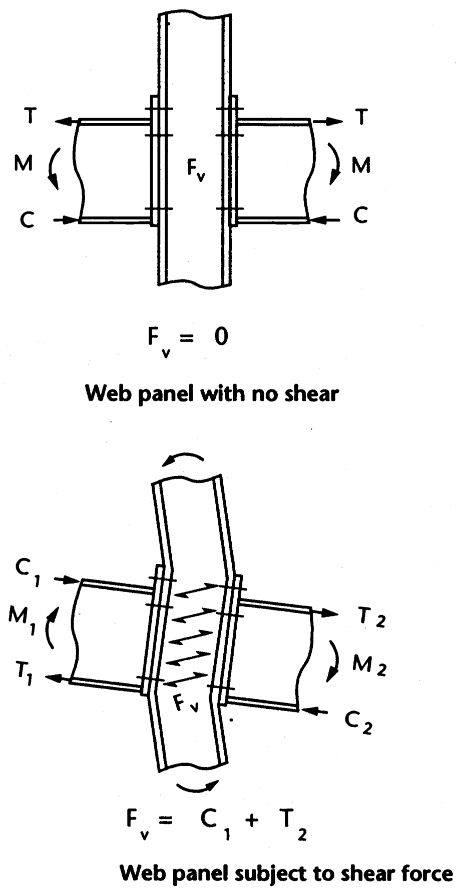


Figure 2.24 Forces and deformation of web panel

STEP 4 CALCULATION OF MOMENT CAPACITY

Force distribution

The bolt row forces in the connection are the potential resistances, reduced if necessary to ensure equilibrium in the horizontal direction. Figure 2.25 shows the potential resistances (P) translated into the actual bolt row forces (F).

Equilibrium is satisfied by:

$$\Sigma F_{ri} + N = F_c$$

where N is the axial load in the beam *(positive for compression)*

and F_c is the smallest of the following:

$\Sigma P_{ri} + N$

or P_c *(column web crushing (bearing))*

or P_c *(column web buckling)*

or P_c *(beam flange crushing (bearing.))*

and Column web panel shear requirements must be satisfied *(see STEP 3)*

For each bolt row:

$$F_{ri} \leq P_{ri}$$

where:

P_{ri} = potential force in bolt row i

F_{ri} = final force in bolt row i.

If there is a surplus capacity in the bolts in tension, then the forces should be reduced, starting with the bottom row and working up progressively until equilibrium is achieved.

Moment capacity

Basic requirement

$$M_c \geq M \text{ (or } M_m \text{ when modified by axial load)}$$

The moment capacity of the connection is :

$$M_c = \Sigma (F_{ri} \times h_i)$$

where:

h_i = distance from the centre of compression to row i.

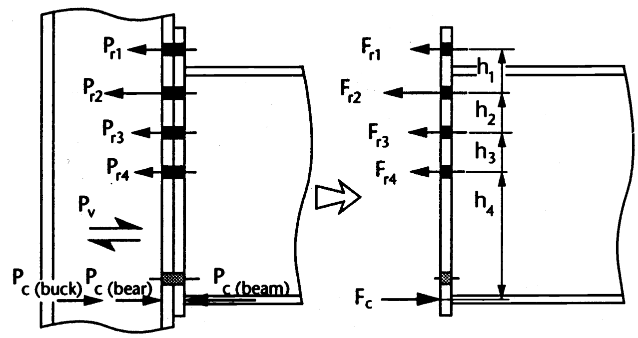
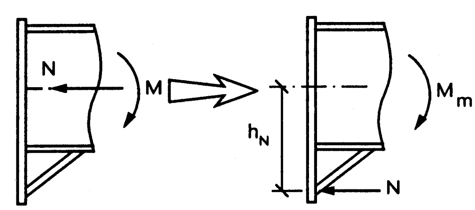


Figure 2.25 Translation of potential resistances into bolt row forces

Applied moment modified by axial load

If an axial load is present it may be considered as being applied at the centre of compression and the applied moment modified accordingly.

The lever arm used must correspond to the location of the force assumed in the analysis (usually the member centre line).



The modified moment M_m is given by:

$$M_m = M - N \times h_N$$

where:

M = applied moment

N = axial force

h_N = distance of axial force from centre of compression.

STEP 5 DESIGN FOR VERTICAL SHEAR FORCES

Comprehensive capacity checks for end plate connections subjected to vertical shear are given in *Joints in Simple Construction, Volume 1.*[(9)]

However, for full depth, fully welded endplates connected to column flanges, many of these checks can be safely omitted. The vertical shear capacity is calculated using a reduced value for bolt rows which are in the tension zone, plus full shear value for bolt rows ignored when calculating moment capacity.

Therefore it is required that:

$$V \leq n_s \times P_{ss} + n_t \times P_{ts}$$

where:

V = design shear force

n_s = number of bolts not in the tension zone

n_t = number of bolts in the tension zone

P_{ss} = shear capacity of a single bolt in shear only which is the least of:

- $p_s A_s$ for bolt shear, or
- $d\ t_p\ p_b$ for bolt bearing on the endplate, or
- $d\ T_f\ p_b$ for bolt bearing on the column flange

P_{ts} = shear capacity of a single bolt in the tension zone which is the least of:

- $0.4\ p_s A_s$ for bolt shear, or
- $d\ t_p\ p_b$ for bolt bearing on the endplate, or
- $d\ T_c\ p_b$ for bolt bearing on the column flange

p_s = shear strength of the bolt (BS 5950: Part 1 Table 32)

A_s = shear area of the bolt (the threaded area is recommended)

T_c = column flange thickness (Figure 2.26)

t_p = end plate thickness (Figure 2.26)

p_b = minimum value of bearing strength for either the bolt, p_{bb} or the connected parts, p_{bs}. (BS 5950: Part 1 Tables 32 & 33)

Note: *The above expression conservatively assumes that all the bolts in the tension zone are fully stressed in tension. Any other assumption involves quantifying the prying force Q, row by row, and is not recommended*

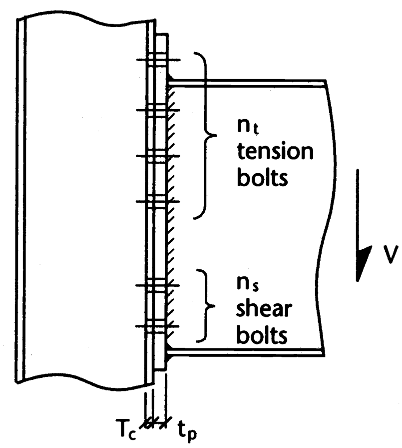


Figure 2.26 Tension and shear bolts

Table 2.8 Shear capacities of single 8.8 bolts

Bolt Size	Bolts in shear only	Bolts in shear and tension
	kN	kN
M20	91.9	36.8
M24	132	53
M30	210	84.2

Capacities are based on the tensile area of the bolt. *See table on page 221 for bearing capacities.*

STEP 6A DESIGN OF COLUMN COMPRESSION STIFFENERS

The resistance in the compression zone, P_c of a column web reinforced with full depth stiffeners as shown in Figure 2.27 is the lower value from equations *(2.11)*, and *(2.12)* below. This must equal or exceed the compressive force, F_c derived in STEP 4.

In addition a further check must be made to ensure that the stiffeners alone can carry, in bearing, 80% of the applied force. See equation *(2.13)*.
(This is usually the formula which governs.)

Stiffeners are usually designed in grade 43 steel.

Effective outstand of compression stiffeners

The outstand of grade 43 compression stiffeners b_{sg} should not exceed $19t_s$ (see Figure 2.27).

When the outstand is between $13t_s$ and $19t_s$ design should be on the basis of a core section of $13t_s$.
(See BS 5950: Part 1 Cl. 4.5.1.2 when stiffeners are designed in other grades of steel.)

where:

b_{sg} = stiffener outstand (see Figure 2.27)
t_s = thickness of stiffener.

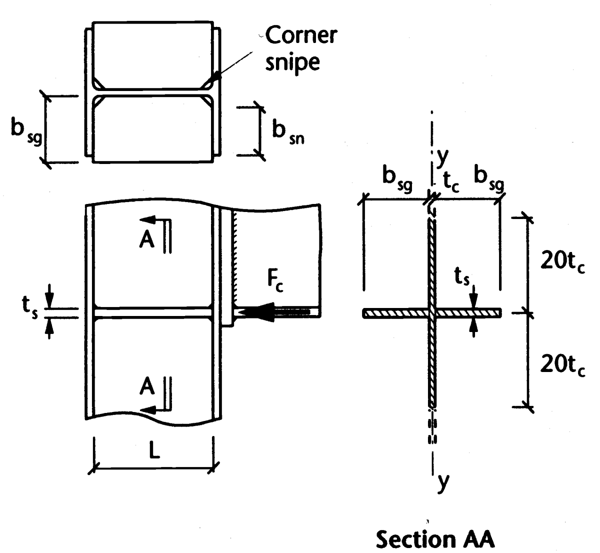


Figure 2.27 Stiffener bearing and buckling check

Stiffener/Column web crushing and buckling

(BS 5950: Part 1 Clauses. 4.5.4.1, 4.5.4.2 and 4.5.5.)

$$P_{c\,buckling} = (A_w + A_{sg}) \times p_c \qquad (2.11)$$

$$P_{c\,crushing} = [A_{sn} \times p_y] + [(b_1 + n_2) \times t_c \times p_y] \qquad (2.12)$$

and:

$$P_{c\,bearing} = \frac{A_{sn} \times p_{ys}}{0.8} \qquad (2.13)$$

where:

A_w = allowable area of column web for buckling (see section AA in Figure 2.27)
= $40t_c \times t_c$ (maximum)

A_{sg} = gross area of stiffeners
= $2 \times b_{sg} \times t_s$ $(b_{sg} \le 13t_s)$

A_{sn} = net area of stiffeners in contact with column flange.
= $2 \times b_{sn} \times t_s$

p_c = compressive strength of stiffeners from BS 5950: Part 1 Table 27(c) with $\lambda = 0.7L / r_y$*

L = Length of stiffener $= D_c - 2T_c$

r_y = radius of gyration of effective area (as shown in section AA, Figure 2.27)

p_y = lesser of the design strength of stiffener or column

p_{ys} = design strength of stiffener

(b_1+n_2) = effective bearing length along web. (see STEP 2)

* *The effective buckling length of the stiffener given here assumes that the column flanges are laterally restrained relative to one another. For other cases refer to BS 5950 Clause 4.5.1.5.*

Mar. 97 Revision: Figure 2.27 and A_w modified

STEP 6A DESIGN OF COLUMN COMPRESSION STIFFENERS (CONTINUED)

Column cap plates

To ensure that yield patterns occur in the column flange and not in the cap plate, the cap plate should be sized such that:

$b_{cap} \geq g$ and $T_{cap} \geq 0.8T_c$

where:

b_{cap} = width of cap plate

T_{cap} = thickness of cap plate.

Cap plate/Column web crushing and buckling must also be checked when the cap plate is in compression in a similar manner to that shown on page 32.

Weld design

The welds connecting compression stiffeners to the column web and flanges will generally be fillet welds and should be designed to BS 5950 Clauses 4.5.9 and 4.5.11 as follows:

Welds to Flanges

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 nominal, say 6mm fillet welds, otherwise the welds should be designed as full strength.

Welds to Web

The web welds must be designed to carry,

for single sided connections :

- the beam compression flange force,

for double sided connections :

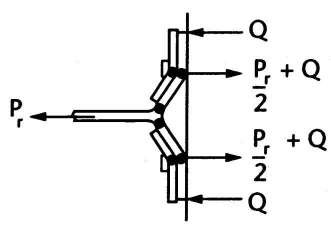
- the sum of the beam flange forces, where the forces act in the same global direction, **or**
- the larger of the beam flange forces, when the forces act in opposite directions.

STEP 6B DESIGN USING COLUMN FLANGE BACKING PLATES

The potential resistance in tension of a column flange strengthened by backing plates is taken as the minimum value obtained from the three equations *(2.14)*, *(2.2)* and *(2.3)* below.

Mode 1 Complete flange yielding

$$P_r = \frac{4M_p + 2M_{bp}}{m} \qquad (2.14)$$



Mode 2 Bolt failure with flange yielding

$$P_r = \frac{2M_p + n(\Sigma P_t')}{m + n} \qquad (2.2)$$

Mode 3 Bolt failure

$$P_r = \Sigma P_t' \qquad (2.3)$$

where:

$$M_{bp} = \frac{L_{eff} \times t_{bp}^2 \times p_y}{4}$$

t_{bp} = thickness of the backing plate

p_y = design strength of the backing plate.

other variables as defined in STEP 1A.

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 not be less than the length of effective T-stub for the bolt group (L_{eff}) and be such that it extends not less than 2d beyond the bolts at each end. (d is the bolt diameter)

This type of strengthening is useful for smaller section columns where flanges are particularly thin. The plates are generally supplied loose or tack-welded in place and their effect is to prevent or increase the resistance to a Mode 1 bending failure. Mode 3 is not affected.

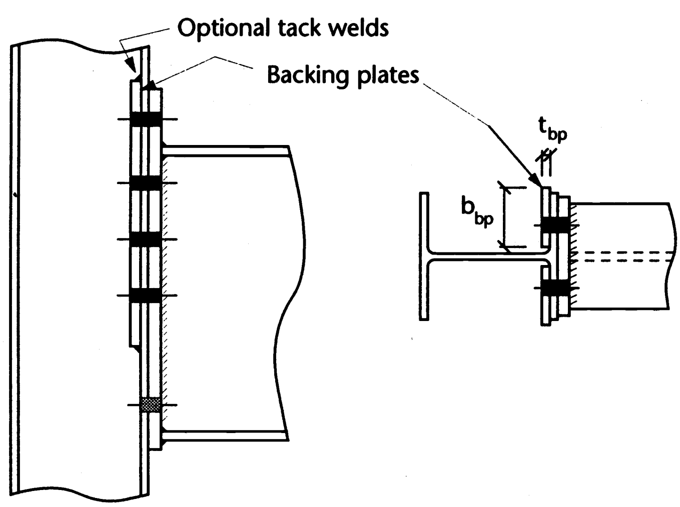


Figure 2.28 Column flange backing plates

STEP 6C DESIGN OF TENSION STIFFENERS

General

Tension stiffeners as shown in Figure 2.29 are generally used to supplement the tension capacity of the column web and/or the capacity of the column flange in bending. Stiffeners may be full depth or partial depth. Partial depth stiffeners are also known as Rib stiffeners.(Figure 2.29)

The design rules given here apply equally to stiffeners on the beam side.

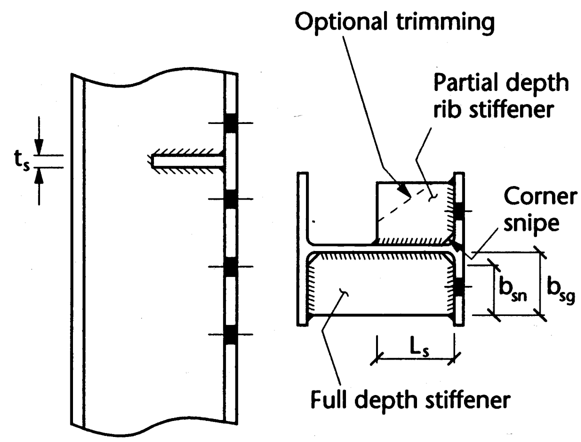


Figure 2.29 Tension stiffeners

Stiffener net area

The net area of the stiffeners, A_{sn} must be not less than the values given in equations *(2.17) and (2.18)*.

Web tension

The rib stiffener is designed to carry the tensile load from the bolts immediately above and below it minus the local capacity of the column web.

Basic requirement:

$$A_{sn} \geq \frac{(F_{ri} + F_{rj})}{p_y} - (L_t \times t_c) \qquad (2.17)$$

where:

- A_{sn} = net area of both stiffeners = $2\,(b_{sn} \times t_s)$
- b_{sg} = width of stiffener. This should generally be proportioned so that the stiffener extends at least 75% across the available flange width, $(B_c - t_c)/2$
- b_{sn} = net width of stiffener.
- t_s = thickness of stiffener.
- F_{ri} = tension from bolt row above the stiffener.
- F_{rj} = tension from bolt row below the stiffener.
- p_y = the design strength of stiffener or column web (the lesser of the two).
- L_t = Available length of web assuming a spread of load at 60° from the bolts (See Figure 2.30).
- t_c = web thickness.

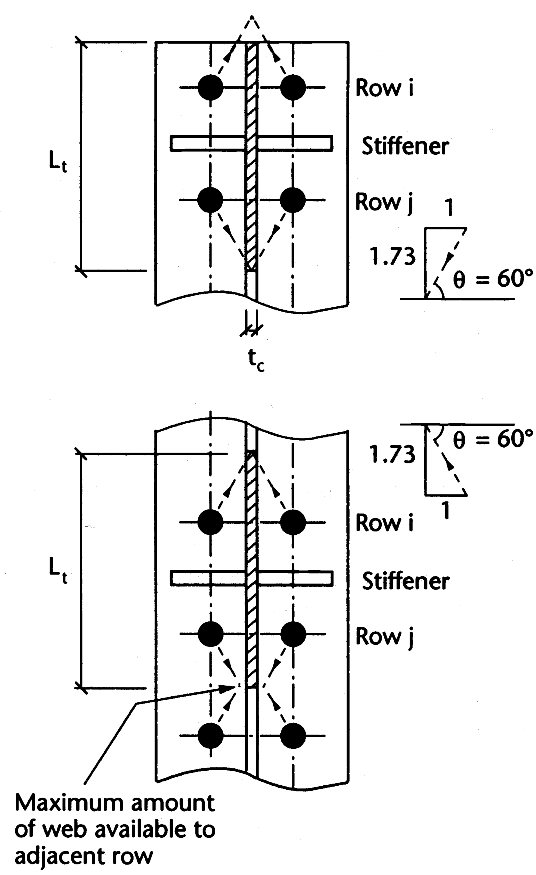


Figure 2.30 Effective web lengths

STEP 6C TENSION STIFFENERS (CONTINUED)

Flange bending

The force carried by the stiffeners is assumed to be inversely proportional to their distance from the bolts.

Basic requirement:

$$A_{sn} \geq \frac{m_1}{p_y}\left[\frac{F_{ri}}{(m_1+m_{2L})}+\frac{F_{rj}}{(m_1+m_{2U})}\right] \qquad (2.18)$$

where:

A_{sn} = net area of both stiffeners

= $2\,(b_{sn} \times t_s)$

m_1 m_{2L} m_{2U} F_{ri} and F_{rj} are as shown in Figure 2.31.

p_y = the design strength of stiffener or column (the lesser of the two).

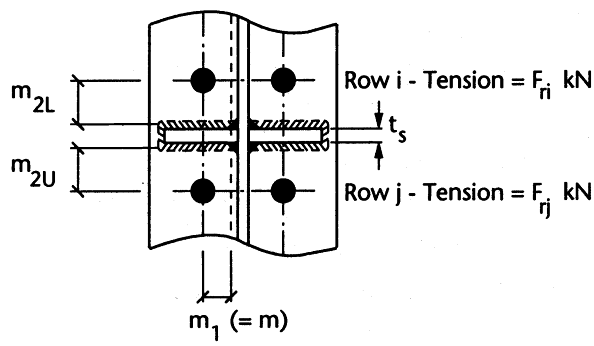


Figure 2.31
Geometry for force distribution to stiffeners

Stiffener length and weld design

If $L_s \geq 1.8\,b_{sg}$ and full strength welds are provided to the flange and web, no further calculations for the welds are required. The column web must still be checked (see below).

If $L_s < 1.8\,b_{sg}$, the welds to the stiffener should be designed assuming rotation about the root of the column. (Figure 2.32).

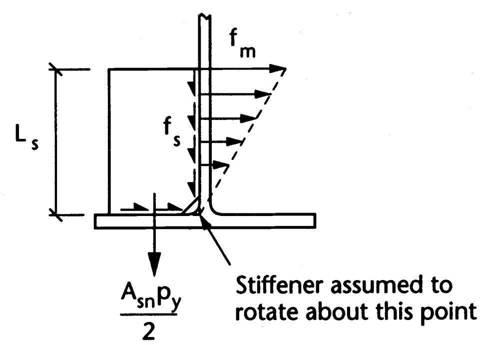


Figure 2.32
Rotation of rib stiffeners about the column root

Additional web tension check

Partial depth rib stiffeners should also be long enough to prevent web tension failure along any line beyond the end of the stiffener.

The stiffener length, L_s should be such that this condition is prevented. See Figure 2.33.

In this particular illustration, for rows 1+2 +3 considered as a group, the basic requirement is:

$$(L_1 + L_2) \geq \frac{F_{r\,(1+2+3)}}{t_c \times p_y} \qquad (2.19)$$

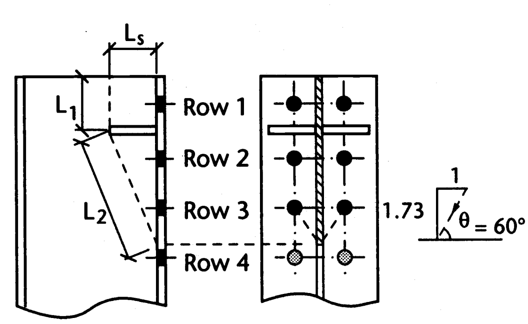


Figure 2.33 Web tension in the presence of stiffeners

STEP 6D DESIGN OF SUPPLEMENTARY WEB PLATES

General

A supplementary web plate (SWP) may be provided to increase the capacity of the column web. Its effect (to EC3) is to:

- Increase **web tension** resistance by:
  50% with a plate on one side or
  100% with plates on both sides
- Increase **web crushing** resistance by:
  50% with a plate on one side or
  100% with plates on both sides
- Increase **web panel shear** resistance by:
  about 75% (see expression for P_v).

Note that in the case of panel shear, plates on both sides provide *no additional* increase over a plate on one side.

The supplementary web plate must have:

- **Thickness,** t_s not less than the column web thickness.
- The same **design strength** as the column.
- **Welds** all round should be, as a minimum, fillet welds of leg length equal to the plate thickness t_s. However, if the supplementary web plate is being used to increase web tension resistance, the vertical weld on the side where the increased capacity is required should be a 'fill in' weld. (See Figure 2.34.) Plug welds are required if b_s exceeds $37t_s$ (design grade 43) or $33t_s$ (design grade 50).
- **Breadth,** b_s so that $b_s \geq d - 2t_s$

  ($b_s = d$ for a "fill in" weld)
- **Length,** $L_s \geq g + L_c + \dfrac{D_c}{2}$

  where:

  g = horizontal spacing of bolts (gauge)

  L_c = length of beam connection end plate

  D_c = depth of column

  (strictly, the length g may be taken as $1.73\frac{g}{2}$, and measured from the top row of bolts.)

Column web tension

For the purpose of column web tension calculations (STEP 1B), the effective web thickness, t_{eff} should be taken as:

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

For SWP's on both sides, $t_{eff} = 2t_c$

Where t_c is the column web thickness.

Column web crushing and buckling

For the purpose of column web crushing and column web buckling calculations (STEP 2), the effective web thickness, t_{eff} should be taken as:

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

For SWP's on both sides, $t_{eff} = 2t_c$

Where t_c is the column web thickness.

Column panel shear

The resistance, P_v of a column web panel with a SWP on one side is given by:

$$P_v = 0.6 \times p_y \times A_v$$

where:

p_y = design strength of the column

A_v = shear area of the column web and SWP combined
$= t_c \times (D_c + b_s)$.

No further increase of the shear area is made if a SWP is added on the other side of the web.

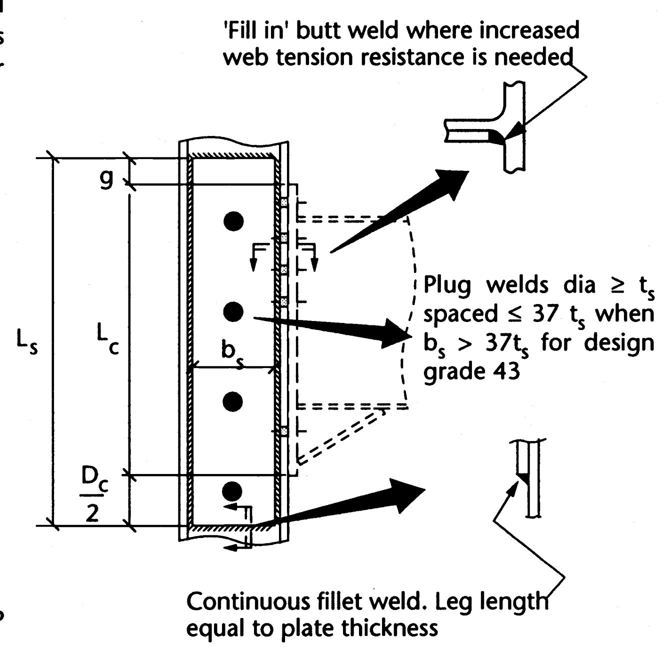


Figure 2.34 Dimensions and welds

STEP 6E DESIGN OF DIAGONAL SHEAR STIFFENERS

Three types of diagonal shear stiffener are shown in Figure 2.35. They are normally designed in grade 43 steel.

Area of stiffeners

The area of the stiffeners, A_{sg} is given by:

$$A_{sg} \geq \frac{(F_v - P_v)}{p_y \cos\theta} \qquad (2.20)$$

where:

A_{sg} = $2 \times b_{sg} \times t_s$

b_{sg} = Width of stiffener on each side

t_s = thickness of stiffener

F_v = the applied shear force (see STEP 3)

P_v = resistance of the unstiffened column web panel (see STEP 3)

p_y = lower design strength of stiffener or column

θ = angle of stiffener from horizontal (see Figure 2.35).

'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 placing and tightening bolts.

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

'N' stiffener

'N' stiffeners are usually placed so that they act in compression due to problems of bolt access if placed so as to act in tension. Check as a compression stiffener as in STEP 6A, 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 UBs as columns, but is difficult to accommodate in the smaller UC sizes.

The horizontal portion carries the same forces as a tension stiffener located in the same position. The length should be sufficient to provide for bolt access (say 100mm).

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 2.35.

Welds connecting the horizontal portion of Morris stiffeners to the column flange should be designed to provide a net throat area at least equal to A_{sn} calculated by formula *(2.18)* in STEP 6C. The throat should be based on b_{sn}.

The welds to the column web may be nominal 6mm or 8mm fillet welds.

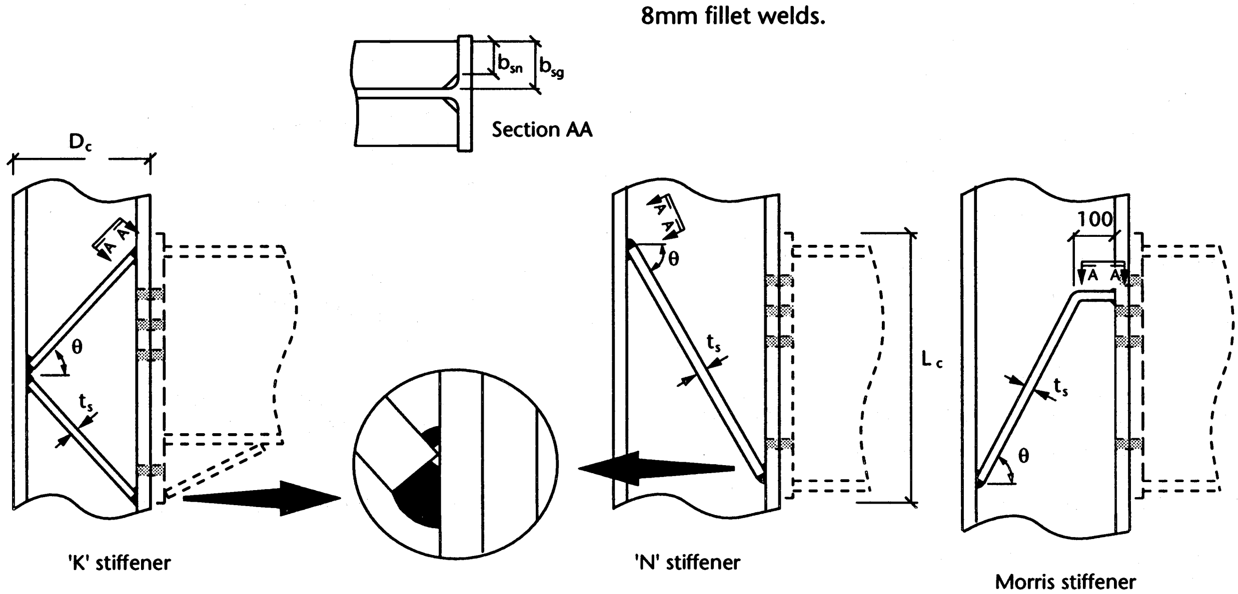


Figure 2.35 Diagonal shear stiffeners

STEP 7 DESIGN OF WELDS

Tension flange welds

The welds between the tension flange and the end plate may be full strength, or should be designed to carry a force which is the lesser of:

(a) The tension capacity of the flange,

$$= B \times T \times p_y$$

(b) The total tension force in the top three bolt rows for an extended end plate (see Figure 2.36),

$$= (F_{r1} + F_{r2} + F_{r3})$$

or the total tension force in the top two bolt rows for a flush end plate.

$$= (F_{r1} + F_{r2})$$

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 12mm then a full strength detail with partial penetration butt welds and superimposed fillets may be a more economical solution.

The transition between a larger flange weld and the web weld should take place where the root of the section meets the web.

The approach given above may appear conservative but, at ultimate limit state, there can be a tendency for the end plate to span vertically between the beam flanges. As a consequence, more load is attracted to the tension flange than from the adjacent bolts alone.

For this reason, care should be taken not to undersize the weld to the tension flange. A simple and safe solution is to provide full strength welds.

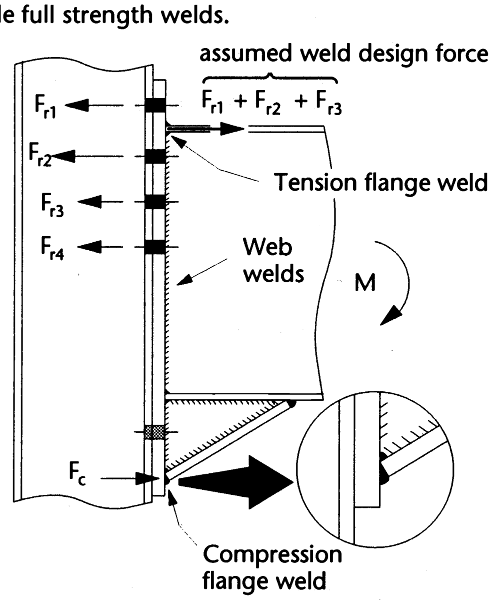


Figure 2.36 Forces in welds

A full strength weld to the tension flange can be achieved by:

- a pair of symmetrically disposed fillet welds, with the sum of the throat thickness equal to the flange thickness, or
- a pair of symmetrically disposed partial penetration butt welds with superimposed fillets, or
- a full penetration butt weld.

If designing a partial penetration butt weld with superimposed fillet, as shown in figure 2.37, note that:

- the weld throat required should be calculated based on the strengths given in BS 5950 Part 1 Table 36. (i.e. 215N/mm^2 for design grade 43 and 255N/mm^2 for design grade 50.)
- the shear and tension stress on the fusion lines should not exceed $0.7p_y$ and $1.0p_y$ respectively. (See BS 5950 Part 1 clause 6.6.5.5.)
- the depth of preparation should be 3mm deeper than the required penetration.
- the angle between the fusion faces for a 'V' preparation should be normally not less than 45°.
- the minimum penetration of $2\sqrt{t}$ specified in BS 5950 Part 1 clause 6.6.6.2 does not apply to the detail shown in figure 2.37.

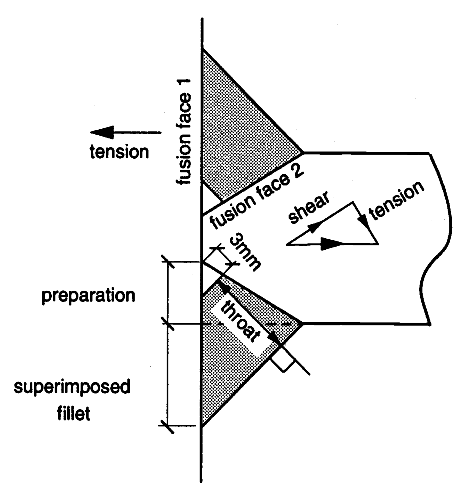


Figure 2.37
A partial penetration butt weld with superimposed fillets

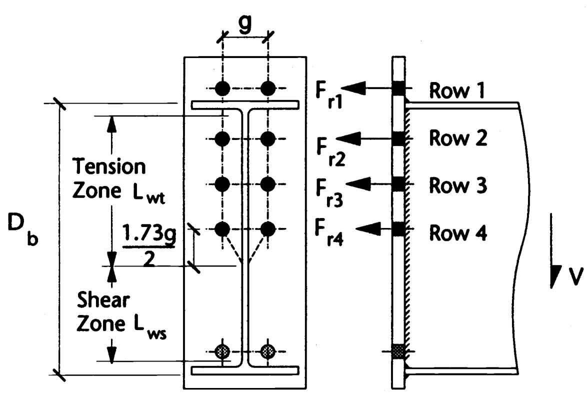
STEP 7 DESIGN OF WELDS (CONTINUED)

Compression flange welds

In cases where the compression flange has a properly sawn end, a bearing fit can be assumed between the flange and end plate and nominal 8mm fillet welds will suffice. For some of the lighter beams (with flange thicknesses of 12mm or less) 6mm fillet welds may be appropriate.

This 'bearing' assumption will be the usual case for most plain beams or for haunches which have been cut from UB's or UC's. Guidance on the necessary tolerances for bearing fit can be found in the NSSS. [10]

If a bearing fit cannot be assumed, or if the haunch is built up from plate as shown in Figure 2.36, then the weld must be designed to carry the full compressive force, F_c.



Note: *The tension zone welds are assumed to start at the bottom of the root radius and must extend down below the bottom bolts resisting tension by a distance of* $\frac{1.73g}{2}$

Figure 2.38 Force distribution in welds

Web welds

It is recommended that web welds in the tension zone **should be full strength.**

For beam webs up to 11.3mm thick, full strength can be achieved with 8mm fillet welds. It is therefore sensible to consider using full strength welds for the full web depth in which case no calculations are needed for tension or shear.

For thicker webs, the welds to the web may be treated in two distinct parts, with a Tension Zone around the bolts which have been dedicated to take tension, and with the rest of the web acting as a Shear Zone as described below:

1. **Tension zone:**

Use full strength welds, i.e. generally fillet welds with the sum of the throat thicknesses not less than the web thickness, t_b.

The full strength welds to the web tension zone should extend below the bottom bolt row resisting tension by a distance of $\frac{1.73g}{2}$ (see Figure 2.38)

2. **Shear zone:**

The capacity of the beam web welds for vertical shear forces should be taken as:

$$P_{sw} = 2 \times a \times p_w \times L_{ws}$$

where:

a = fillet weld throat thickness ($0.7s_w$)

p_w = design strength of fillet weld (BS 5950: Part 1 table 36)

L_{ws} = length of shear zone welds

$= D_b - 2(T_b + r_b) - L_{wt}$

STEP 8 DESIGN OF A HAUNCHED CONNECTION

General

Haunches may be used to:

- provide a longer lever arm for the bolts in tension;
- increase member size over part of its length, in addition to providing a longer lever arm.

Sizing the haunch

The haunch should be proportioned so that welding can be carried out without difficulty, and to ensure that it can resist the bending moment, shear force and axial force in the member. To achieve this the haunch should be arranged with:

- Design grade to match that of the member (or adjust the calculation accordingly)
- 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 2.39.
- when using haunches cut from universal sections, the butting surface of the haunch flange to the end plate to be in accordance with the NSSS[10], which in this situation permits a maximum gap of 1mm. When building up a haunch from plate, bearing contact will generally not be achieved.

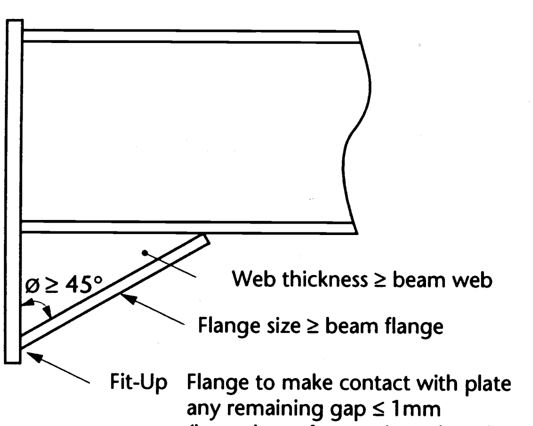


Figure 2.39 Haunch dimensions and fit-up

Haunch design checks

The haunch flange and (when needed) some parts of its web provides the resistance in compression. The lower beam flange is ignored for purposes of calculation. The design of the haunch flange is the STEP 2B compression check. No design checks are needed except that the web of the beam must be checked locally at the sharp end of the haunch for web crushing and buckling.

The distribution of forces in the web and flange of the haunch will depend upon its proportions. For design purposes, it may be conservatively assumed that the compressive force F_c is resolved into the haunch flange.

The beam web is checked for the component of the force normal to the member. See Figure 2.40. The force C_1 is applied to the main member and the checks carried out for bearing and buckling of the web are as those made in Step 2 on the column web. The length of stiff bearing being as shown in Figure 2.40.

Haunch welds

Flange weld to end plate

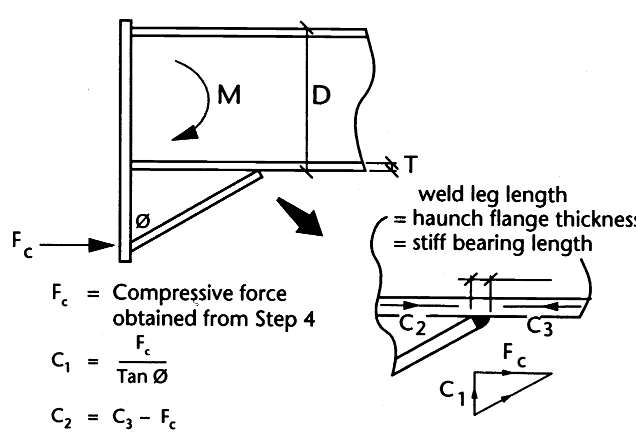
See STEP 7 (Compression flange weld)

Flange weld to main member

It is sufficient to provide a fillet weld with a leg length equal to the flange thickness, as Figure 2.40

Web weld

The force in the web weld can be taken as C_2 as shown in Figure 2.40. Usually 6mm fillets will suffice. Suitably designed intermittent welds may be used where aesthetics and corrosion conditions permit.



F_c = Compressive force obtained from Step 4

$$C_1 = \frac{F_c}{\tan ø}$$

$$C_2 = C_3 - F_c$$

$$C_3 = \frac{M}{D - T}$$

Figure 2.40 Forces in haunch

2.9 ABRIDGED METHOD FOR MANUAL DESIGN

Although the rigorous method is mainly intended as a specification for computer software, it can be adapted for designing connections by hand.

All steps in the rigorous method can be important to the integrity of the connection but, by making a number of simplifications, and by using engineering judgement to omit many checks altogether, the experienced connection designer will be able to achieve reasonable results quite quickly.

The process for manual design is likely to be different from the 'Connection Check' sequence described in Section 2.8. It can be tailored to suit conditions. The worked example which follows proceeds along the lines of the flow diagram in Figure 2.41.

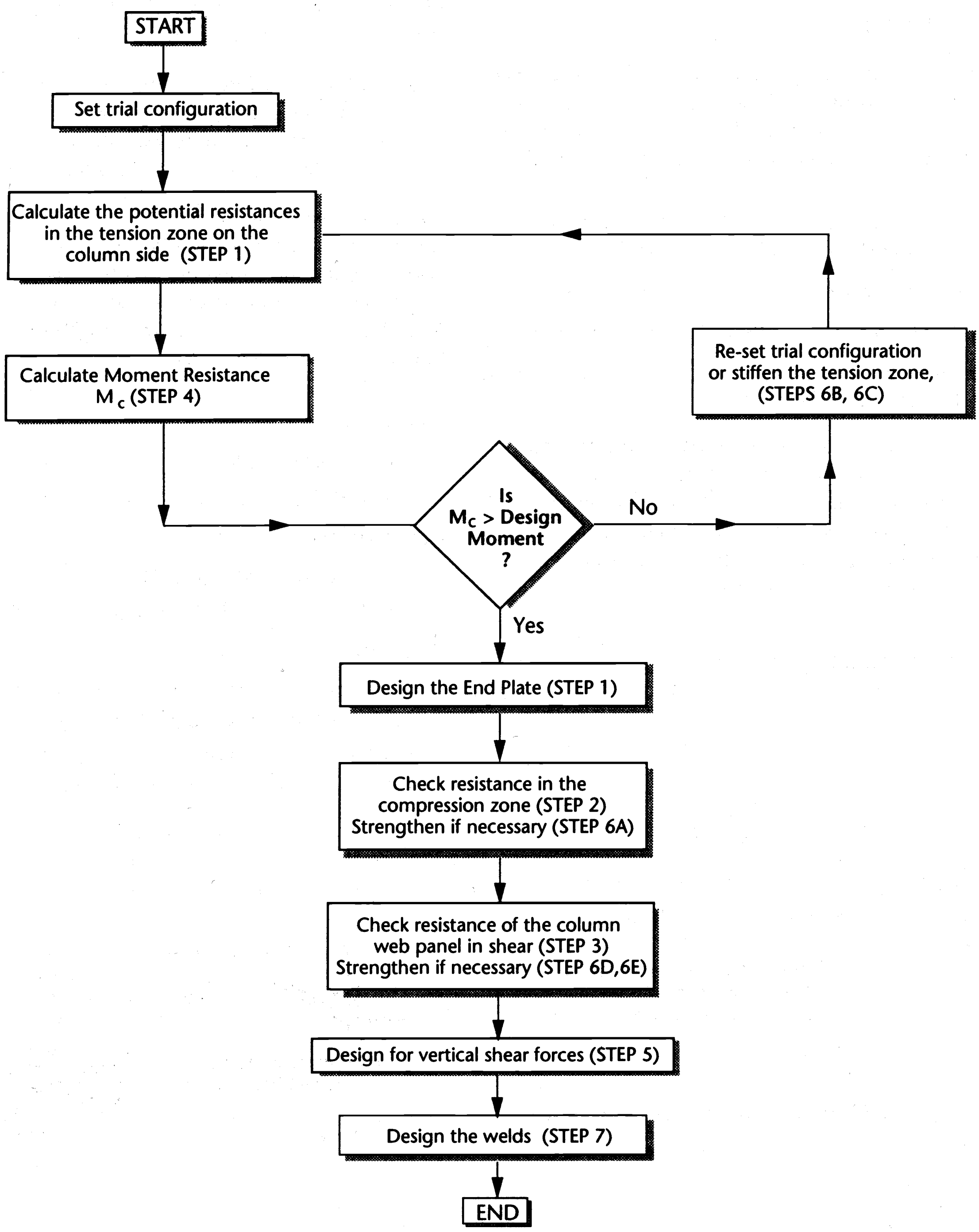


Figure 2.41 Typical flow diagram for abridged manual design

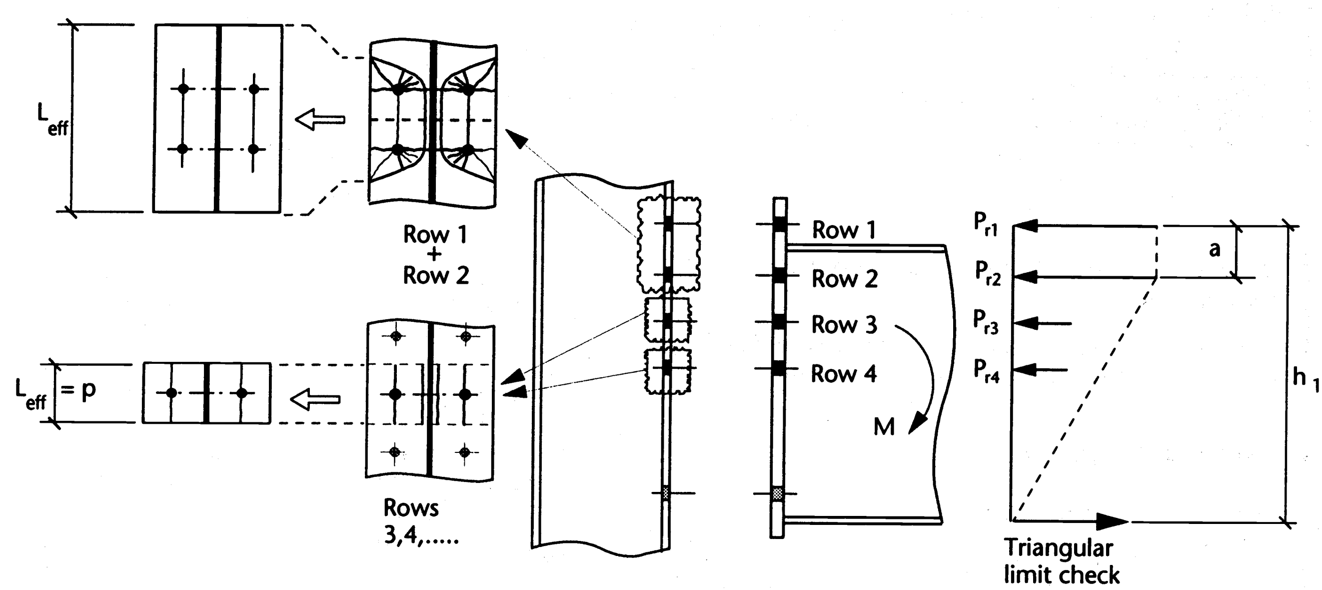
Bolt row force distribution

Most of the short cuts can be taken in the tension zone by assuming a simplified distribution of bolt forces and by sizing the beam end plate so that it is at least as strong as the column flange.

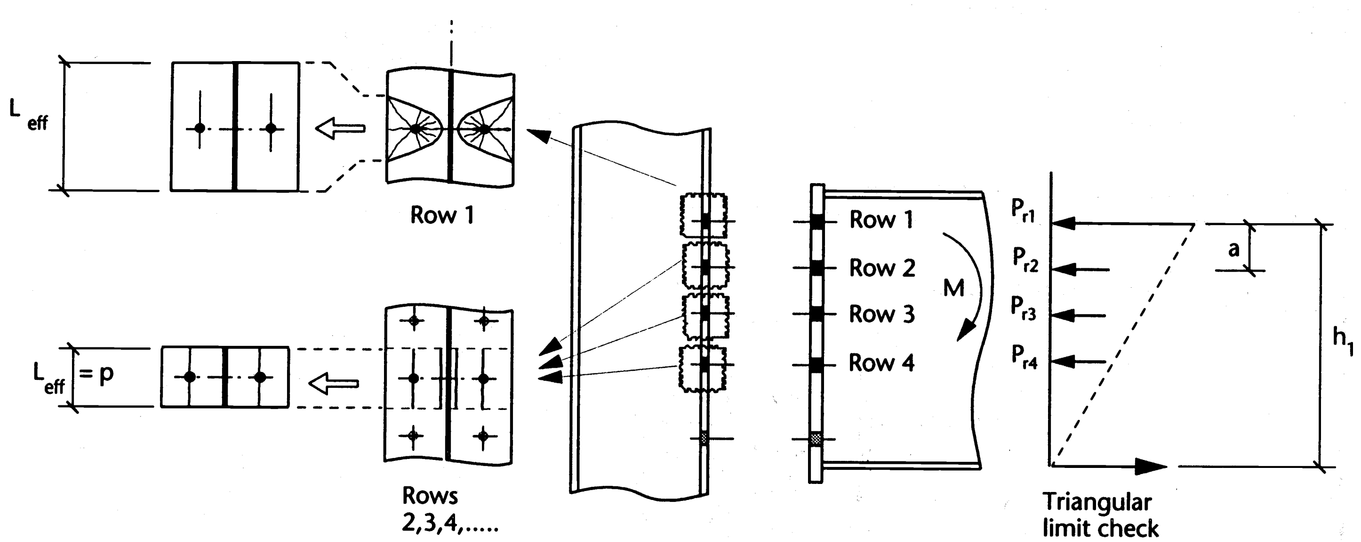
On the column side of an extended end plate connection, the two top bolt rows can be taken as acting together as a group with the combined potential resistance shared equally between rows 1 and 2. Each lower row is then conservatively based on a T-stub length of vertical pitch 'p', but they are also checked to ensure they are within the triangular limit.

A similar approach can be used for flush end plate connections, but only the top bolt row is taken in isolation. (See Figure 2.42 - mechanisms 'A' and 'B'.)

For deep flush end plate connections where $a \leq 0.1\ h_1$, it is reasonable to assume the development of mechanism 'A', with the combined potential resistance shared equally between rows 1 and 2.



Mechanism 'A' : Extended end plate connection



Mechanism 'B' : Flush end plate connection

Figure 2.42 Simplified distribution of bolt row forces

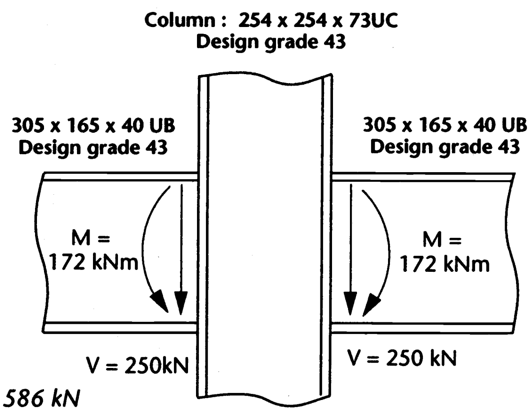
2.10 WORKED EXAMPLE USING THE ABRIDGED METHOD FOR MANUAL DESIGN

In this example a non-standard end plate has been deliberately chosen to illustrate the need for manual calculation when the capacity tables for standard sizes cannot be used. However, the capacity tables are employed to provide a guide to a suitable bolt configuration.

CALCULATION SHEET BCSA	Job *Moment Connections*		Sheet *1 of 6*
	Title *Worked Example - Abridged Method*		
	Client *SCI/BCSA Connections Group*		
	Calcs by *SMCF*	Checked by *ASM*	Date *Apr 95*

Design by hand a bolted moment end plate connection for the following joint.

The connection should be full strength, i.e. capable of carrying the moment capacity of the beam, M_{cx}=172kNm



PRELIMINARY SIZING

Consider an extended endplate,

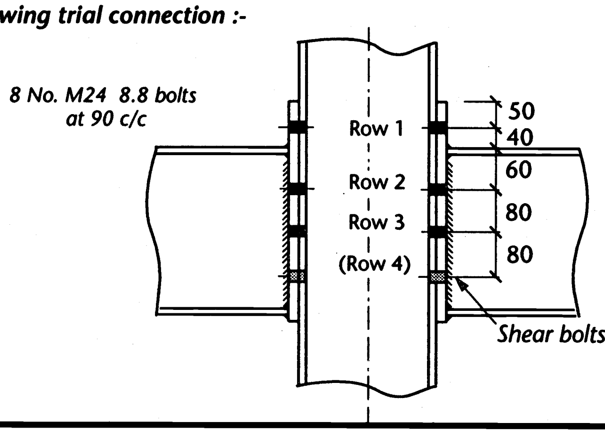
$$C = T = \frac{M}{(D_b - T_b)} = \frac{172 \times 10^3}{(303.8 - 10.2)} = 586 \text{ kN}$$

Capacity Table for 305 x 165 x 40 UB beam shows:
moment capacity of an extended end plate with
3 rows of M20 bolts $\quad = 163$ kNm *(@ 90 gauge & 90 pitch) (page 161)*
$\quad < 172$ kNm $\therefore$ *unsatisfactory*

Try 4 M24 8.8 bolts, $\Sigma P_{t'} = 4 \times 198 = 792$ kN *(Table 2.1)*
$\quad > 586$ kN

Capacity Table for 254 x 254 x 73 UC column shows:
with 3 rows of M24 bolts, $\Sigma F_{ri} = 297 + 215 + 140 = 652$ kN *(@ 100 gauge & 90 pitch) (Page 182)*
$\quad > 586$ kN

So try the following trial connection :-



254 x 254 x 73UC

B_c = 254.0mm
T_c = 14.2mm
t_c = 8.6mm
r_c = 12.7mm

305 x 165 X 40 UB

D_b = 303.8mm
B_b = 165.1mm
T_b = 10.2mm
t_b = 6.1mm
r_b = 8.9mm

Title	Worked Example - Abridged Method	Sheet 2 of 6

TENSION ZONE

STEP 1

Assume a distribution of bolt forces with Rows 1 and 2 acting together as a group with equal distribution of force, and Row 3 based on a T-stub with $L_{eff} = p$

At this stage no checks will be made on the end plate. A suitable plate will be selected later.

For the column :-

$$m = \frac{90}{2} - \frac{8.6}{2} - 0.8 \times 12.7 = 30.5 \text{ mm}$$

$$e = \frac{254.0 - 90}{2} = 82.0 \text{ mm}$$

$$m_p = \frac{1 \times T_c^2}{4} \times p_{yc} = \frac{1 \times 14.2^2 \times 275}{4 \times 10^3} = 13.9 \text{ kNmm/mm of T-stub}$$

For the end plate :- (Assume b_p = 200mm)

$$e = \frac{200 - 90}{2} = 55.0 \text{ mm}$$

and n = minimum [82.0, 55.0, (1.25 x 30.5)] = 38.1 mm

Bolt Rows 1 and 2 combined:

Check column flange yielding :- **STEP 1A**

Table 2.6

$$L_{eff} = 4m + 1.25e + p_{(1\text{-}2)} = 4 \times 30.5 + 1.25 \times 82.0 + 100 = 325\text{mm}$$

$$\text{or} = 2\pi m \times 2 = 2 \times \pi \times 30.5 \times 2 = 383\text{mm}$$

$$M_p = 325 \times 13.9 = 4518 \text{ kNmm}$$

$$P_{r(1+2)} = \frac{4M_p}{m} = \frac{4 \times 4518}{30.5} = 593\text{kN}$$ **Mode 1**

$$\text{or} = \frac{2M_p + n\Sigma P_t'}{(m+n)} = \frac{(2 \times 4518) + (38.1 \times 4 \times 198)}{(30.5 + 38.1)} = 572\text{kN}$$ **Mode 2**

$$\text{or} = \Sigma P_t' = 4 \times 198 = 792 \text{ kN}$$ **Mode 3**

Check column web tension :- **STEP 1B**

$$L_t = 2\left[\frac{90}{2} \times 1.73\right] + 100 = 255.7 \text{ mm}$$

$$P_{r(1+2)} = L_t \times t_c \times p_{yc} = 255.7 \times 8.6 \times 275 \times 10^{-3} = 605 \text{ kN}$$

$$\therefore P_{r(1+2)} = \text{(lower of 572 kN and 605 kN)} = 572 \text{ kN}$$

$$\therefore P_{r1} = P_{r2} = \frac{572}{2} = 286 \text{ kN}$$

(beam web tension not applicable here due to presence of beam flange)

Title	*Worked Example - Abridged Method*	Sheet 3 of 6

Bolt Row 3

STEP 1A

Check for column flange yielding with $L_{eff} = 80\ mm$

$$M_p = 80 \times 13.9 = 1112\ kN\ mm$$

$$P_{r3} = \frac{4M_p}{m} = \frac{4 \times 1112}{30.5} = 146kN \qquad \textbf{Mode 1}$$

$$\text{or} \quad = \frac{2M_p + n\Sigma P_t'}{m+n} = \frac{(2 \times 1112) + (38.1 \times 2 \times 198)}{(30.5 + 38.1)} = 252kN \qquad \textbf{Mode 2}$$

$$\text{or} \quad = \Sigma P_t' = 2 \times 198 = 396kN \qquad \textbf{Mode 3}$$

STEP 1B

Check beam web tension (the beam web is thinner than the column web) :-

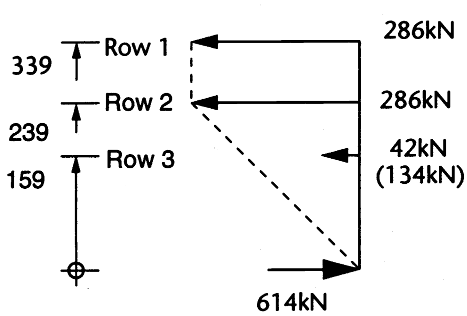
$$L_t = p_{(2\text{-}3)} = 80mm$$

$$P_{r3} = 80 \times 6.1 \times 275 \times 10^{-3} = 134\ kN$$

$$\therefore\ P_{r3} = \text{(lower of 146 kN and 134 kN)} = 134\ kN$$

CALCULATE RESISTANCE MOMENT

STEP 4



$$286 \times 339 \times 10^{-3} = 97.0$$
$$286 \times 239 \times 10^{-3} = 68.4$$
$$134 \times 159 \times 10^{-3} = 21.3$$

$$= 186.7\ kNm > 172\ kNm$$

$\therefore$ *OK, but the full potential of row 3 is not needed:-*

Moment contribution required from Row 3

$$= 172 - (97.0 + 68.4) = 6.6\ kNm$$

$$\therefore \text{ reduced force in Row 3} = \frac{6.6}{159 \times 10^{-3}} = 42\ kN$$

$$\Sigma F_{ti} = F_c = 286 + 286 + 42 = 614\ kN$$

END PLATE DESIGN

STEP 1

Choose an end plate which is a practical size with a thickness equal to or greater than the column flange thickness. When the plate is narrower than the column flange it will usually need to be thicker.

Maximum force per bolt row in column $= 286kN$

Try 200mm wide by 20mm thick plate

Web tension on the beam side need not be checked since Row 2 bolts are adjacent to the flange and Row 3 bolts have been checked above

STEP 1A

Check the plate extension in bending:

Table 2.4(vii)

$$L_{eff} = \frac{b_p}{2} = \frac{200}{2} = 100mm$$

with 8mm tension flange welds $\quad m_x = 40 - (0.8 \times 8) = 33.6\ mm$

$$e_x = 50\ mm$$

$$n_x = \min[\,50, (1.25 \times 33.6)] = 42mm$$

Title	*Worked Example - Abridged Method*	Sheet 4 of 6

$$M_p = \frac{L_{eff} \times t_p^2 \times p_{yp}}{4} = \frac{100 \times 20^2 \times 265 \times 10^{-3}}{4} = 2650 \text{ kNmm}$$

Mode 1

$$P_{r1} = \frac{4M_p}{m_x} = \frac{4 \times 2650}{33.6} = 316 \text{ kN}$$

Mode 2

$$\text{or} = \frac{2\,M_p + n\,(\Sigma P_t')}{m_x + n_x}$$

$$= \frac{(2 \times 2650) + (42 \times 2 \times 198)}{(33.6 + 42)} = 290 \text{ kN}$$

Mode 3

$$\text{or} = \Sigma P_t' = 2 \times 198 = 396\text{kN}$$

critical value of P_{r1} = *290kN > 285kN, OK.*

By Inspection:

- *the end plate design for bolt row 2 will not be critical due to the strengthening effect of the web compared to the end plate extension.*
- *the end plate design for subsequent bolt rows will not be critical as the end plate is thicker than the column flange.*

the selected 200 x 20 end plate is satisfactory.

COMPRESSION ZONE

STEP 2B

On Beam side:

$$P_c = 1.4 \times p_{yb} \times T_b \times B_b$$

$$= 1.4 \times 275 \times 10^{-3} \times 10.2 \times 165.1 = 648\text{kN} > 614\text{kN} \quad \text{OK}$$

STEP 2A

On Column side:

Web crushing(bearing) is usually critical for UC's, so checking crushing first

$$b_1 = 10.2 + 8 + 8 + 20 + 20 = 66.2 \text{ mm}$$

$$n_2 = (14.2 + 12.7) \times 2.5 \times 2 = 134.5 \text{ mm}$$

$$\therefore \quad P_c = (66.2 + 134.5) \times 8.6 \times 275 \times 10^{-3} = 475 \text{ kN} < 614 \text{ kN -Unsatisfactory}$$

Eqn (2.7)

Provide a compression stiffener

COMPRESSION STIFFENER DESIGN

STEP 6A

Column web is over stressed by 614/ 475 = *29%*

by inspection 80% rule will govern

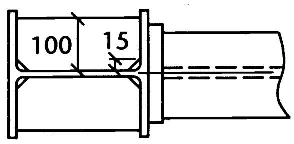
$$P_c = \frac{A_{sn} \times P_{ys}}{0.8} \geq F_c = 614 \text{ kN}$$

$$\text{required} \quad A_{sn} = \frac{614 \times 10^3 \times 0.8}{275} = 1786 \text{ mm}^2$$

Eqn (2.13)

Title *Worked Example - Abridged Method*	Sheet 5 of 6

Try 100 wide stiffeners with 15 corner snipe



$$\therefore\ b_{sn} = 100 - 15 = 85\ mm$$

$$t_s > \frac{1786}{2 \times 85} = 10.5\ mm$$

Use 100 x 12 thick stiffeners

Weld to flanges:

Assuming stiffeners are not fitted, full strength weld required, $s = \dfrac{10.5}{1.4} = 7.5$ ***Use 8 mm FW***

Weld to web:

$$F_c = 614kN$$

$$\text{length of weld} = 4 \times (254 - 2(14.2 + 15)) = 782mm.$$

try 6mm fillet welds, capacity $= 782 \times 0.903 = 706kN > 614kN$ *OK*

Weld capacity table is on page 224

Provide 6mm fillet welds to web

STEP 3

COLUMN WEB PANEL SHEAR

For a balanced two-sided connection, such as this, the check is unnecessary.

However, if the connection was one-sided then:

$$F_v = F_c = \Sigma F_{ti} = 614\ kN$$

Eqn. (2.10)

and $P_v = 0.6 \times p_{yc} \times t_c \times D_c$

$$P_v = 0.6 \times 275 \times 10^{-3} \times 8.6 \times 254.0 = 360\ kN < 614\ kN$$

Unsatisfactory

The web panel is inadequate for a one sided connection. ***Stiffen as shown in Step 6D or 6E.***

STEP 5

VERTICAL SHEAR IN BOLTS

$$\text{Shear capacity} = n_s \times P_{ss} + n_t \times P_{ts}$$

M24 8.8 bolts, 14.2 thick flange, $P_{ss} = 132\ kN$

and $P_{ts} = 0.4 \times 132 = 52.8\ kN$

$\therefore$ *Shear capacity* $= (2 \times 132) + (6 \times 52.8) = 581\ kN > 250kN$ *OK*

Bolt capacity table is on page 221

Title	*Worked Example - Abridged Method*	Sheet 6 of 6

STEP 7

WELDS

Beam Tension Flange :

Provide full strength welds

$$s \geq \frac{T_b}{1.4} = \frac{10.2}{1.4} = 7.3 \text{ mm}$$

Provide 8 mm fillet welds

Beam Web :

For simplicity, provide full strength welds to tension and shear zones

$$s \geq \frac{t_b}{1.4} = \frac{6.1}{1.4} = 4.4 \text{ mm}$$

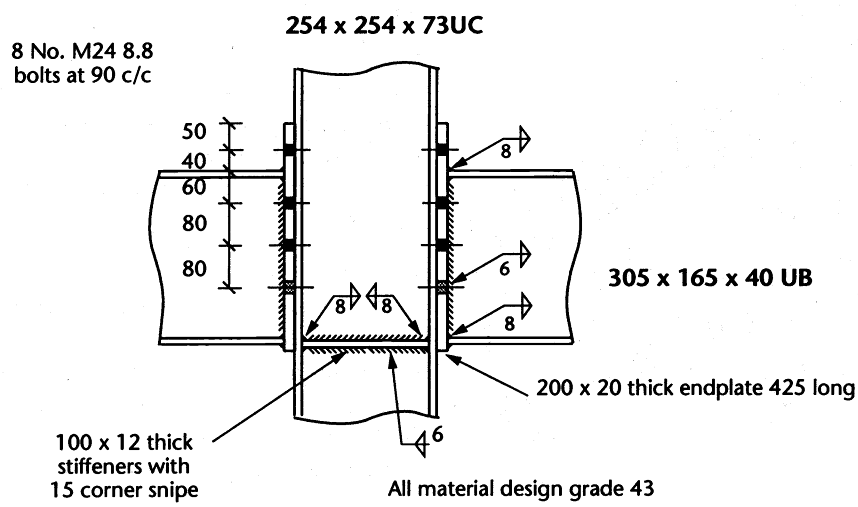
Provide 6 mm fillet welds

Beam Compression flange :

Assume an accurately sawn beam end; therefore the flange is fit for bearing. Nominal welds are required.

Provide 8 mm fillet welds

SUMMARY



3. WIND-MOMENT CONNECTIONS

3.1 INTRODUCTION

The 'wind-moment' method for unbraced frames is well established, having been used to design many of the first multi-storey steel buildings which appeared at the beginning of the 20th century.

Under gravity loads, the connections are assumed to be pinned and the beams are designed by the 'simple' method. Then, for lateral wind loads, the frame is designed as if the joints are rigid with points of contraflexure at the mid point of each column.

The strong axis beam-to-column connections in wind-moment frames are partial strength and generally consist of flush or extended end plates with little or no stiffening in the columns as shown in Figure 3.1.

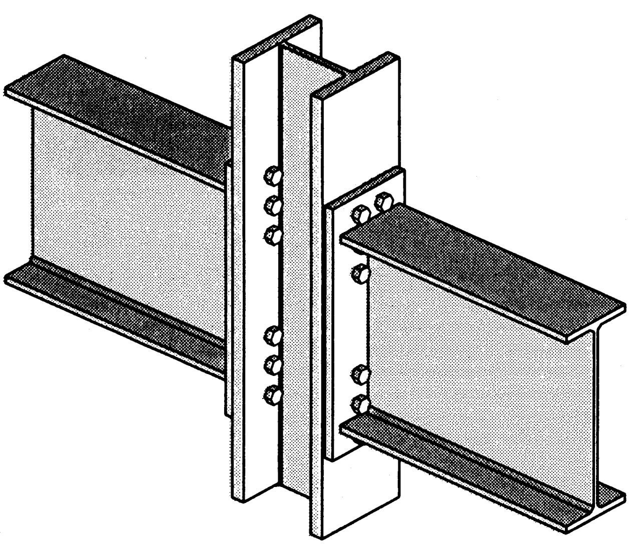


Figure 3.1 Typical wind-moment connection

Because the fabrication remains simple it provides a cost-effective solution for low-rise unbraced buildings.

Detailed rules for designing building frames by this method are given in the SCI publication *Wind-moment design for unbraced frames*[11]. This section gives guidance for designing the strong axis connections to such frames. A set of standard details is provided with full dimensions and capacity tables on pages 205-219.

3.2 DESIGN METHOD

Apart from the obvious need to resist the design forces, the key requirement for wind-moment connections is that they should be **ductile.** In other words, they must be able to rotate as plastic hinges under gravity loading and still retain sufficient strength to withstand the wind moments.

It is important to remember that in many cases the moment induced in the connection by gravity loads will exceed the design wind moment, and the connection will therefore need to rotate plastically until equilibrium is reached. This plastic rotation is likely to take place even under service loads.

Rotation capacity

The actual rotation capacity needed will vary depending on the circumstances but is usually expected not to exceed 0.02 to 0.03 radians (approaching 2°) for wind-moment frames.

End plate connections can achieve this **provided** that the end plate is thin enough to be a 'weak link' relative to the bolts (i.e. mode 1 failure occurs). Bending deformation of the end plate provides ductility to such a large degree that it is considered unnecessary to carry out any checks to quantify it.

It is necessary to ensure that non-ductile failure mechanisms are prevented, especially those involving bolt tension and welds. End plate deformation is not the only ductile failure mechanism, but it is the one which can most easily be controlled by the connection designer. Other ductile mechanisms are column web shear deformation and column flange bending deformation.

End plate thickness

The end plate thickness, in relation to the size and strength of the bolts, must be carefully selected. If it is too thick, the bolts will fail first making the connection non-ductile; if too thin, both stiffness and strength will suffer.

Using grade 8.8 bolts, and the geometry set out in the standard details, it is found that the appropriate end plate thickness is around 60% of the bolt diameter.

3.3 DESIGN RULES

Wind-moments may act in either direction so the connections will normally be symmetrical, i.e. the lower half mirrors the upper half, except that any additional bolts required for vertical shear should be in the lower half.

The connection should be designed for strength using the methods given in Section 2, ensuring adequate rotation capacity by checking that the moment capacity is governed by a ductile mechanism in accordance with Table 3.1.

Guidance on the design of ductile connections can be found in Eurocode 3 Annex J which simply states that adequate rotation capacity is achieved as long as Mode 1 failure controls. Applied literally, this can lead to end plates which could be unacceptably thin for use in wind-moment frames, where connection flexibility reduces the stability and increases service deflections of the frame. The standard connections presented here are designed to cope with these problems and have been verified by testing(13). They can therefore be used as connections in frames designed by the methods given in *Wind-moment design for unbraced frames* (11)

3.4 STANDARD DETAILS

Four standard wind-moment connections are presented in Table 3.2 and these have been designed to provide quick-reference solutions to most practical wind-moment frames. They are based on UB beams and UC columns although some sections at the extremes of the ranges have been excluded. The bolts, end plate thickness and geometry have been chosen to ensure that the connections perform in a ductile manner. The plates must be made from design grade 43 steel.

Dimensionally they have much in common with the standard details of Section 2 for conventional moment connections. Bolt sizes M24 and M20 are offered. The first preference for a wind-moment frame should normally be M24 bolts which are used with a thicker end plate and, therefore, make a stiffer connection. It should be noted that a horizontal bolt spacing of 90mm is maintained here for M24 bolts as well as M20.

Table 3.3 shows the comparative performance of the standard wind-moment connections for a selection of beam sizes. Moment capacity is shown both in kNm and as a percentage of the moment capacity of the beam.

The standard connection details presented are designed to maximize stiffness within the ductility constraint. They feature 'compact' bolt spacings in an end plate whose thickness is approximately 60% of the bolt diameter, and would generally be expected to fail by Mode 2. Their ductile performance has been verified by testing on beams up to and including 686 x 254 Universal Beams(13).

The deeper the beam, the greater the deformation required to achieve the target rotation capacity. At the time of writing (December 1994), there is insufficient test evidence to justify the application of the standard details to beams deeper than 686 x 254 Universal Beams. In the interim, the conservative approach of reducing the end plate thickness to 50% of the bolt diameter could be followed for deeper beams (i.e. 12mm thick plates with M24 bolts). Moment resistance should be calculated using the normal design procedures in Section 2.

Capacity Tables

Capacity Tables for the standard wind-moment connections are included in the yellow pages, with notes on their use. Both flush and extended end plate types are included. Two standard plate widths are offered with both M24 and M20 bolts.

Beam sections which are deeper than 686 Universal Beams are excluded from the tables for the reasons given above.

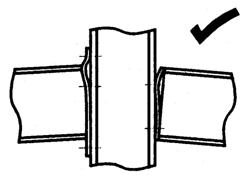
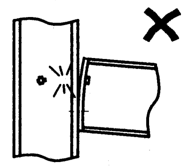
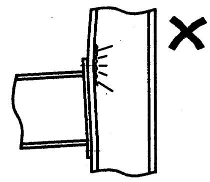
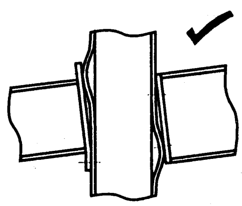
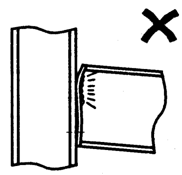
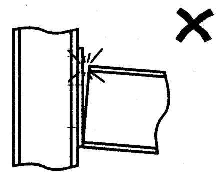
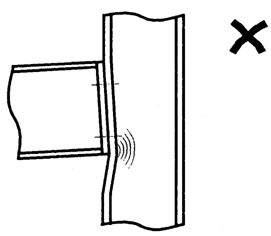
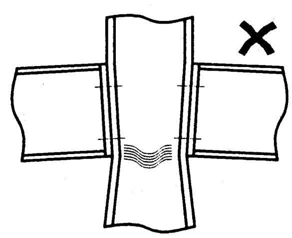
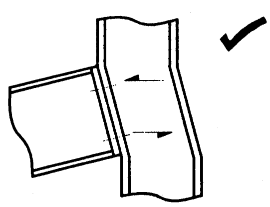
For each connection type the following is provided:

- A table giving the moment capacity of the beam side of the connection calculated according to the procedures in Section 2.
- A table which is a checklist for the column, indicating which UC sections are able to carry the tabulated moments. Where the column is unable to carry the tabulated moment without being stiffened, the table also shows reduced bolt row forces from which a reduced moment can be calculated.
- The vertical shear capacity of the connection.

In practice, many UC sections are capable of accepting most of the standard connections without any strengthening. This is as it should be. It is fundamental to the wind-moment philosophy that the columns should not become costly stiffened fabrications which would be more appropriate to a 'rigid' frame.

In cases where column flange bending is the limiting factor for the design of the connection, ductility is not impaired and a reduced moment resistance may be calculated. It should be noted that this means accepting that the column flange rather than the end plate is deforming - perhaps visibly.

Table 3.1 Wind-moment connections

REGION	CRITICAL FAILURE MECHANISM		
Tension Zone	 End plate bending (Mode 1)	 Bolts in tension (Mode 2 or 3) *See note 1*	 Tension in column web
	 Column flange bending (Mode 1) *See note 2*	 Web welds	 Flange welds
Compression Zone	 Column web crushing	 Column web buckling	
Shear Zone	 Column web shear *See note 3*		

✓ = *Ductile*

✗ = *Non-ductile*

Notes: 1 *Bolt failure is liable to occur if either Mode 2 or 3 is critical for both the column flange and the end plate.*

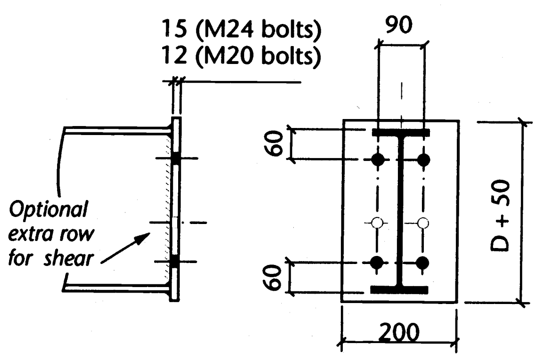
2 *Excessive deformation of the column flange may be unacceptable from an architectural viewpoint, or if there are exceptionally high axial stresses in the column*

3 *Column web shear may be the critical mode for a single sided connection such as a perimeter column. It is **not** acceptable for a two-sided connection where wind reversal can lead to alternating plasticity in the column web.*

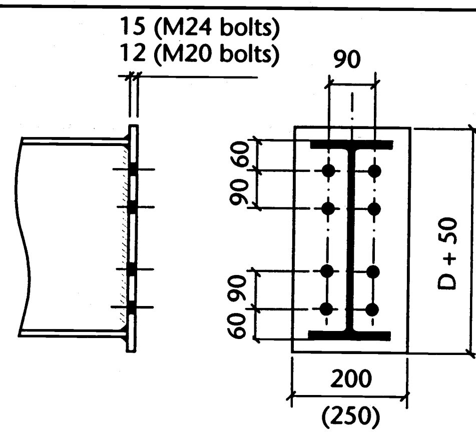
Some warnings:

- Standard wind-moment connections use design grade 43 steel end plates designed to be the weakest element, even though the beams may be in design grade 50 steel. Care must be taken **not** to substitute design grade 50 steel or other excessively over-strong material for the end plates.
- A 'minimum connection' capable of resisting ±20% of the free moment of the beam is recommended. This is intended to avoid alternating plasticity under variable gravity load when the wind moment is relatively small.
- The column checks assume that the column is continuous past the top of the beam. If this is not possible, for example at roof level, then either a cap plate should be provided or a separate check must be carried out on the column capacity. In most cases 100mm above the top bolt row will suffice.
- Dimensions shown in the standard details are, in many cases, critical. Deviations may either reduce the resistance of the connection, compromise its ductility or invalidate the column check.

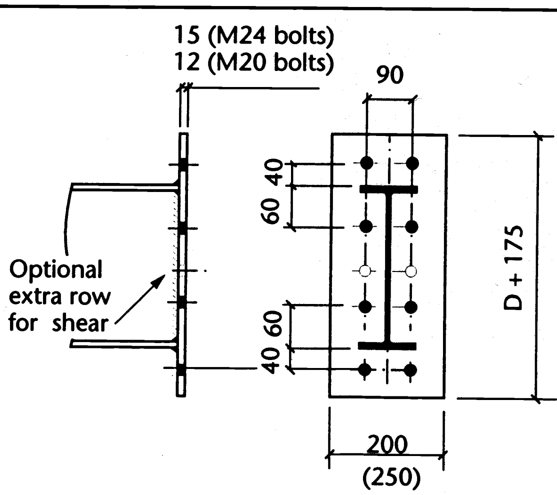
Table 3.2 Standard wind-moment connections



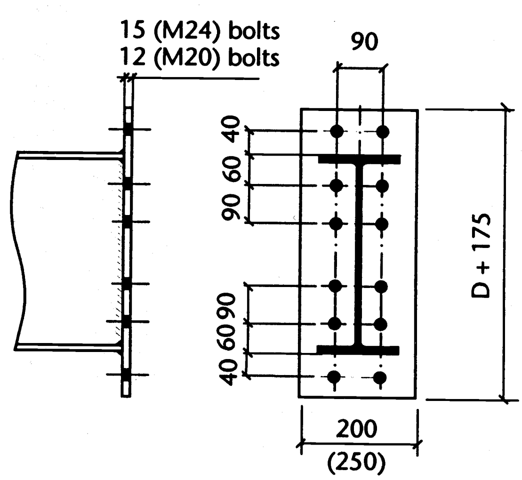
Flush End Plate 1 Row M20 or M24



Flush End Plate 2 Rows M20 or M24



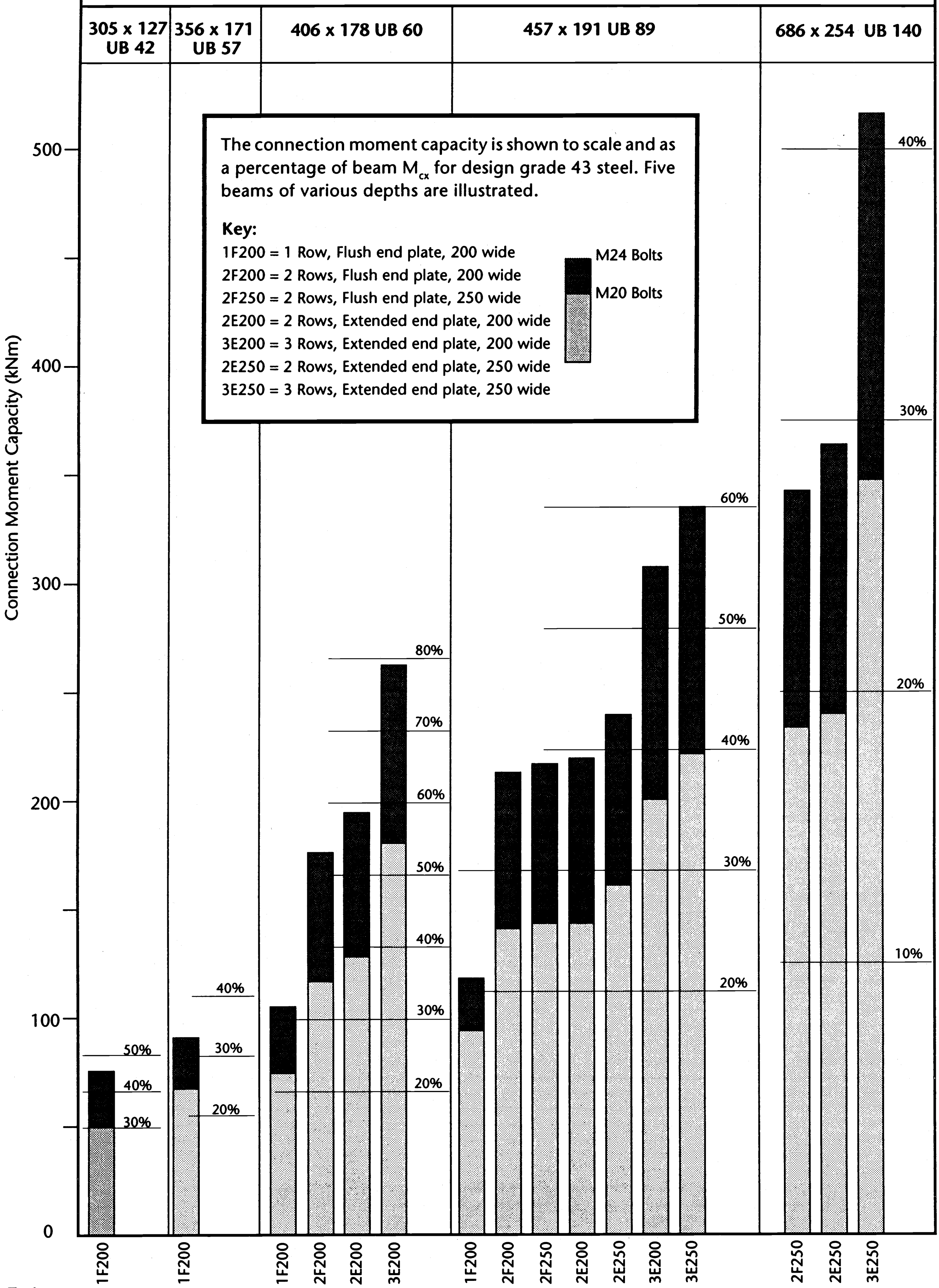
Extended End Plate 2 Rows M20 or M24



Extended End Plate 3 Rows M20 or M24

Notes:
1. *2-Row and 3-Row connections can have 250 wide end plates.*
2. *End plates design grade 43.*
3. *All bolts 8.8.*
4. *Flange to end plate welds to be full strength with a visible fillet of 10mm all round, since this is one of the assumptions of the design method.*
5. *Web to end plate welds to be 8 FW both sides.*

Table 3.3 Comparative performance of the standard wind-moment connections



4. WELDED BEAM TO COLUMN CONNECTIONS

4.1 SCOPE

This Section gives guidance for the design and detailing of welded moment connections. The design rules are modelled around beam to column flange connections, although they can be adapted for other joints such as welded beam or column splices.

Two main types of connections are covered in this publication :

- Shop welded beam-column connections.
- Site welded beam-column connections.

The intention with shop welded construction is to ensure that the main beam-column welds are made in a factory environment. This is achieved by shop welding short stubs of the beam section to the columns. The remainder of the beam is erected separately with a simple site splice at each end as shown in Figure 4.1.

Site welded moment connections are used extensively in America and Japan where continuous unbraced frames are a popular structural solution for buildings in seismic zones. Despite their efficiency for resisting high moments, and suitability for a number of framing systems such as the parallel beam approach[14], site welded moment connections are currently under used in the UK. However, with careful planning and sensible procedures there is no reason why there should not be a place for them in UK construction.

When site welded connections can be accommodated, the beams are prepared in the shop so that the flanges can be welded directly to the column on site as shown in Figure 4.2. Column splices are also site welded, but secondary and tertiary steel may be site bolted.

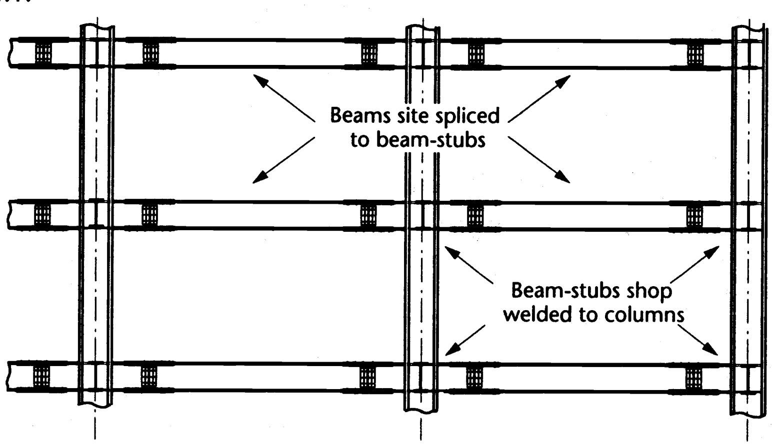


Figure 4.1 Shop welded beam to column connections

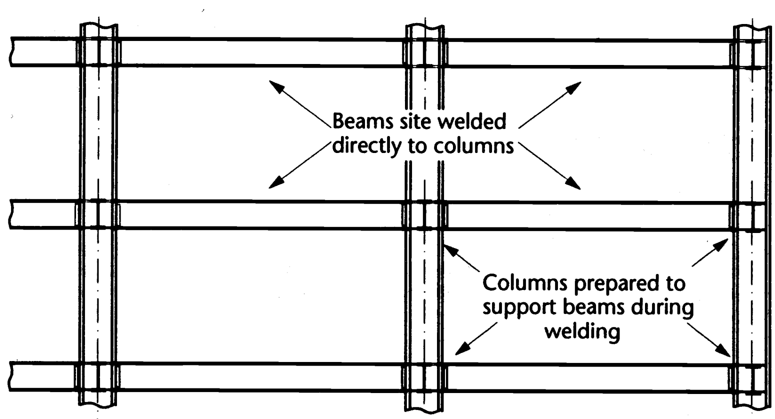


Figure 4.2 Site welded beam to column connections

4.2 SHOP WELDED CONNECTIONS

The shop welded connection, shown in Figure 4.3, consists of a short section beam stub shop welded on to the column flanges, and a tapered stub welded into the column inner profile on the other axes. 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 connections
- all the welding to the column is carried out under controlled conditions
- the workpiece can be turned to avoid or minimize 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 12mm (Figure 4.3).

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 4.4).

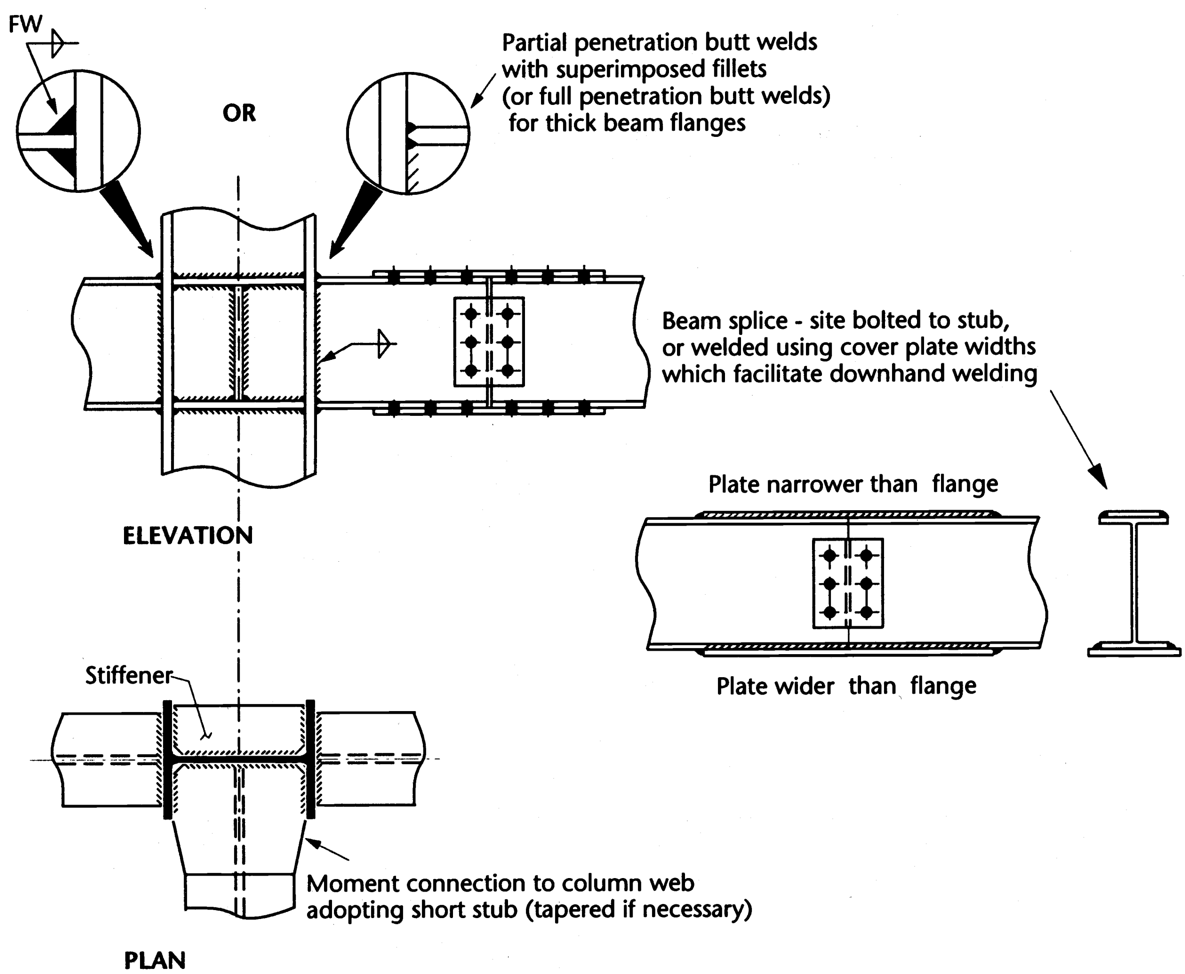


Figure 4.3 Shop welded beam stub connection

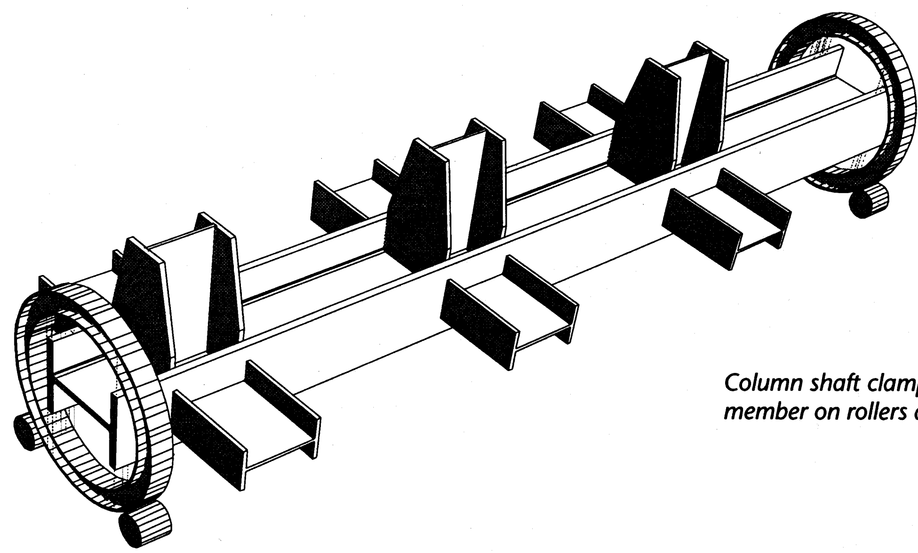


Figure 4.4 Column manipulator for welding beam stubs to columns

4.3 SITE WELDED CONNECTIONS

General

In this type of connection, the beams are prepared in the shop so that the flanges can be welded directly to the column on site using full strength butt welds. The beam web is either butt welded directly to the column, or connected using preloaded ('HSFG') bolts to a fin plate.

Beams framing into the web of the column are connected to vertical and horizontal stiffeners in the column web.

Advantages:

- Relatively cheap shop fabrication and extremely efficient connections.
- A neat solution with aesthetically clean lines.

Disadvantages:

- Care must be taken to provide good working access and weather protection for site welding.
- Ultrasonic (U/S) examination of site butt welds must be carried out.
- High level of site works may be required, although it will often be found that site welding is not on the critical path.
- Some corrosion protective systems can be difficult to make good on site.

Detailed procedures for site welding should be prepared and agreed before work commences. The procedures should include details such as:

- The geometry and tolerance of the weld preparation and fit-up, and any requirement for run-on/run-off plates.
- The weather conditions that would preclude welding, or means of protection to be used.
- The means of access for the welders.
- The amount - if any - of preheat required.
- The weld consumables.
- The number and size of weld runs.
- The specification for the weld testing (see the National Structural Steelwork Specification for Building Construction[10] for scope of visual, MPI and U/S inspection).

It is essential that a temporary working platform is provided for the welders in accordance with Guidance Note GS28[15]. The simplest method is to build a small cubicle around the columns at each floor to provide a platform for welding and inspection, protection from the weather, and a screen to shield others against the weld arc. It can be made by slinging scaffold over the beam, or using purpose-made platforms.

4.3.1 Fully welded connections

In this type of connection, both flanges and the beam web are welded to the column, as shown in Figure 4.5. The flanges are prepared for full penetration butt welds, prepared for down hand welding as shown. Cope holes in the beam web give access for welding and for the backing strips. Small cope holes do not need to be filled, and backing strips may normally remain in position. The flange bevels and web copes will usually be prepared by machine flame cutting. Numerically controlled (NC) coping machines are able to carry out these operations in one pass.

The web should be butt welded to the column flange, using a fin plate or angle cleat as a permanent backing strip. Normally, no preparation of the web is required. The fin plate or angle cleat is used to temporarily support the beam until welding is complete.

After the connections have been site bolted and the frame has been plumbed and lined, the backing strips are tack welded in place ready for applying the butt welds. The strips must be at least 5mm thick to prevent weld blow-through.

It may be necessary to provide run-on/run-off plates which are tack welded into position. Like the backing strips, these plates should be at least equal in steel grade to the beams and are usually left in place after welding.

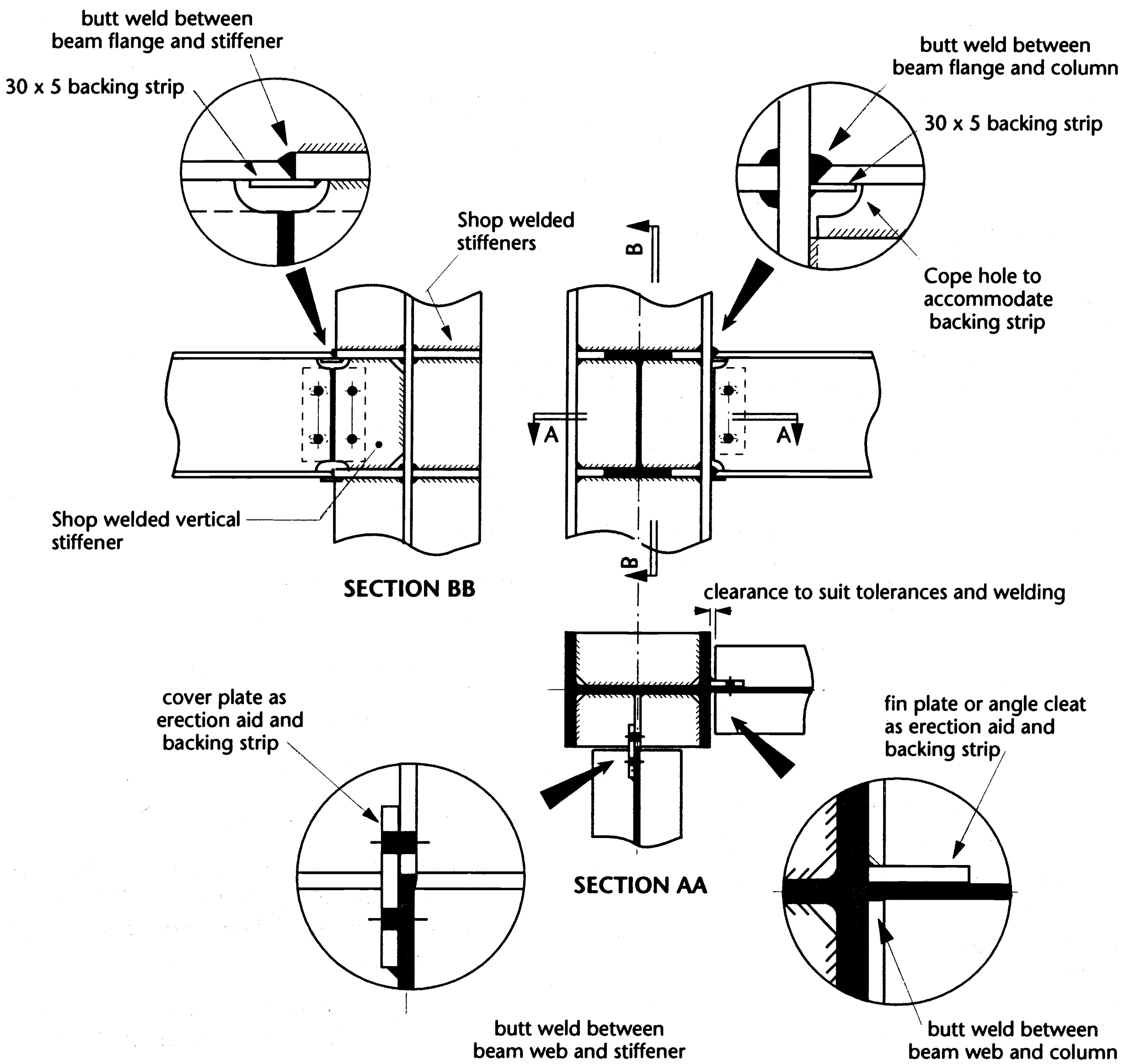


Figure 4.5 Site connection - fully welded

4.3.2 Flange welded, web bolted connections

In this type of connection, shown in Figure 4.6, the beam flanges are prepared and welded in similar fashion to the all-welded connection. The beam web is connected to the column flange by preloaded ('HSFG') bolts to a fin plate.

The fin plate is designed to resist the vertical shear, and is shop welded to the column. The final tightening of the preloaded ("HSFG") bolts must be strictly controlled, and must be carried out only after the flanges have been welded, to allow the welds to shrink without restraint.

Since non-preloaded bolts can only take up load after slip has taken place between the joining surfaces, they cannot be used (other than as temporary erection bolts) in this detail.

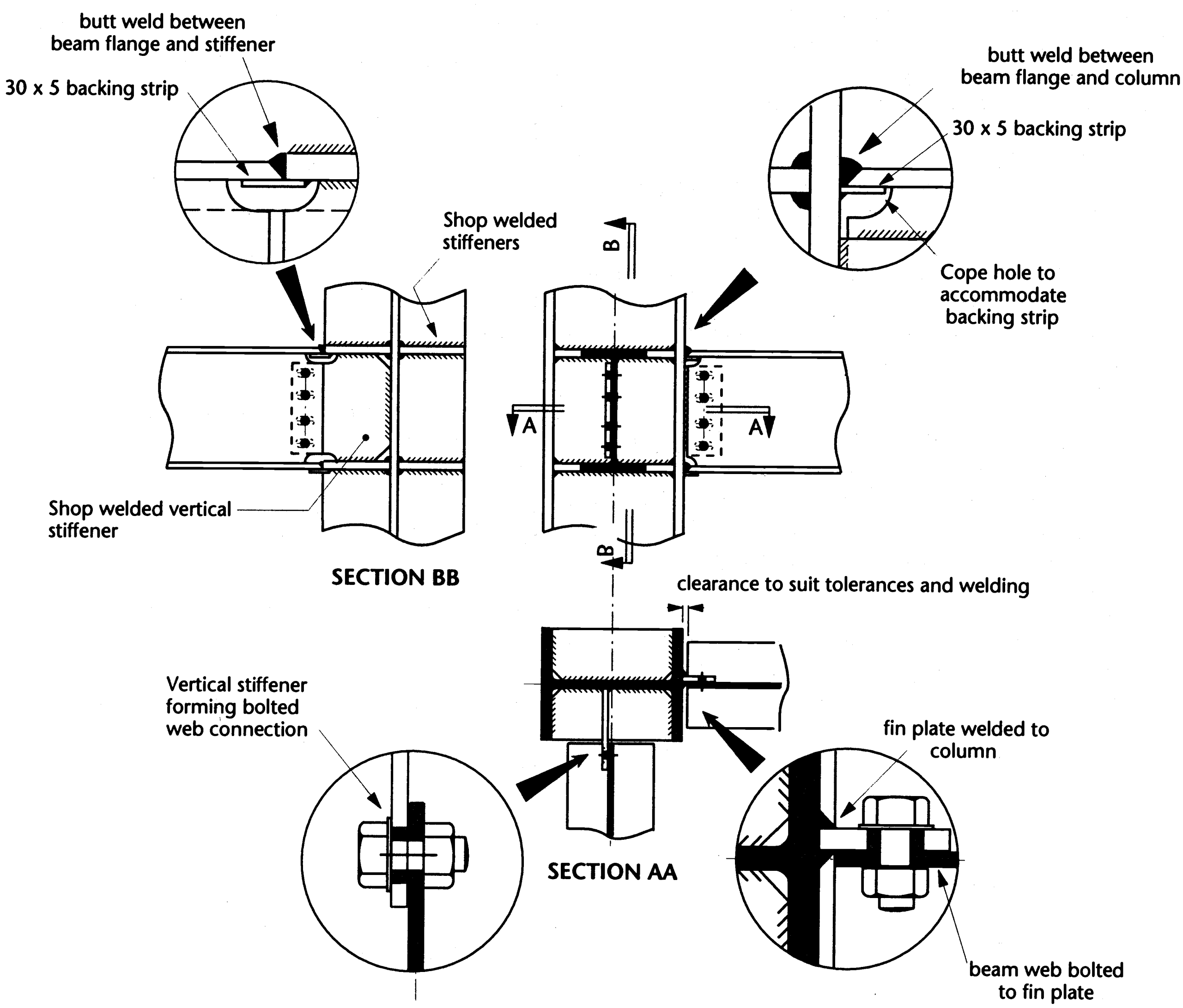


Figure 4.6 Site connection - flanges welded, web bolted

4.4 DESIGN PHILOSOPHY

In statically determinate frames, a partial strength connection designed for the applied 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. To achieve this, the welds in the connection must be made full strength.

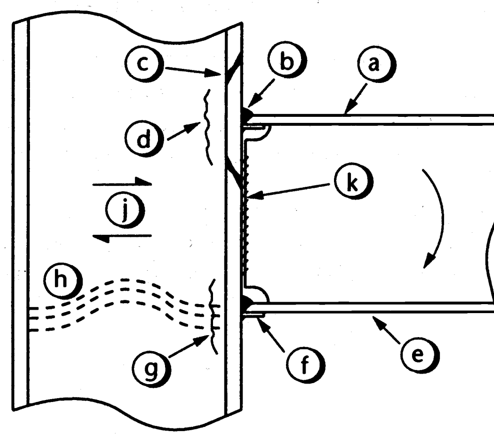
Figure 4.7 illustrates the full set of design checks required which are in accordance with BS 5950.

The distribution of forces in the beam is modelled on the well established assumption that the bending moment is resisted entirely by the beam flanges, with the vertical shear being resisted by the beam web. Any axial load in the beam is shared between the flanges.

American research(16) has shown that under certain conditions, connections modelled with the bending moment resisted entirely by the beam flanges can develop the full plastic moment capacity of the beam. Until this research is validated in the UK, it is recommended that the beam flange stress should be limited to $1.2p_y$, which allows for strain hardening. This rule should be used unless full strength welding is employed for both flanges and web.

A welded flange/bolted web connection designed under these interim rules will therefore not achieve the moment capacity of the beam section in most cases. When a full strength connection is required, the beam web and flanges will have to be fully welded to the column (except for small cope holes).

Fully welded connections made either in the fabrication shop, or directly between beam and column on site, are described in Sections 4.2 and 4.3.



ZONE	*REF.*	*CHECKLIST ITEM*	*See Procedure*
TENSION	*a*	*Beam flange capacity*	*2A*
	b	*Flange weld*	*5*
	c	*Column flange in bending*	*2B*
	d	*Column web in tension*	*2C*
COMPRESSION	*e*	*Beam flange capacity*	*2A*
	f	*Flange weld*	*5*
	g	*Column web crushing*	*3*
	h	*Column web buckling*	*3*
HORIZONTAL SHEAR	*j*	*Column web panel shear*	*4*
VERTICAL SHEAR	*k*	*Fin plate or*	*6*
		direct weld to column	*5*

Figure 4.7
Components of the connection requiring design checks

4.5 DESIGN PROCEDURES

The following procedures present a method for checking the strength of beam to column flange welded connections.

The full set of checks needed is shown in Figure 4.7. They are carried out in sequence in the three zones as shown in Figures 4.8 and 4.9.

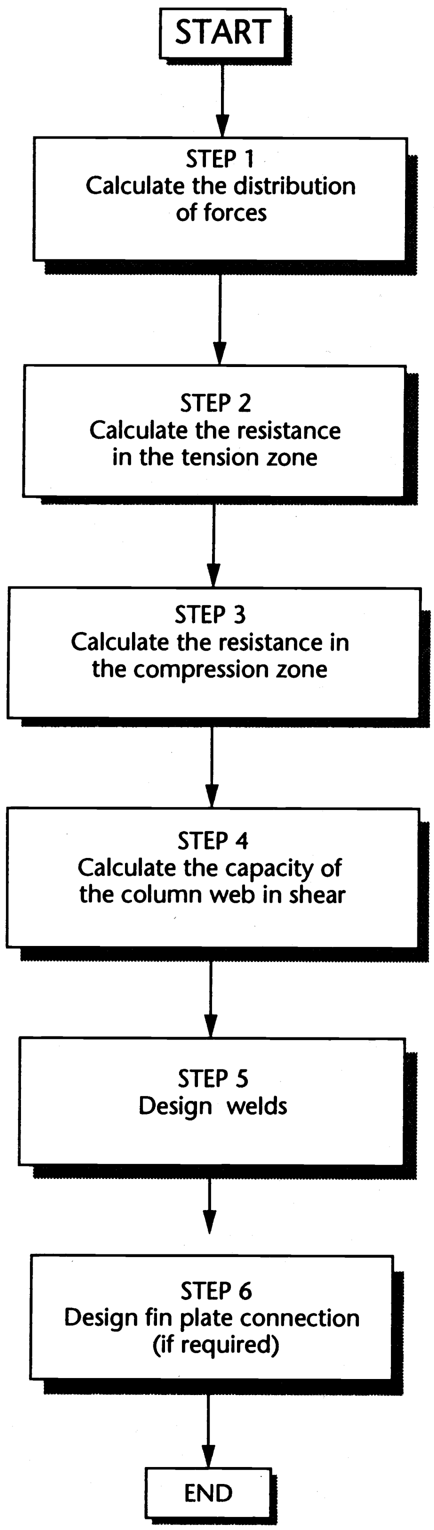


Figure 4.8 Flow diagram - design checks

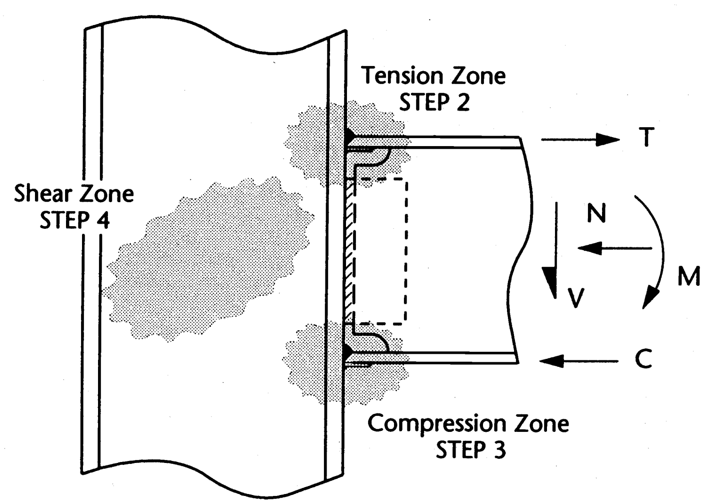


Figure 4.9 Check zones

STEP 1 — DISTRIBUTION OF FORCES IN BEAM

The forces in the beam tension flange 'T' and in the beam compression flange 'C', shown in Figure 4.10, are given by:

$$T = \frac{M}{(D_b - T_b)} - \frac{N}{2}$$

$$C = \frac{M}{(D_b - T_b)} + \frac{N}{2}$$

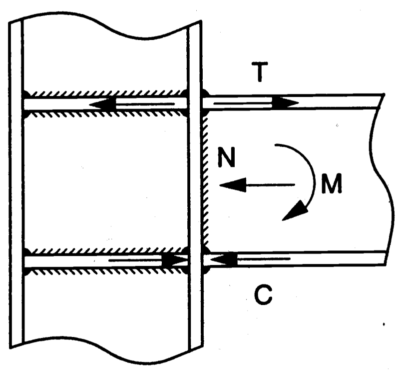


Figure 4.10 Calculation of flange forces

where:

M = design moment

N = axial force in the beam (+ve for compression)

D_b = overall depth of beam section

T_b = beam flange thickness.

STEP 2A — BEAM FLANGE CAPACITY CHECK

This check only applies if the web of the beam is connected with preloaded ('HSFG') bolts to a fin plate; or if $B_b > B_c$.

It is required that:

$P_{tf} \geq T$, and :

$P_{cf} \geq C$

Note. This is an interim rule until further research validation is carried out. It is discussed in Section 4.4.

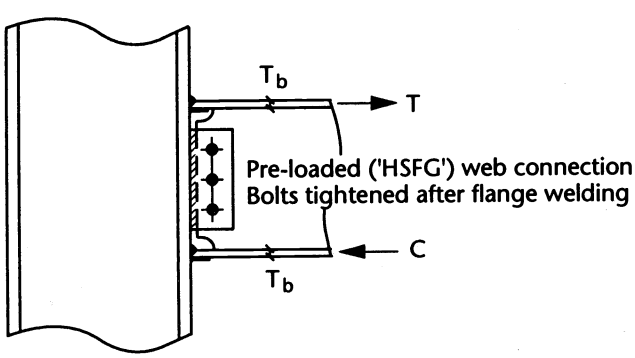


Figure 4.11 Typical connection requiring a STEP 2A check

where:

T = force in tension flange (STEP 1)

C = force in compression flange (STEP 1)

P_{tf} = P_{cf}

= beam flange capacity in tension or compression

= $1.2 \times (\text{minimum } B_b, B_c) \times T_b \times p_{yb}$

B_b = beam flange width

B_c = Column flange width

T_b = beam flange thickness

p_{yb} = design strength of the beam.

If $P_{tf} < T$, or $P_{cf} < C$ then both web and flanges must have full strength welds to the column. (excepting cope holes.)

STEP 2B — RESISTANCE OF COLUMN FLANGE IN BENDING

The column flange is required to resist the beam flange tension force T. See Figure 4.12

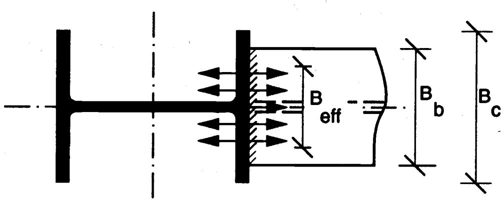


Figure 4.12 Resistance of column flange to tension

If $P_{tc} \geq T$

and $B_{eff} \geq 0.7 \times B_b$

then no column stiffeners are required.

If $P_{tc} < T$

or $B_{eff} < 0.7 \times B_b$

then tension stiffeners are required, such that the stiffener net area and steel grade are not less than that of the beam tension flange, and of similar thickness to the flange, or sized such that:

$$A_{sn} \geq \frac{T}{p_{ys}}$$

P_{tc} = column flange tension capacity

= $B_{eff} \times T_b \times p_{yb}$

B_{eff} = effective flange width, which, for a rolled I or H section

= $t_c + 2r_c + 7T_c$

but ≤ $t_c + 2r_c + 7\left[\frac{T_c}{T_b} \times \frac{p_{yc}}{p_{yb}}\right] \times T_c$

and ≤ B_b, B_c

where:

t_c = column web thickness
r_c = column root radius
T_c = column flange thickness
T_b = beam flange thickness
p_{yc} = design strength of the column
p_{yb} = design strength of the beam
p_{ys} = design strength of the stiffener
A_{sn} = net stiffener area
B_b = beam flange width
B_c = column flange width.

STEP 2C — RESISTANCE OF COLUMN WEB IN TENSION

The spread of tension force T is taken as 1:2.5, as shown in Figure 4.13. When the beam is near an end of the column the effective length of web must be reduced to that available.

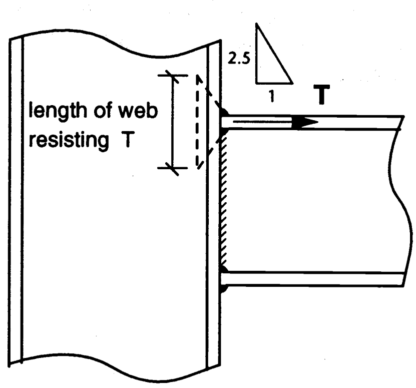


Figure 4.13 Resistance of column web in tension

If $P_t \geq T$

no stiffeners are required.

If $P_t < T$

then a pair of tension stiffeners are required, such that the stiffener net area and steel grade are not less than that of the beam tension flange, and of similar thickness to the flange, or sized such that:

$$A_{sn} \geq \frac{T}{p_{ys}}$$

where:

P_t = tension capacity of the unstiffened column web

= $p_{yc} \times t_c \times [T_b + 2s_f + 5(T_c + r_c)]$

s_f = weld fillet leg length to beam tension flange (when available)

p_{ys} = design strength of the stiffener

A_{sn} = net stiffener area.

STEP 3	COMPRESSION CHECK - COLUMN RESISTANCE OF THE COLUMN WEB IN THE COMPRESSION ZONE

The resistance in the compression zone, P_c is the lesser of the values given by *(2.7)* or *(2.8)* below.

For the resistance of stiffened columns, reference should be made to Section 2 STEP 6A.

Column web crushing (bearing)

An area of web providing resistance to crushing is calculated on the force dispersion length taken from Figure 4.14.
(BS 5950: Part 1 Cl 4.5.3)

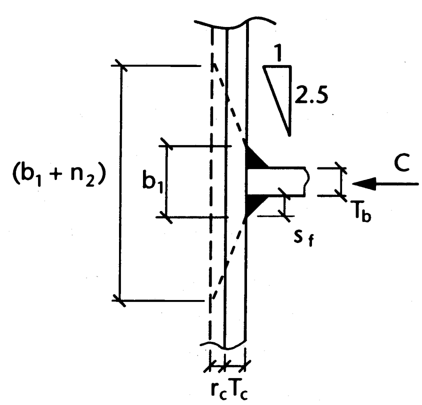


Figure 4.14 Force dispersion for web crushing

$$P_c = (b_1 + n_2) \times t_c \times p_{yc} \qquad (2.7)$$

where:

b_1 = stiff bearing length
  = $T_b + 2s_f$
T_b = beam flange thickness
s_f = fillet weld leg length to beam flange (if available)
n_2 = length obtained by a 1:2.5 dispersion through the column flange and root radius
t_c = column web thickness
p_{yc} = design strength of the column
T_c = column flange thickness
r_c = column root radius.

Note: b_1, n_1, n_2, must be reduced if the column projection is insufficient for full dispersal.

Column web buckling

An area of web providing resistance to buckling is calculated on a web length taken from Figure 4.15.
(BS 5950: Part 1 Cl 4.5.2.1)

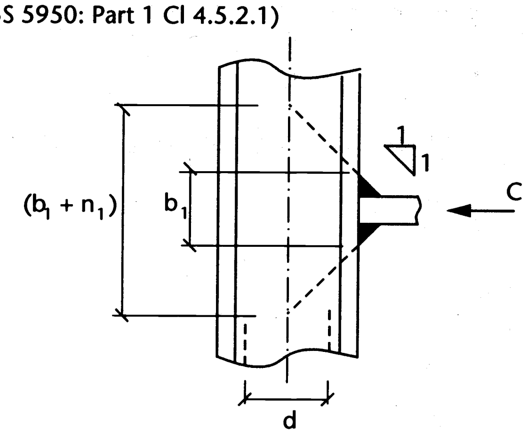


Figure 4.15 Length for web buckling

$$P_c = (b_1 + n_1) \times t_c \times p_c \qquad (2.8)$$

where:

b_1 = stiff bearing length as above
n_1 = length obtained by a 45° dispersion through half the depth of the column
  = column depth (D_c)
t_c = column web thickness
p_c = compressive strength of the column web from BS 5950: Part 1 Table 27(c) with $\lambda = 2.5d/t_c$
d = depth of web between fillets.

The above expression assumes that the column flanges are laterally restrained relative to one another. (BS 5950: Part 1 clause 4.5.2.1). If this is not the case, further reference should be made to BS 5950: Part 1 clause 4.5.1.5 and 4.5.2.1.

STEP 4 RESISTANCE OF THE COLUMN WEB PANEL IN SHEAR

The resultant local panel shear from connections to both column flanges must be taken into account when checking the web. Examples are shown in Figure 4.16.

In a one-sided connection, with no axial force in the beam, the shear in the column web F_V will be equal to the compressive force C.

For a two-sided connection with balanced moments, the panel shear is zero, but in the case of a connection with moments acting in the same direction, as in an unbraced frame, the shears are additive.

For resistance of a stiffened column refer to Section 2 STEPS 6D and 6E.

the basic requirement is:

$$P_v \geq F_v$$

where:

F_v = column web panel shear (See Figure 4.16)

P_v = column web panel shear capacity

= $0.6 \times t_c \times p_{yc} \times D_c$

p_{yc} = design strength of the column

t_c = column web thickness

D_c = column depth.

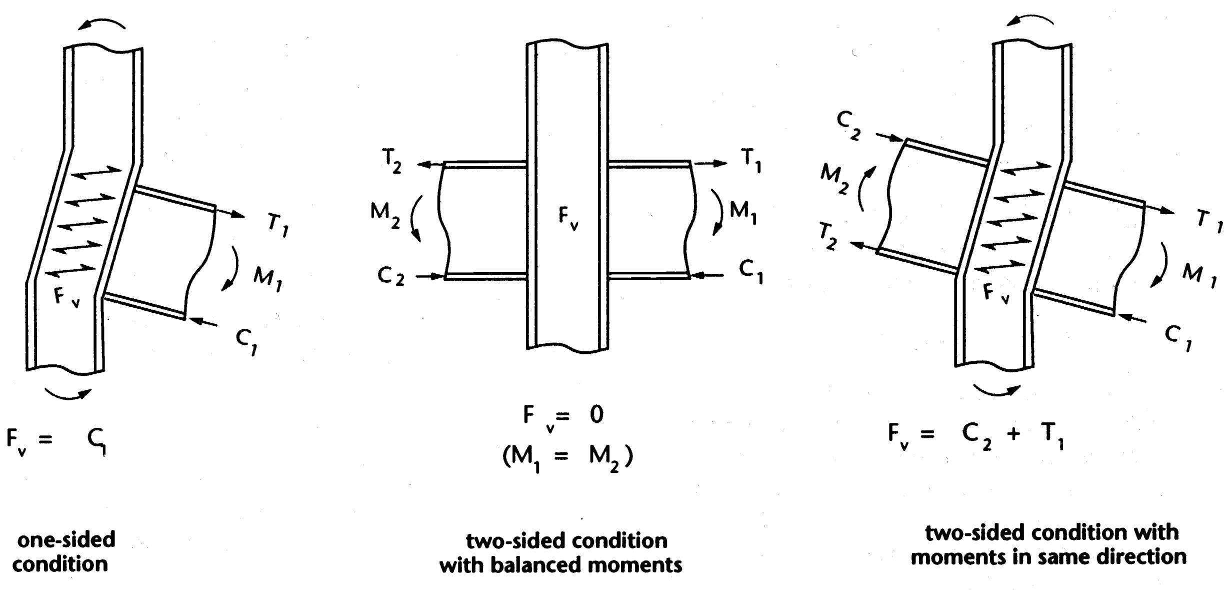


Figure 4.16 Examples of shear conditions in web panel

STEP 5 DESIGN OF WELDS

Statically indeterminate situations

All welds must be full strength welds.

Statically determinate situations

Full strength welds may be provided. Alternatively, the tension and compression flange welds may be designed to resist the flange forces T or C as appropriate, distributed over the effective width B_{eff}. If the column is stiffened, B_{eff} is replaced by the lesser of B_c or B_b.

The web weld may be designed to resist the vertical shear.

Full strength welds

Full strength welds are illustrated in Figure 4.17.

In the fabrication shop, these welds may be achieved by:

- symmetrically disposed fillet welds, where the sum of the throat thicknesses equals the thickness of the element being connected (beam flange or web).
- symmetrically disposed partial penetration butt welds with superimposed fillet welds. These are described in section 2, step 7.
- full penetration butt welds.

Owing to the practical difficulties of overhead welding in the site situation, full penetration butt welds prepared for down-hand welding onto a backing strip should be specified for the flanges. The web weld should be made with a full penetration butt weld using the fin plate or angle cleat as a backing plate.

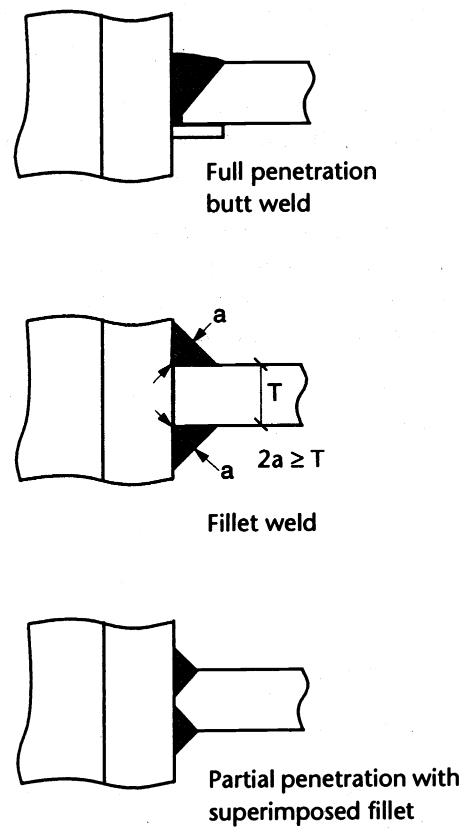


Figure 4.17 Full strength welds

STEP 6 DESIGN OF FIN PLATE CONNECTIONS

A bolted connection for the beam web onto a fin plate is illustrated in Figure 4.18. The design procedures for such plates, including the design of welds connecting the fittings to the column, are given in *Joints in Simple Construction* [9].

The procedure for preloaded ("HSFG") bolts connecting the beam web to the fitting is provided here.

Note that the final tightening of the bolts must be carried out only after completion of the welding.

The basic requirement is:

$$P_{bs} \geq F_r$$

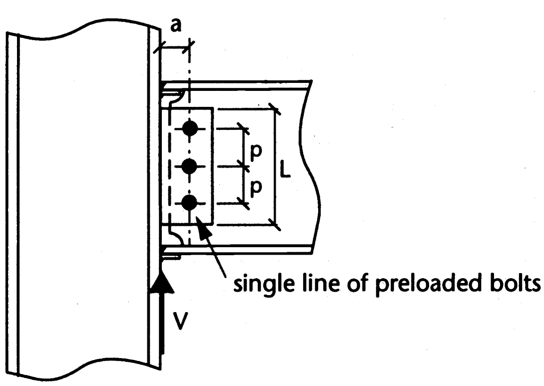


Figure 4.18 Preloaded ("HSFG") bolts to fin plate

where:

F_r = the resultant bolt load

$= \sqrt{(F_m^2 + F_v^2)}$

$F_m = \dfrac{V \times a}{Z_b}$

Z_b = elastic modulus of bolt group

$= \dfrac{n\,(n + 1)\,p}{6}$ (for a single line of bolts)

n = number of bolts

F_v = force on bolt due to direct shear

$= \dfrac{V}{n}$

P_{bs} = the lesser of:

the slip resistance of the bolt per interface

or the bearing capacity of the bolt on the fin plate

or the bearing capacity of the bolt on the web.

4.6 SITE WELDED WORKED EXAMPLE

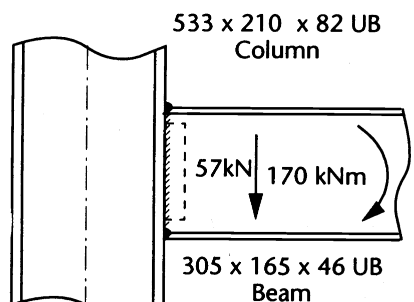
The connection shown in the worked example is subject to a bending moment and shear force where, if it was in a statically determinate situation, partial strength welds would suffice (See STEP 5 procedure). In a statically indeterminate situation full strength fillet welds would provide for design requirements. However, since the welding has to be carried out after erection of the beam, full penetration downhand welds on to backing strips are adopted for either situation.

CALCULATION SHEET BCSA	Job *Moment Connections*		Sheet *1 of 5*
	Title *Worked example of a site welded beam-to-column connection*		
	Client *SCI/BCSA Connections Group*		
	Calcs by *N C*	Checked by *RS*	Date *Apr 95*

Design a site welded connection between a 305 x 165 x 46 UB (Design grade 43) beam and a 533 x 210 x 82 UB (Design grade 43) column. Any column stiffening required is to be carried out in the fabrication shop.

The moment and shear force are ultimate limit state values.

The column flanges are restrained in position by other steelwork not shown.



D_c = 528.3mm D_b = 307.1mm
T_c = 13.2mm T_b = 11.8mm
t_c = 9.6mm t_b = 6.7mm
B_c = 208.7mm B_b = 165.7mm
r_c = 12.7mm
d_c = 476.5mm

DISTRIBUTION OF FORCES

STEP 1

Force T in beam tension flange, and C in beam compression flange.

$$T = C = \frac{M}{(D_b - T_b)} = \frac{170 \times 10^3}{(307.1 - 11.8)} = 576\text{kN}$$

BEAM FLANGE TENSION CAPACITY

STEP 2A

Check not needed since full strength welds to flanges and web will be specified.

Title	Worked example of a site welded beam-to-column connection	Sheet	2 of 5

COLUMN FLANGE IN BENDING

STEP 2B

Basic requirement is:

$P_{tc} \geq T$ and $B_{eff} \geq 0.7 \times B_b$

$B_{eff} = t_c + 2r_c + 7T_c$ but $\leq t_c + 2r_c + 7\left[\frac{T_c}{T_b} \times \frac{p_{yc}}{p_{yb}}\right] T_c$

$= 9.6 + (2 \times 12.7) + (7 \times 13.2) = 127mm$

but $\leq 9.6 + (2 \times 12.7) + 7\left[\frac{13.2}{11.8} \times \frac{275}{275}\right] \times 13.2 = 138mm$

$B_{eff} = 127mm$

$0.7 \times B_b = 0.7 \times 165.7 = 116mm < 127mm$ OK

$P_{tc} = B_{eff} \times T_b \times p_{yb}$

$= 127 \times 11.8 \times 275 \times 10^{-3} = 412kN < 576\ kN$

Therefore the the column flanges must be stiffened

COLUMN WEB IN TENSION

STEP 2C

Note: *Since stiffeners are required for the flanges, this step could be omitted, but is included here for completeness.*

Basic requirement is:

$P_t \geq T$

$P_t = p_{yc} \times t_c \times [\,T_b + 2s_f + 5(\,T_c + r_c)]$

$= 275 \times 9.6 \times [11.8 + 0 + 5(13.2 + 12.7)] \times 10^{-3} = 373kN$

Therefore, $P_t = 373kN < 576kN$

Hence, the web requires strengthening

COLUMN WEB IN COMPRESSION

Note: *Crushing and buckling values can be taken from a properties handbook table, but are calculated here as shown in STEP 3.*

For web crushing, basic requirement is:

STEP 3

$P_c \geq C$

$P_c = (\,b_1 + n_2) \times t_c \times p_{yc}$

$= (11.8 + 5(13.2 + 12.7)) \times 9.6 \times 275 \times 10^{-3}$

$= 373kN < 576kN$

Title	Worked example of a site welded beam-to-column connection	Sheet 3 of 5

For web buckling, basic requirement is: | STEP 3

$$P_c \geq C$$

$$P_c = (b_1 + n_1) \times t_c \times p_c$$

$\left[p_c = 92\text{N/mm}^2 \text{ from BS5950 table 27(c) for } \lambda = \frac{2.5d_c}{t_c} = \frac{2.5 \times 476.5}{9.6} = 124\right]$

$$\therefore P_c = (11.8 + 528.3) \times 9.6 \times 92 \times 10^{-3}$$

$$= 477\text{kN} \qquad < 576\text{kN}$$

The compression zone of the column requires strengthening on both counts.

COLUMN WEB PANEL SHEAR

STEP 4

Basic requirement is:

$$P_v \geq C$$

$$P_v = 0.6 \times p_{yc} \times A_v$$

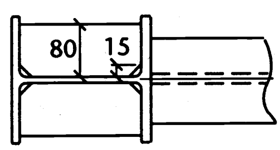
$$A_v = t_c \times D_c = 9.6 \times 528.3 = 5072\text{mm}^2$$

Therefore, $P_v = 0.6 \times 275 \times 5072 \times 10^{-3} = 837\text{kN} \qquad > 576\text{kN OK}$

COMPRESSION STIFFENERS

Section 2
STEP 6A

Try a pair of 80mm wide stiffeners.
(80mm reduces to 65mm with 15mm corner snipe)



By inspection the 80% rule will govern.

Basic requirement is:

$$\frac{A_{sn} \times p_{ys}}{0.8} \geq C$$

Giving, $t_s \geq \frac{0.8 \times C}{2 \times 65 \times p_{ys}} = \frac{0.8 \times 576 \times 10^3}{2 \times 65 \times 275} = 12.9\text{mm, try } 15\text{mm}$

Crushing (bearing) capacity:

$$P_c = P_c \text{ (for unstiffened web)} + (A_{sn} \times p_{ys})$$

$$= 373 + (2 \times 65 \times 15 \times 275 \times 10^{-3}) = 909\text{kN} \qquad > 576\text{kN}$$

Title *Worked example of a site welded beam-to-column connection*	Sheet *4 of 5*

Buckling capacity:

$P_c = (A_w + A_{sg}) \times p_c$

A_w = *allowable area of column web for buckling*

$= 40t_c \times t_c = (40 \times 9.6) \times 9.6 = 3686mm^2$

A_{sg} = *gross area of stiffeners* $= 2 \times 80 \times 15 = 2400mm^2$

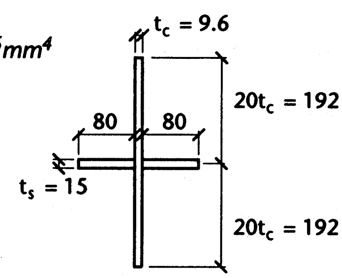
p_c = *compressive strength from table 27(c) of BS 5950, with:*

$\lambda = \dfrac{0.7\,L}{r_y}$

$L = D_c - 2T_c = 528.3 - (2 \times 13.2) = 502mm$

$r_y = \sqrt{\dfrac{I}{A}}$ *(of the section shown opposite)*

$I = \dfrac{15 \times 169.6^3}{12} + \dfrac{384 \times 9.6^3}{12} = 6.12 \times 10^6 mm^4$



$r_y = \sqrt{\dfrac{6.12 \times 10^6}{(3686 + 2400)}} = 32mm$

$\lambda = \dfrac{0.7 \times 502}{32} = 11$

$\therefore p_c = 275N/mm^2$

and, $P_c = (3686 + 2400) \times 275 \times 10^{-3} = 1674kN > 576kN$ *OK*

Therefore use 2 Nº. 80 x 15 stiffeners

Compression stiffener welds:

Weld to flanges:

As these stiffeners will be fitted, provide 6mm fillet weld to flanges

Weld to web:

length of weld $= 4 \times (528.3 - 2(13.2 + 15)) = 1888mm.$

try 6mm fillet welds, capacity $= 1888 \times 0.903 = 1705kN > 576kN$ *OK*

Weld capacity table is on page 224

Provide 6mm fillet welds to web

TENSION STIFFENERS

Provide an area equivalent to the beam flange area and of similar dimensions.

Use 2 Nº. 80 x 15 stiffeners.

(same size as compression stiffeners)

Flange area $= B_b \times T_b = 165.7 \times 11.8 = 1955mm^2$

Effective stiffener area $= (2 \times 65 + 9.6) \times 15 = 2094mm^2 > 1955mm^2$ *OK*

Mar. 97 Revision: A_w *calculation modified*

Title	Sheet
Worked example of a site welded beam-to-column connection	*5 of 5*

Tension stiffener welds

Provide full strength welds to the flange, (based on effective thickness needed, ie beam flange = 11.8mm).

Leg length required per fillet weld,

$$s_f = \frac{11.8}{2 \times 0.7} = 8.4mm,$$ ***therefore use 10mm fillet welds***

Assuming that the total force is transferred to the web via the welds, the load per mm of weld required is:

$$Load/mm = \frac{576}{4 \times (528.3 - 2 \times 13.2 - 2 \times 15)} = 0.31kN/mm,$$ ***therefore use 6mm welds to the web*** *(0.903kN/mm)*

Weld capacity table is on page 224

STEP 5 FLANGE WELDS

Because of the stiffeners, the full width of the beam flange is effective.

Provide a full penetration butt weld using a backing strip to facilitate site welding.

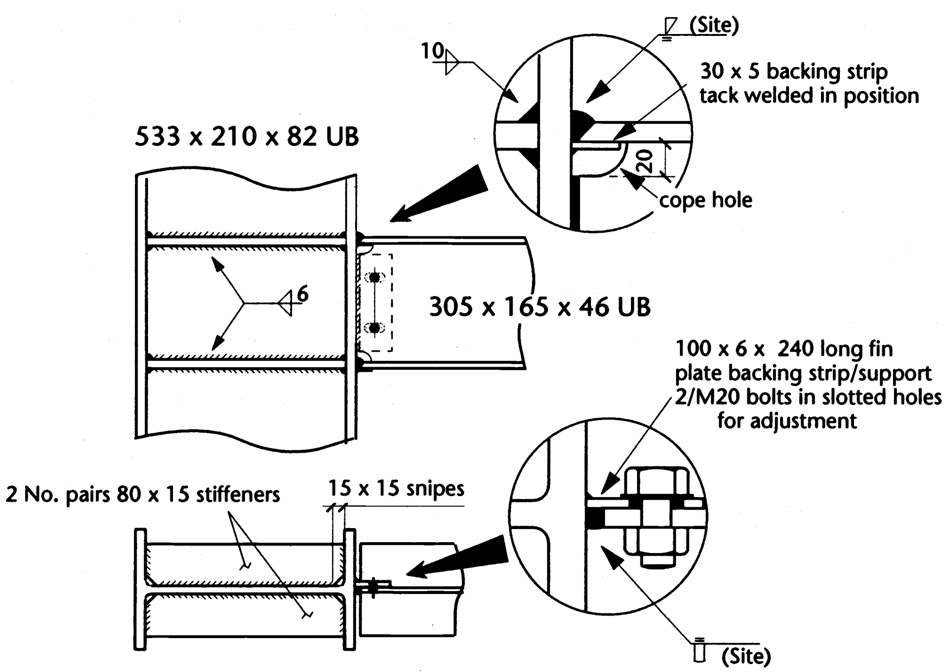
STEP 6 VERTICAL SHEAR

Web welded directly to column

A full penetration butt weld will be provided, using a fin plate which acts both as a backing strip to facilitate site welding and provides a temporary support for the beam until welding is complete.

Use 100 x 6 plate of length equal to the depth between the cope holes, and weld in position on far side only.

The final connection is thus:



5. SPLICES

5.1 SCOPE

This section deals with the design of beam and column splices subjected to bending moment in addition to axial force and transverse shear force.

Both bolted and welded splices are considered, assuming the bending moment is resisted by the flanges alone. An example of a deep plate girder splice designed with the web cover plates sharing in the transfer of moment can be found in *Structural Steelwork Connections*[17].

The procedures for column splices subject to dominant compressive forces, where bolt slip is not a factor, are dealt with in *Joints in Simple Construction*[9]

5.2 BOLTED COVER PLATE SPLICES

5.2.1 Connection details

Typical bolted cover plate splice arrangements are shown in Figure 5.1.

It is generally the case that joint rotation within a beam splice as a result of bolt slip is both visually and functionally unacceptable. Rotation at the splice may also invalidate the frame analysis where continuity has been assumed.

It is therefore recommended that friction grip bolted connections are used in bolted cover plate splices.

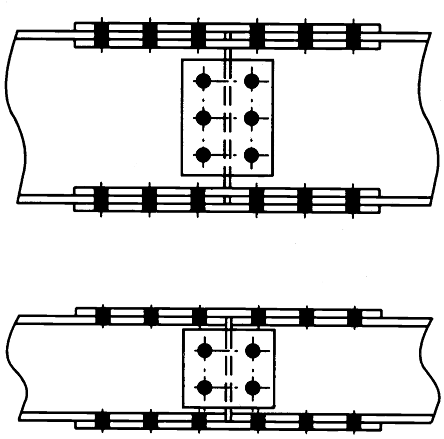
Bolt slip can also be minimized by the use of fitted precision bolts in close tolerance holes, but this is expensive to produce in the fabrication shop and difficult to assemble on site.

It is usual to place the same number of bolts in each set of flange cover plates. However, in beam splices not subject to reversal, the compression flange force may be transferred in direct bearing with a reduced number of bolts. Bearing contact must be achieved to the tolerance specified in the *NSSS*[10]. The designer must, however, consider the requirements for continuity about both axes, and any moment due to strut action (see Section 5.2.2).

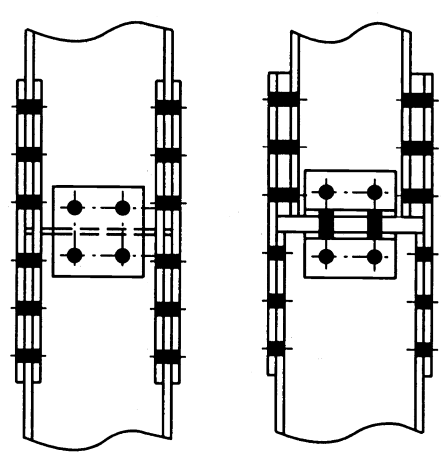
When bearing contact has been assumed in design, the fabrication details must clearly show that a bearing contact is required, and the site erection procedure must ensure the members are brought into contact before tensioning the bolts.

Bolted cover plate splices can be used over the whole range of section sizes, and are not always required to be full strength. For these reasons no formal standard sizes or details are recommended. M24 or M20 should be the first choice for bolt assemblies.

Fastener spacing and edge distances should comply with Clauses 6.2 and 6.4 of BS 5950: Part 1.



Beam splices



Column splices

Figure 5.1 Typical bolted cover plate splices

5.2.2 Design philosophy

The design philosophy for bolted cover plate splices subject to moment is:

- The applied moment is resisted by the flange plates.
- Transverse shear is resisted by the web plates.
- Any axial force in beams is divided equally between the flange plates.

The designer should note that bolt holes in the flanges may prohibit the development of a full strength connection at that point.

An unrestrained member in bending must also be capable of resisting a weak axis moment in its length due to lateral torsional buckling effects. Similarly, axial compression gives rise to a weak axis design moment between points of restraint. When a splice is located away from a point of restraint, account has to be taken of this moment which can be calculated from the strut action formula given in Appendix C3 of BS 5950: Part 1.

5.2.3 Capacity checks

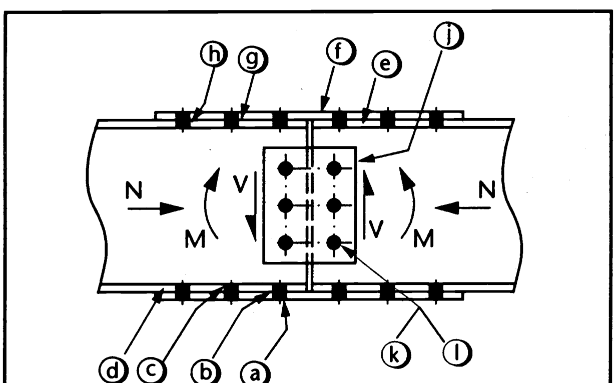
Figure 5.2 illustrates three sets of checks to be made on the member, splice plates and bolts. Each of these checks is outlined in detail in the procedures in Section 5.2.5. If member centrelines do not coincide, the additional forces arising from the eccentricity of force should be included in design.

5.2.4 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 225mm, on either side of the splice.

5.3 DESIGN PROCEDURES

Figure 5.3 presents the design sequence for a bolted splice in the form of a flow chart. The procedures are given on the following pages.



ZONE	REF	CHECKLIST ITEM	See STEP
Tension	a	Flange plate(s) capacity	2
	b	Bolt shear	3
	c	Bolt bearing	3
	d	Flange capacity	2
Compression	e	Member flange compression	2
	f	Flange plate(s) capacity	2
	g	Bolt shear	3
	h	Bolt bearing	3
Shear	j	Web plate(s) capacity	4
	k	Bolt shear	4
	l	Bolt bearing	4

Figure 5.2 Design checks for bolted splices

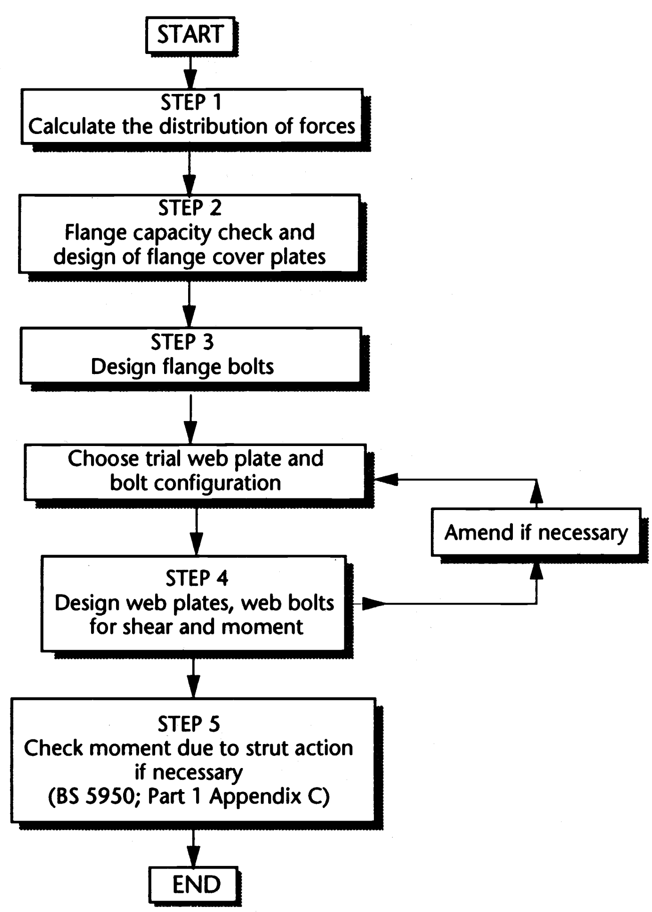


Figure 5.3 Design sequence

STEP 1 DISTRIBUTION OF FORCES IN MEMBER FLANGES

The forces in the member tension flange 'T' and in the member compression flange 'C', shown in Figure 5.4, are given by:

$$T = \frac{M}{(D_b - T_b)} - \frac{N}{2}$$

$$C = \frac{M}{(D_b - T_b)} + \frac{N}{2}$$

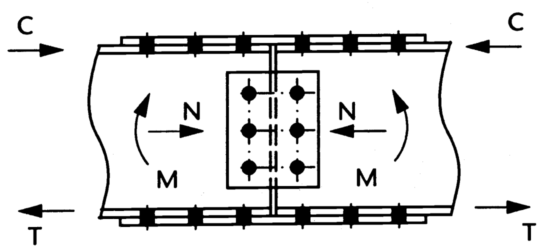


Figure 5.4 Calculation of flange forces

where:

M = design moment

N = axial force in the member (+ve for compression)

D_b = overall depth of member

T_b = member flange thickness

A_f = area of the member flange
= $B_b \times T_b$

B_b = member flange width

A_b = member cross-sectional area
(UB table pages 229-231)
(UC table page 232)

STEP 2 FLANGE CAPACITY CHECK AND DESIGN OF FLANGE PLATES

These procedures are for friction grip connections using preloaded bolts. If the designer considers the circumstances allow the use of ordinary bolt assemblies the detail checks should be adjusted accordingly.

Flange capacity

This check ensures that, where holes occur in the member flanges, the effective area satisfies the requirements of BS 5950: Part 1, clause 3.3.3. The basic requirement is:

$$A_{ef} \geq \frac{F_f}{p_{ys}}$$

Flange plates

The effective area of the cover plates must also be in accordance with the requirements of BS 5950: Part 1, clause 3.3.3. The basic requirement is:

$$A_{ep} \geq \frac{F_f}{p_{yp}}$$

where:

A_{ef} = the effective flange area
= K_e x net area of flange after deduction of holes,
but ≤ gross area

A_{ep} = the effective area of the flange plate (or plates if both internal and external plates are used).
= K_e x net area of plate(s) after deduction of holes,
but ≤ gross area

K_e = 1.2 for design grade 43

K_e = 1.1 for design grade 50

F_f = the force in the flange
= C or T as appropriate (STEP 1)

p_{ys} = the design strength of the section

p_{yp} = the design strength of the plate.

Note: The use of inner and outer splice plates may reduce the number of bolts required. The effective area can be conservatively taken as taken as twice the effective area of the inner plates and the plates made the same thickness.

STEP 3 DESIGN OF FLANGE BOLTS

Design of friction grip connection

It is required that:

$$F_f \leq n_b \times P_s$$

where:

F_f = the force in the flange

= C or T as appropriate (STEP 1)

n_b = the number of bolts per side of the joint

P_s = the lesser of:

the slip resistance of the bolt per interface

or the bearing capacity of the bolt in the flange plate(s)

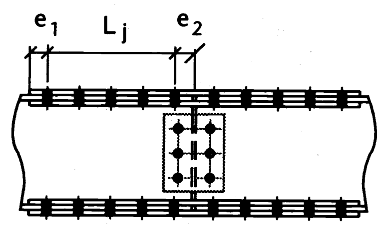
or the bearing capacity of the bolt in the flange.

The slip resistance of the bolt

$= 1.1 K_S \mu P_o$

where n_p = number of flange plates (1 or 2)

See page 222 for slip resistance and bearing values.



Note:

- For friction grip connections, the slip resistance/ shear capacity should be reduced if the length of the bolted connection each side of the splice, L_j exceeds 500mm (BS 5950: Part 1, clause 6.3.4 and 6.4.2.3).
- If end distance e_1 or e_2 are less than 3d, the bearing capacity of the affected pair of bolts should be reduced (BS 5950: Part 1, clause 6.4.2.2).

  Where: d = bolt diameter
- In friction grip connections, the outer plies (in this case the cover plates) should not be thinner than d/2 or 10mm, whichever is less. This requirement is contained in BS 4604.

STEP 4 DESIGN OF WEB PLATES AND BOLTS

Web plates and their fasteners are designed to resist the vertical shear force V only, but account must be taken of the moment in the bolt group as in Figure 5.5. A single line of bolts each side of the splice is considered here.

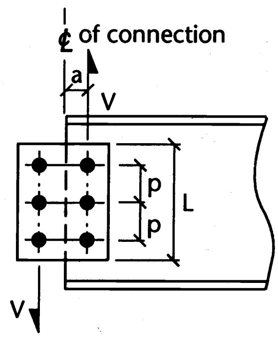


Figure 5.5 Eccentricity of bolt row

Web plate design

It is required that:

$$V \leq P_v$$

$$M \leq M_c$$

where:

V = applied shear force

P_v = shear capacity of the web plates

= $0.6 \times A_{vnet} \times p_{yp}$

= $0.6 \times 0.9 \times (L - n_r \times d_h) \times t_p \times p_{yp} \times n_p$

M = applied moment

= $V \times a$

M_c = $p_{yp} \times Z_{net}$

Z_{net} = is section modulus of plates minus holes

n_r = number of bolt rows

d_h = hole diameter

n_p = the number of web plates (normally 2)

p_{yp} = design strength of the web plate

t_p = thickness of web plate

($t_p \geq 10$mm or d/2 (whichever is less) for friction grip connections.)

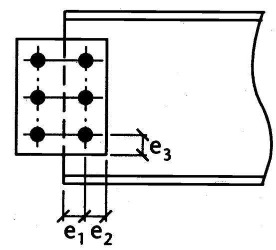


Figure 5.6 End and edge distances critical for bearing

Design of friction grip connection

It is required that:

$$F_r \leq P_s$$

where:

F_r = the resultant bolt load

= $\sqrt{(F_m^2 + F_v^2)}$

F_m = $\dfrac{V \times a}{Z_b}$

Z_b = elastic modulus of bolt group

= $\dfrac{n_r(n_r + 1)p}{6}$ (for a single line of bolts)

F_v = force on bolt due to direct shear

= $\dfrac{V}{n_b}$

n_b = the number of bolts per side of the joint

P_s = the lesser of:
the slip resistance of the bolt per interface (See STEP 3)

or the bearing capacity of the bolt in the cover plate(s)

or the bearing capacity of the bolt in the web.

Note:

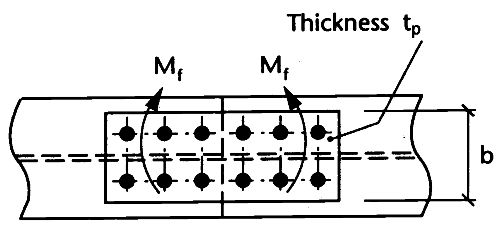
If any end distances e_1 e_2 and e_3 shown in Figure 5.6 are less than 3d, the resultant end distances should be calaculated and the bearing capacity reduced if necessary (BS 5950: Part 1, clause 6.4.2.2).

Where: d = bolt diameter

STEP 5 DESIGN FOR MOMENT FROM STRUT ACTION (ALSO APPLICABLE TO BIAXIAL BENDING)

When a splice is located away from a point of restraint, this check is required to ensure that moments generated by strut action can be resisted. (Figure 5.7)

Derivation of the applied moment M_{yy} is from Appendix C3, BS 5950: Part 1 and is illustrated in *The Steel Designers' Manual*[(18)]



$$M_f = \text{moment in each flange} = \frac{M_{yy}}{2}$$

Figure 5.7 Weak axis moment to be considered

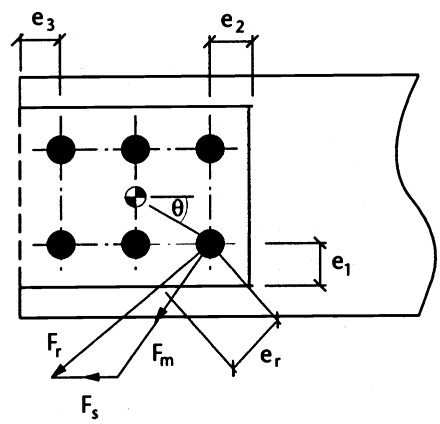


Figure 5.8 Maximum bolt force

Flange plates

This check is not necessary if the flange plates are equal, with respect to area and modulus, to the flange itself.

Checking the flange plates under axial load and bending.

the basic requirement is:

$$\frac{F_f}{A_e \times p_{yp}} + \frac{M_f}{M_{cx}} \leq 1$$

where:

A_e = the effective area
 = K_e x net area, but ≤ gross area
K_e = 1.2 for design grade 43
K_e = 1.1 for design grade 50
F_f = maximum axial force in the flange
 = C or T (STEP 1)
p_{yp} = the design strength of the plate
M_f = the moment in each flange due to strut action or any biaxial bending moment (Appendix C3 BS 5950: Part 1)
M_{cx} = the moment capacity of the flange plate
 $p_y \times Z_{net}$
Z_{net} = is section modulus of plates minus holes.

Check preloaded bolts

It is required that:

$$F_r \leq P_s$$

where:

P_s = bolt capacity (see STEP 3)
F_r = maximum bolt force (Figure 5.8)
 = $\sqrt{[F_s^2 + F_m^2 + (2 \times F_s \times F_m \cos\theta)]}$
F_s = $\frac{F_f}{n}$
n = number of bolts (each side)
F_m = $\frac{M_f}{Z_b}$
Z_b = elastic modulus of bolt group
θ = angle determined in figure 5.8.

Note: that if e_1, e_2 or e_3 is less than 3d, the resultant end distance e_r should be determined and the bearing capacity reduced if necessary.
Where: d = bolt diameter.

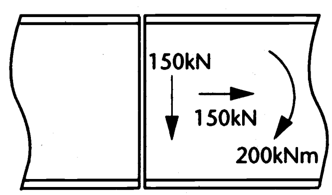
5.4 BOLTED SPLICE - WORKED EXAMPLE

In this worked example the splice is subjected to bending moment, shear and axial forces. It is assumed that local restraints prevent out-of-plane buckling and therefore STEP 5 need not be considered.

CALCULATION SHEET BCSA	Job: *Moment Connections*		Sheet: *1 of 3*
	Title: *Bolted Splice Cover Plate Beam-to-Beam Connection*		
	Client: *SCI/BCSA Connections Group*		
	Calcs by: *DGB*	Checked by: *R S*	Date: *Apr 95*

Design a bolted cover plate splice in a 457 x 191 x 67 UB (Design grade 43).

The connection is to carry the bending moment, shear force and axial tension shown (at ultimate limit state).



$$D_B = 453.6\text{mm}$$
$$T_b = 12.7\text{mm}$$
$$t_b = 8.5\text{mm}$$
$$B_b = 189.9\text{mm}$$

DISTRIBUTION OF INTERNAL FORCES — STEP 1

Calculate the larger force, F_f in the beam tension flange, or the beam compression flange.

$$\text{In tension, } T = \frac{M}{(D_b - T_b)} - \frac{N}{2} = \frac{200 \times 10^3}{(453.6 - 12.7)} - \frac{(-150)}{2} = 529\text{kN } (= F_f)$$

$$\text{In compression, } C = \frac{M}{(D_b - T_b)} + \frac{N}{2} = \frac{200 \times 10^3}{(453.6 - 12.7)} + \frac{(-150)}{2} = 379\text{kN}$$

FLANGE CAPACITY AND DESIGN OF FLANGE COVER PLATES — STEP 2

It is required that:

$$A_{ef} \geq \frac{F_f}{p_{ys}}$$

A_{ef} = *the effective flange area*

= *1.2 x net area (for design grade 43), but ≤ gross area*

$= 1.2 \times [189.9 - (2 \times 22)] \times 12.7$ but $\leq 189.9 \times 12.7$

$= 2224\text{mm}^2$ but $\leq 2412\text{mm}^2$

$$\frac{F_f}{p_{ys}} = \frac{529 \times 10^3}{275} = 1924\text{mm}^2 < 2224\text{mm}^2,\ \text{OK}$$

Title	*Bolted Splice Cover Plate Connection*	Sheet	*2 of 3*

Plates

Try a single, external cover plate, 180mm wide (design grade 43), with M20 preloaded ("HSFG") bolts (General grade) in 22mm clearance holes. Design the plates for the (larger) tension force.
By inspection, A_{ep} will govern.

It is required that:

$$A_{ep} \geq \frac{F_f}{p_{yp}}$$

$$1.2 \times t_p \times [180-(2 \times 22)] \geq \frac{529 \times 10^3}{275}$$

$$\text{Giving } t_p \geq \frac{529 \times 10^3}{275 \times 1.2 \times [180-(2 \times 22)]} = 11.8\text{mm, say } 12\text{mm}$$

Use 180 x 12 cover plate.

DESIGN OF FLANGE BOLTS

STEP 3

It is required that: $F_f \leq n_b \times P_s$

P_s = *minimum of slip resistance or bearing capacity of bolt in flange or cover plate*

Slip resistance = *71.3kN (Slip factor = 0.45)*

values from Tables on page 222

Cover plate thickness (12mm) is less than the flange thickness (12.7mm), therefore will be more critical in bearing.

Bearing capacity in 12mm plate is more than 148kN, > 71.3kN, OK.

Therefore, $P_s = 71.3\text{kN per bolt}$

and $n_b = \frac{F_f}{P_s} = \frac{529}{71.3} = 7.4,$ *therefore* ***use 8 Bolts (4 pairs) each side***

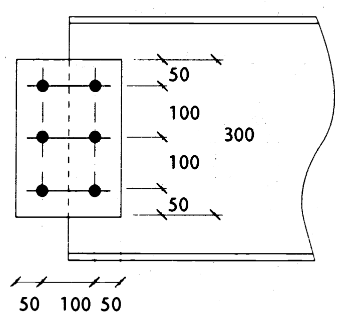
Note: Compression flange forces are lower, but an identical detail is chosen for consistency and to avoid potential errors.

WEB PLATES AND BOLTS

STEP 4

Try the following:

2 N° 10 mm web plates
M20 preloaded ("HSFG") bolts (General grade)
in 22mm holes



Web plates

In shear it is required that:

$$V \leq P_v$$

$$P_v = 0.6 \times A_{vnet} \times p_{yp}$$

$$= 0.6 \times 0.9 \times (L - n_r \times d_h) \times t_p \times p_{yp} \times n_p$$

$$= 0.6 \times 0.9 \times (300 - 3 \times 22) \times 10 \times 275 \times 2 \times 10^{-3} = 695\text{kN} > 150\text{kN} \quad \text{OK}$$

In bending:

$$M \leq M_c$$

$$M = V \times a = 150 \times 0.05 = 7.5\text{kNm}$$

$$I_p = \frac{10 \times 300^3}{12} - \frac{3 \times 10 \times 22^3}{12} - (2 \times 10 \times 22 \times 100^2) = 18.1 \times 10^6\,\text{mm}^4$$

$$M_c = p_{yp} \times Z_{net} = 275 \times 10^{-3} \times \frac{2 \times 18.1 \times 10^6}{150 \times 10^3} = 66.4\text{kNm} > 7.5\text{kNm} \quad \text{OK}$$

Title *Bolted Splice Cover Plate Connection*	Sheet *3 of 3*

STEP 4 continued

Web plate bolts

It is required that:

$$F_r \leq P_s$$

$$F_r = \text{resultant bolt load}$$

$$= \sqrt{(F_m^2 + F_v^2)}$$

$$F_m = \frac{V \times a}{Z_b}$$

$$Z_b = \frac{n_b\,(n_b + 1)\,p}{6} = \frac{3 \times 4 \times 100}{6} = 200$$

$$F_m = \frac{150 \times 50}{200} = 37.5\text{kN}$$

$$F_v = \frac{V}{n_b} = \frac{150}{3} = 50\text{kN}$$

$$\text{Therefore, } F_r = \sqrt{(37.5^2 + 50^2)} = 62.5\text{kN}$$

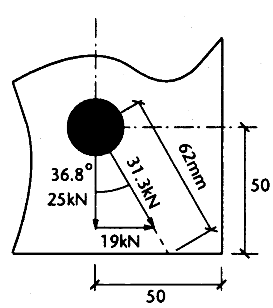
P_s = *the lesser of the slip resistance of the bolt, the bearing capacity of the bolt in the cover plate(s), or the bearing capacity of the bolt in the web*

Slip resistance	= 71.3×2	= 142.6kN	>	62.5kN	OK
Bearing in plates	= 165×2	= 330kN	>	62.5kN	OK
Bearing in web		= 140kN	>	62.5kN	OK

values from Tables on page 222

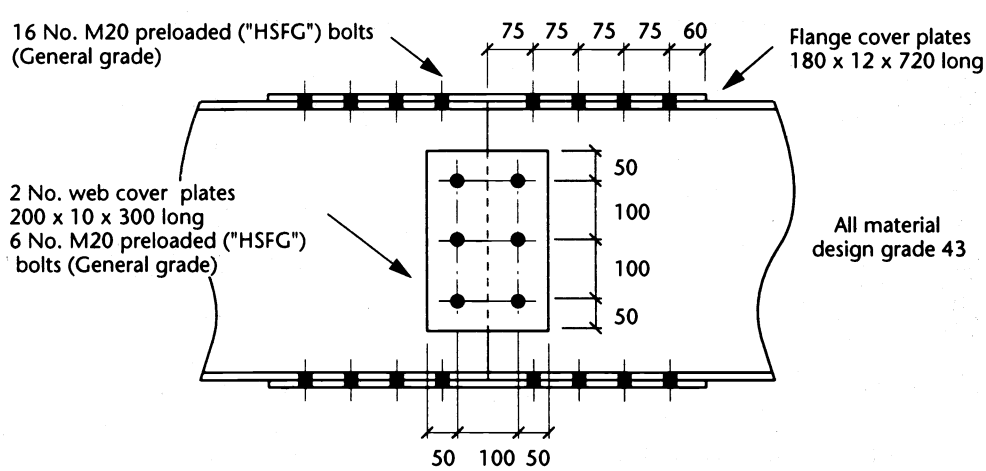
Note: *Although the bearing value in the cover plates is low (31.3kN each plate), the edge distances are smaller than 3d, and for completeness the end distance in the direction of the resultant bolt load should be checked.*

$$e_r = \frac{e_2}{\cos\theta} = \frac{50}{\cos 36.8} = 62\text{mm} > 3d,\ \text{OK}$$



Note: *If edge distance is less than 3d, the bearing capacity of the plate is reduced proportionally to the reduced edge distance.*

The final connection is thus:



5.5 BOLTED MOMENT END PLATE SPLICES

5.5.1 Connection details

Bolted moment end plate connections, as splices, are simply the beam side of the connections covered in Section 2 mirrored to form a pair. They have the advantage over the cover plate type in that preloaded bolts and faying surfaces are not required. However they provide less rigidity than cover plate splice details.

Connection rotational stiffness is discussed in Section 2.5. The bolted moment end plate splice is regularly used in single storey portal frames for both apex connections and intermediate splices in rafters where it is commonly assumed to be 'Rigid' for the purposes of elastic global analysis. When they are used in multi-storey unbraced frames, a more cautious approach is recommended.

Figure 5.9 illustrates a variety of bolted moment end plate splices.

Some of the details shown will only provide partial strength splices, or full strength for a moment in one direction.

A bolted end plate splice can be used for columns in multi-storey braced frames, and may be considered for unbraced frames where the criteria in Section 2.5 are met.

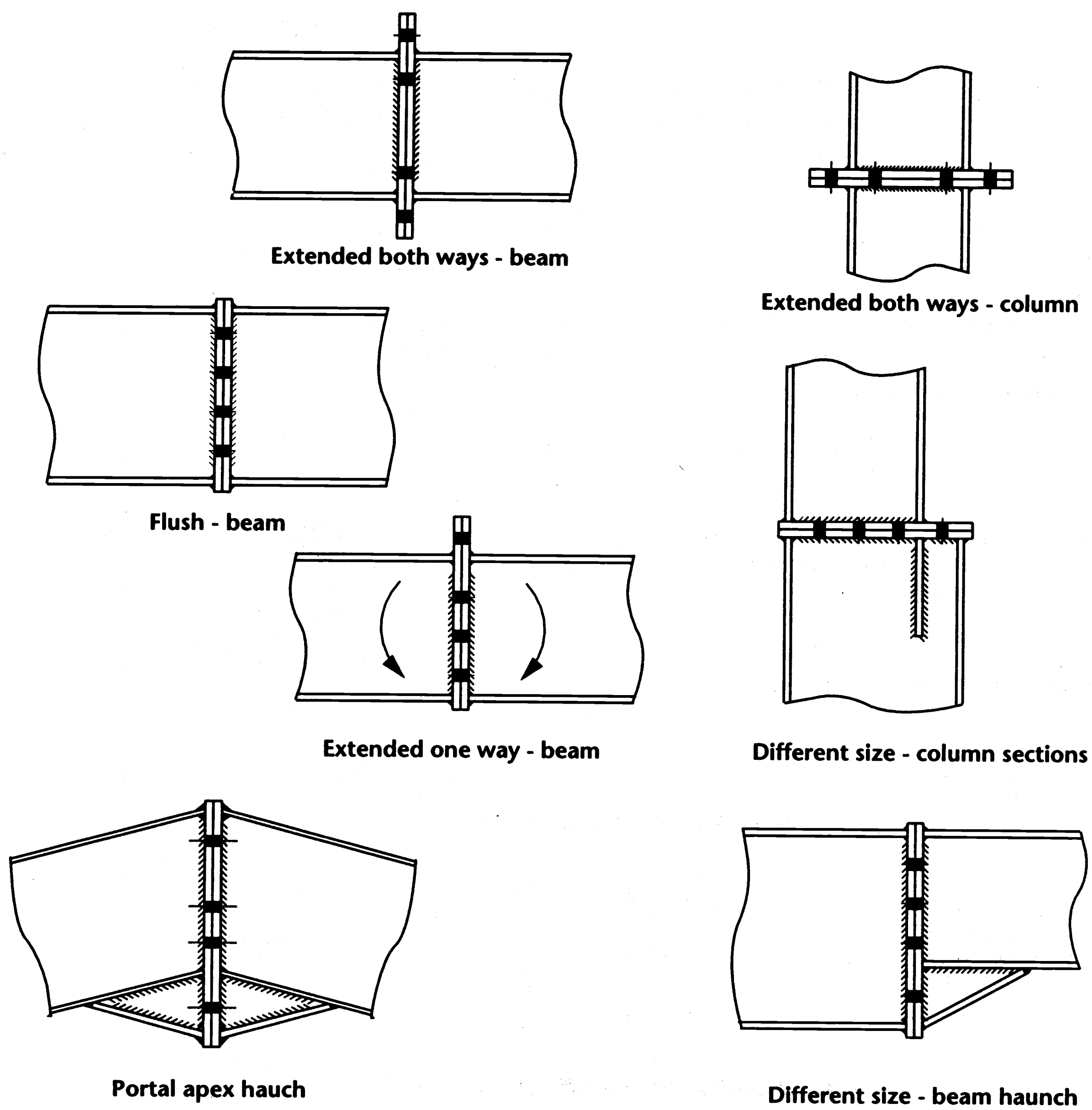


Figure 5.9 Typical bolted moment end plate splices

5.5.2 Design procedures

Design procedures are those employed in Section 2 for the 'beam side' of the beam-column connection. The moment capacity tables on pages 150 to 181 can be used directly for splice connections of this type, providing moment and shear capacities.

When splicing beams of different depths, the longitudinal stiffeners should be sized to equal in area the opposing flange. They must be of sufficient length to transfer the applied force from the flange to the adjacent web in shear, and to develop the force in the weld between the stiffener and the web.

5.6 BEAM-THROUGH-BEAM MOMENT CONNECTIONS

5.6.1 Connection details

Typical beam-through-beam connections are shown in Figure 5.10. The extended end plate and flush end plate types would normally employ non-preloaded 8.8 bolts. The connection using end plates to the web and a cover plate to the tension flange would require preloaded bolts to avoid slip. Although the web connection could be made with normally tightened ordinary bolts, the *NSSS*[10] discourages a mix of preloaded and non-preloaded bolts being used in the same connection.

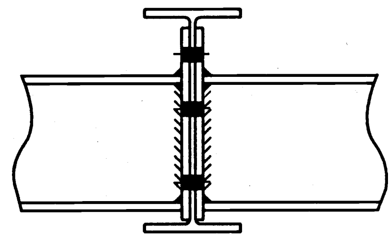
The effect of the rotational flexibility of end plate type splices must be borne in mind. The guidance given in Section 5.5.1 applies here.

5.6.2 Design procedures

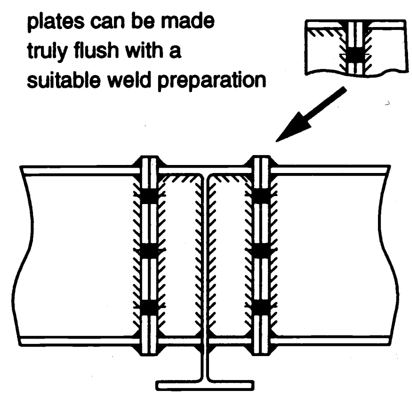
Design principles for the elements of each connection are as presented in earlier sections. The procedures in section 2.8 apply to the end plate elements, and the moment capacity tables on pages 150 to 181 can be used directly.

Where end plate connections are made to the web of the supporting beam the vertical shear from the beams on both sides must be considered when checking bolts in bearing on the web.

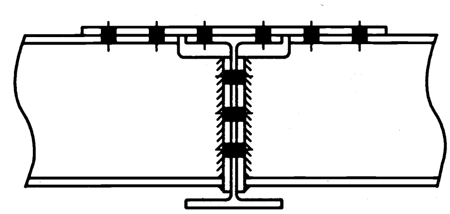
When the end plate/cover plate detail is used, the cover plate element is designed in accordance with the procedures in 5.3. The bolts shown in Figure 5.10 between the cover plate and the supporting beam serve only to keep the parts in contact. The end plate bolts are assumed to carry vertical shear only and should be checked in the normal way.



Extended end plates to beam web



Flush end plates to beam web stiffeners



End plate and cover plate

Figure 5.10 Typical beam through beam splices

5.7 WELDED SPLICES

5.7.1 Connection details

Typical welded splices are shown in Figure 5.11.

Welded shop splices are often employed to join shorter lengths delivered from the mills or stockists. In these circumstances the welds are invariably made 'full strength', although the effect of small cope holes may be neglected.

Where the sections being joined are not from the same 'rolling' and consequently vary slightly in size because of rolling tolerances a division plate separates the two. When joining components of a different serial size a web stiffener is needed, or a haunch to match the depth of the larger size.

A site splice can be made with fully welded cover plates. In this case the width of the top and bottom flange plates are chosen to allow downhand welding of the longitudinal welds. Bolts in the web covers are for temporary erection purposes.

In connections where a division plate is used, the plate must be able to sustain tensile forces through its thickness. "Hy-zed" material (BS EN 10164) should be considered for the plate, and checking for laminations before and after welding is recommended.

It is also possible to weld one flange and one web plate to each member, and complete the splice by bolting on site. This detail has the disadvantage that both pieces require both drilling and welding in the fabrication shop. In addition the projecting splice plates are prone to damage during transportation.

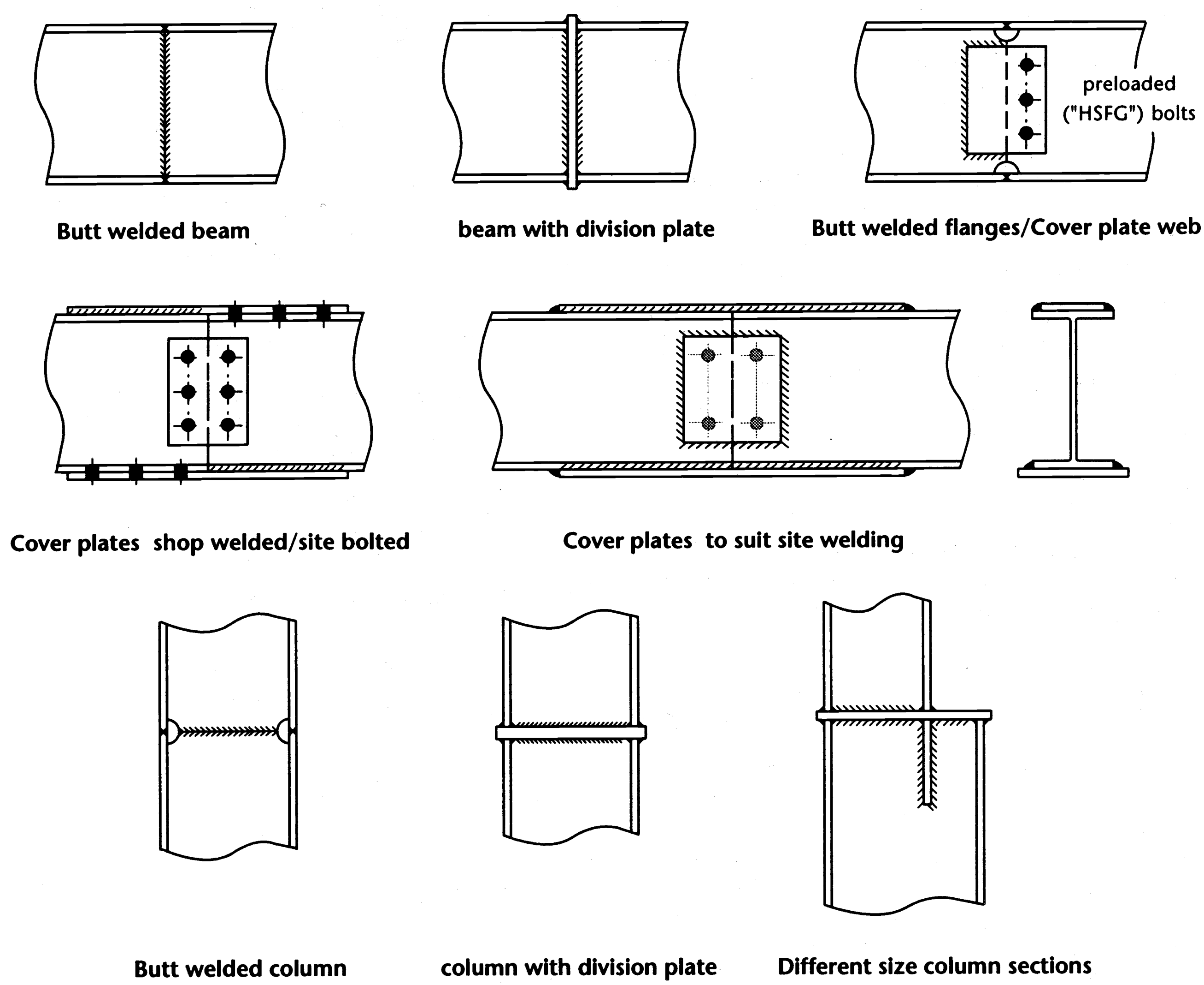


Figure 5.11 Typical welded splices

5.7.2 Design Philosophy

For fully welded splices the general design philosophy is:

- The applied moment is carried entirely by the flanges.
- Any axial forces are shared in proportion to the area of flanges and web.
- Transverse shear is carried by the web.
- Full strength welds must always be used in connections for statically indeterminate frames, whether designed plastically or elastically.

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

In statically determinate frames splices may be designed to resist an applied moment which is less than the member moment capacity.

Where there are mixed bolted and welded elements the design philosophy is that the bolted portions can be designed for the applied forces, since it is considered that sufficient ductility is present; however, **the flange welds must be full strength.**

American research(16) has shown that the full moment capacity of the section can be achieved when a bolted web is used with welded flange connections, but until this work has been validated in the UK, the stress in the flange should be limited to $1.2p_y$ when a bolted, or partly bolted, web detail is used.

A welded flange/bolted web splice designed under these interim rules will therefore not achieve the moment capacity of the beam in most cases. When a full strength connection is required, the beam web and flanges will have to be fully welded (except for small cope holes).

Practical details

When butt welds are used to connect the flanges and the web, care must be taken to ensure that full penetration is achieved.

When welding is from both sides, the weld procedure would normally include back gouging after the first side has been welded to remove slag from the root of the weld.

When welding from one side, a backing strip is provided and full penetration is then achieved. This is the type of weld commonly used on site. See Figure 5.12

Where a division plate is used, the weld will be continuous round the profile of the section, and the fillet welds or partial penetration welds employed should be designed in accord with Section 2, Step 7. The division plate itself should be of the same grade as the component it connects, and have a thickness at least equal to the flange thickness. As stated in Section 5.7.1, "Hy-zed" material should be considered for the division plate, and checking for laminations before and after welding is recommended.

5.7.3 Capacity checks

Separate procedures are not included for each of the splices shown in Figure 5.11 but the appropriate steps in Section 4.5 for beam to column welded connections may be used.

When designing such splices the following points must not be overlooked:

- Welds may be designed for the applied loads in statically determinate all welded details only. In all other cases, welds must be full strength.
- In splices with plate and bolt elements the flange stresses must not exceed 1.2 p_y (See discussion under 4.4 on page 60).

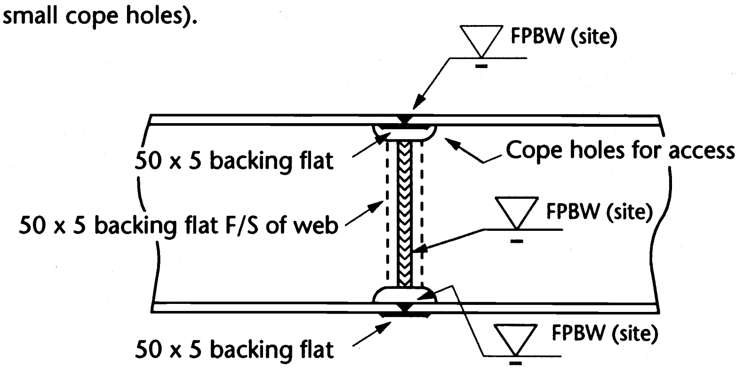


Figure 5.12 Full strength site welds with backing strips

6. COLUMN BASE CONNECTIONS

6.1 SCOPE

This chapter deals with the design of connections which transmit moment between steel members and concrete substructures.

In practice most such connections occur at the feet of columns, but the same principles may be applied to non-vertical members. Typical details are shown in Figure 6.1.

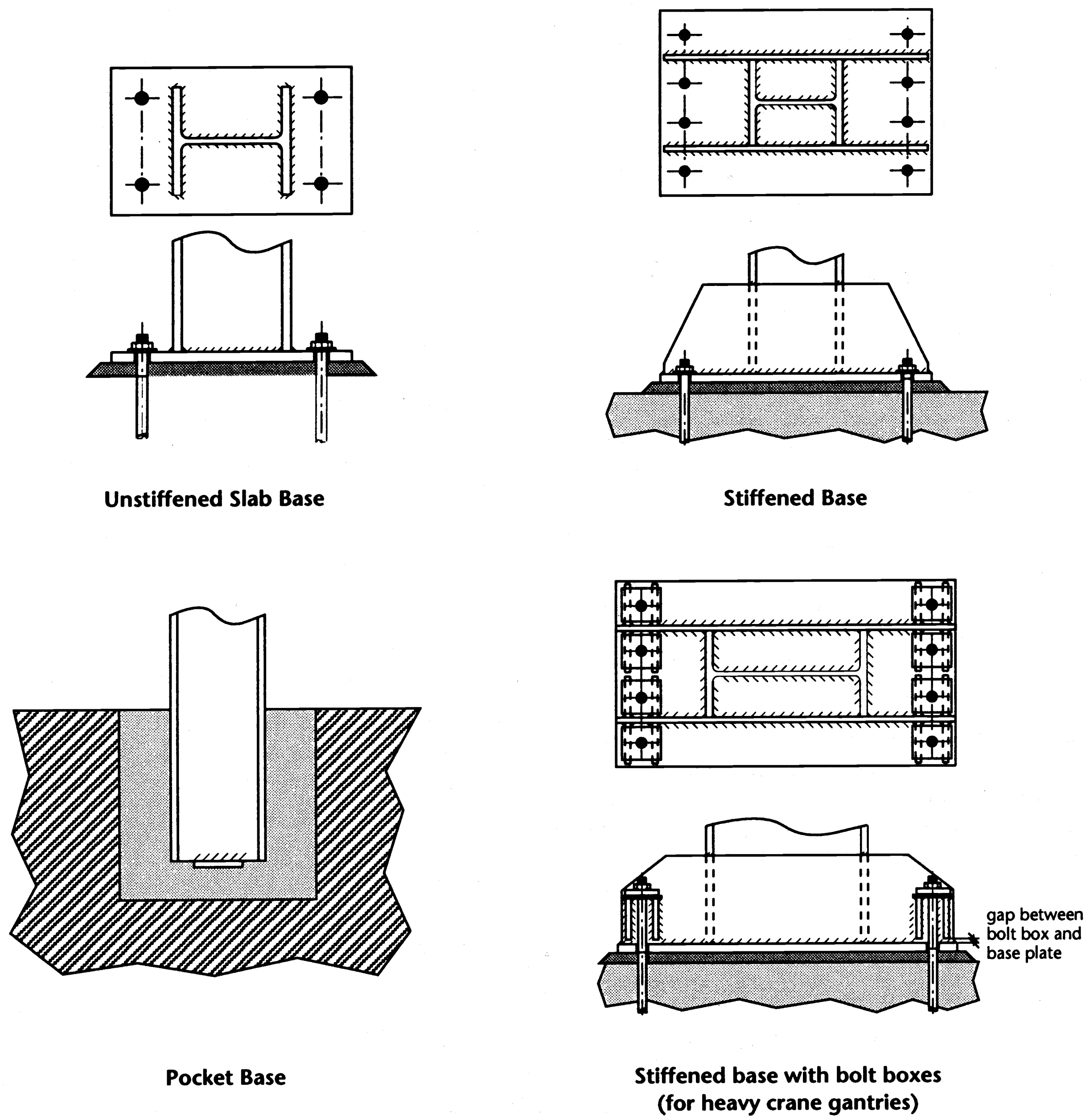


Figure 6.1 Typical column base connections

6.2 DESIGN PHILOSOPHY

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

- Axial forces are more liable to be important than is generally the case in end plate connections.
- On the compression side force is distributed over an area of steel-to-concrete contact which is determined by the strength of the concrete and packing mortar or grout.
- On the tension side the force is transmitted by holding down bolts which must be adequately anchored in the concrete substructure.
- Unlike steel-to-steel contact, concrete on the tension side cannot be relied upon to generating prying forces so as to reduce the bending moment in the end plate. The base plate must be considered to bend in single curvature similar to Mode 3 for a bolted end plate.

As a consequence, the base plate tends to be either very thick, or heavily stiffened, by comparison with end plates of steel-to-steel connections. However the principle is similar:

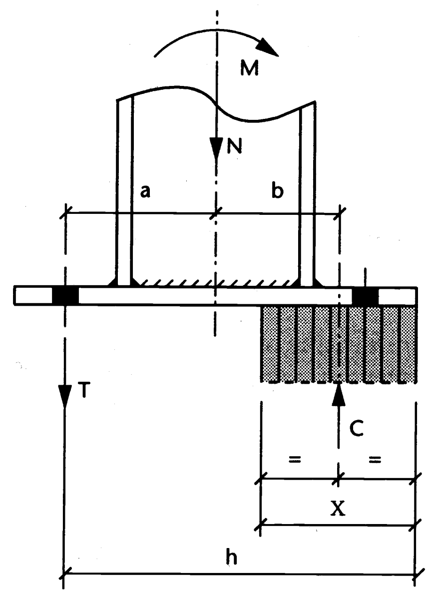
An interface compression force is coupled with a tensile force in the bolts to balance the applied axial compression and bending moment.

The method is analogous to reinforced concrete design, with a plastic distribution of bolt force, and concrete/ grout compression taken as a rectangular stress block. There are certain provisos, discussed in Section 6.3.

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 are appropriate.

The connection will usually be required to transmit horizontal force (base shear), either by friction or via the bolts. If the latter, it must be remembered that the concrete may limit the force that can be accepted. Shear keys set in pockets (or shallow pockets embracing the feet of the columns) can be used to transfer higher shear forces.

Figure 6.2 shows the distribution of forces and gives the equations which must be satisfied simultaneously for a simple base with one row of bolts on each side.



$$N = C - T$$

$$M = (T \times a) + (C \times b)$$

Figure 6.2 Distribution of forces and equations of equilibrium

More complicated bases (eg unsymmetrical, or more than one tensile bolt row) can be treated similarly.

When there is biaxial bending, a similar approach can be used, but with a certain amount of trial and error, since the extent of the stress block must be assumed so as to satisfy equilibrium in both directions.

Pocket bases

The 'Cast in Pocket' base is entirely different and is not dealt with in detail here. Design must take account of the horizontal and vertical pressures on the concrete encasement. A small plate or angle can be welded to the underside of the member to assist with levelling of the column. See Figure 6.1.

Bolt boxes

When bases have to be designed for heavy gantries with large cranes causing an overturning moment at the base from high surge loads, holding down bolts as large as M100 are sometimes necessary. In these cases, bolt boxes are provided to carry bolt tensions directly into the column shaft, or into the base stiffening gussets. Base plate bending is then only considered on the compression side.

6.3 CAPACITY CHECKS

Compression stress block

The magnitude of the assumed compressive stress depends on:

- The compressive strength of the concrete.
- The compressive strength of the grout/packing mortar.
- The quality of workmanship to be expected.

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.

Table 6.1 gives typical cube strengths for mortar, fine concrete and non shrink grout, and is taken in part from *Holding Down Systems for Steel Stanchions* [19].

Table 6.1 Strength of bedding material

Bedding Material	Characteristic cube strength at 28 days f_{cu} (N/mm²)
Mortar	20 - 25
Fine Concrete	30 - 50
Non shrink Grout	50 - 60

Table 6.2 gives values of characteristic cube strengths and bearing strengths which are used for concrete bases. They are based on the grades of concrete given in BS 5328: Part 1.

Table 6.2 Concrete Strengths

Concrete grade	Cube strength at 28 days f_{cu} (N/mm²)	Design bearing stress for stress block $0.6 f_{cu}$ (N/mm²)
C25	25	**15**
C30	30	18
C35	35	21
C40	40	24

It must be emphasized 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.**

Bolt tension

A plastic distribution of bolt force may be adopted provided that all bolts assumed to act in tension are a reasonable distance outside the area of the compressive stress block. It is recommended that the extent of the compression stress block 'X' is limited to two-thirds of the distance 'h' from the compression edge to the tension bolts ('X' and 'h' are shown in Figure 6.2).

Recommended bolt design tensions are the enhanced values given in Table 2.1.

Anchorage lengths depend on the concrete properties and foundation details. Commonly used bolt sizes and lengths are given in Table 6.3.

All holding down bolts should be equipped with an embedded anchor plate for the head of the bolt to bear against. Sizes of anchor plates are also given in Table 6.3. 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.

When necessary, more elaborate anchorage systems (eg traditional back-to-back channel sections) can be designed. 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.

Table 6.3 Preferred sizes of holding down bolts & anchor plates

Diameter		M20	M24	M30
Length (mm)		**300** 375 450	375 **450** 600	450 **600**
Anchor Plates (mm)	size	100 x 100	120 x 120	150 x 150
	thickness	12 (4.6) 15 (8.8)	15 (4.6) 20 (8.8)	20 (4.6) 25 (8.8)

Standard lengths of holding down bolts are in bold type.

6.4 RIGIDITY 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 base plates, whether stiffened or unstiffened, are substantially more rigid than the 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, but borderline cases present difficulties as with steel-to-steel connections. In principle, a target stiffness needs to be defined and the stiffness of the proposed connection needs to be quantified and compared. In practice, this is generally not done.

Base connections for wind-moment frames are generally treated no differently from other moment-resisting base connections. Although the collapse mechanism is likely to involve a plastic hinge at each column foot, the normal requirement for ductility (to redistribute end moments) does not apply.

6.5 STANDARDISATION

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 *Joints in Simple Construction*(9), 4.6 holding down bolts are advocated as standard. However, such a restriction is an unacceptable handicap where high bending moments are to be transmitted, and 8.8 bolts are preferred.

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 6.3.

6.6 BEDDING SPACE FOR GROUTING

A bedding space of at least 50mm is normal. This 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 700mm x 700mm or larger, 50mm diameter holes should be provided to allow trapped air to escape and also for inspection. A hole should be provided for each $0.5m^2$ of base area. If it is intended to place grout through these holes the diameter should be increased to 100mm.

6.7 PRELIMINARY SIZING OF BASE PLATE

When using grade 8.8 holding down bolts and the suggested 'default' bearing stress of $15N/mm^2$, suitable approximate base plate dimensions can be determined as a first trial from Table 6.4.

Table 6.4 First trial plate dimensions

Bolt Size	Base plate thickness (unstiffened)	Maximum outstand	Edge distance	Bolt spacing
	mm	mm	mm	mm
M20	35	100	50	120
M24	45	150	75	150
M30	50	150	75	180

For UC column serial sizes the dimensions given in Table 6.4 translate into the preferred sizes of square base plate given in Table 6.5. The table also provides some indication of the moment resistance available.

Moment resistances are approximate (they vary with actual section size used) and should be considered as a guide to preliminary sizing only.

When axial loads are higher than those given in Table 6.5 the base should be checked for axial load combined with the overturning moment, to see if there is tension in the holding down bolts. Where a viable stress block can be postulated within the confines of the base plate, equilibrium may be achieved without tension in the bolts.

The distance from the column centre line to the centre of the stress block is found by dividing the moment by the axial force. If there is no tension in the bolts, the calculated reaction will be equal to or greater than the axial load.

Table 6.5 Preliminary sizing chart for UC column bases with C25 concrete						
Column	305UC	254 UC		203 UC		152 UC
Base Plate (mm)	600 x 600	550 x 550	450 x 450	500 x 500	400 x 400	350 x 350
Plate thickness (mm)	50	50	35	50	35	35
8.8 HD Bolts (each side)	4 M24	4 M24	4 M20	3 M24	3 M20	3 M20
Bolt edge distance (mm)	75	75	50	75	50	50
Axial Load kN	**Moment Resistance (kNm)**					
Zero	381	338	197	229	130	107
250	430	379	228	267	157	125
500	473	413	250	298	175	132
750	509	438	263	320	182	
1000	537	457		333		
1250	559	467				
1500	574					
1750	582					
2000	577					

6.8 STIFFENED BASE PLATES

Holding down bolts cannot be as compactly spaced as bolts in other situations in the structure. An unstiffened slab base therefore tends to require a plate which is thick by comparison with bolted end plate connections. There is little scope to increase bolt spacing and base plate dimensions before the thickness gets out of hand.

Most steelwork contractors would prefer to use unstiffened bases of plate thickness up to 75mm. Above this thickness, the decision becomes a balance between weldability and availability of the thicker material, compared to the high work content of a stiffened base. The base plate must be free from lamination in the area of the welds and flat enough to ensure bearing as required by the *NSSS*[10].

When a stiffened base is chosen, it is normally appropriate to use the base plate thicknesses given in Table 6.6.

The stiffener arrangement should be made such that the holding down bolts are about 50 mm from the face of the stiffener.

Table 6.6 Suitable base plate thickness for stiffened column bases			
Bolt size	M20	M24	M30
Thickness (mm)	30	35	40

The design of a stiffened base generally follows the same procedures as outlined for the unstiffened base, whilst the stiffeners themselves must be designed to resist the local load they attract by virtue of their position.

Stiffeners may be sized using the following guidelines:

- Outstand from column to extend to approximately 20mm inside the edge of the base plate.
- Height equals two times outstand (corner may be trimmed at 2:1).
- Thickness not less than 10 mm or height/16 (unless restrained by an intersecting stiffener).
- Stiffeners must be sufficiently thick to resist the compression they attract from the assumed compressive stress block.
- Tension side is checked on similar basis to web tension in a beam end plate.

Base moments are usually reversible; welds between stiffeners and base plate may therefore be sized for tension plus any shear assigned to them, provided that the stiffeners bear directly on the base plate.

Welds between stiffeners and column must be sized to resist all forces attracted by stiffener.

6.9 DESIGN PROCEDURE

An iterative approach must be taken in the design procedure for a base plate connection.

The starting point is to determine the eccentricity (M/N). This leads to an indication of the necessary base size if no bolt tension was available. If, with different load cases, the eccentricity is substantially greater for one direction than the other, it may be appropriate to consider an asymmetrical base detail.

An unstiffened base plate should be considered first. Even if fairly thick, it will be cheaper than a stiffened base. If the eccentricity is high, stiffening may be unavoidable .

The procedure given on the following pages is for the simple case of uniaxial bending and a single row of bolts acting in tension. The design sequence is indicated in Figure 6.3 and the critical dimensions in Figure 6.4.

For biaxial bending the determination of a compressive stress block in STEP 1 is more complex and requires a trial-and-error process, but the design sequence is otherwise the same.

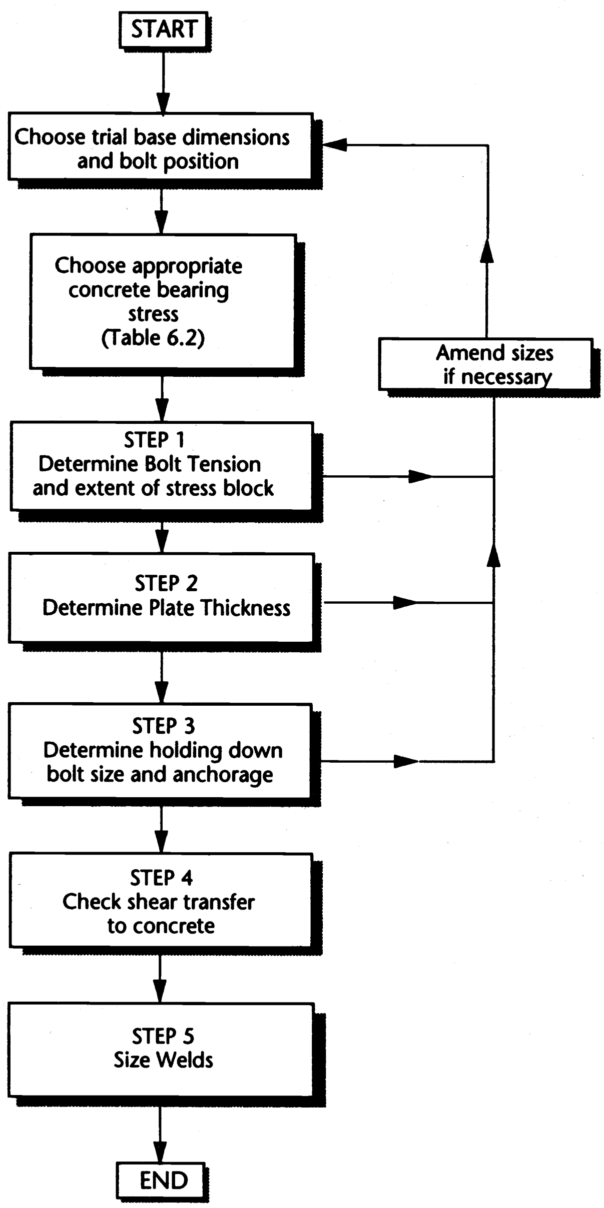


Figure 6.3 Flow diagram for base design

STEP 1 CALCULATION OF BOLT TENSION AND CONCRETE COMPRESSION

With the trial base plate dimensions chosen, and the design bearing stress decided (Figure 6.4), the equilibrium equations are:

$$N = C - T \qquad (6.1)$$

$$M = Ta + Cb \qquad (6.2)$$

Substituting for a and b, equation *(6.2)* becomes:

$$M = T\left(h - \frac{h_p}{2}\right) + C\left(\frac{h_p - X}{2}\right) \qquad (6.3)$$

also,

$$C = 0.6f_{cu}b_pX \qquad (6.4)$$

$$T = C - N \qquad (6.5)$$

Substituting *(6.4)* and *(6.5)* into *(6.3)* gives

$$M = 0.6f_{cu}\,b_p X\left(h - \frac{X}{2}\right) - N\left(h - \frac{h_p}{2}\right) \qquad (6.6)$$

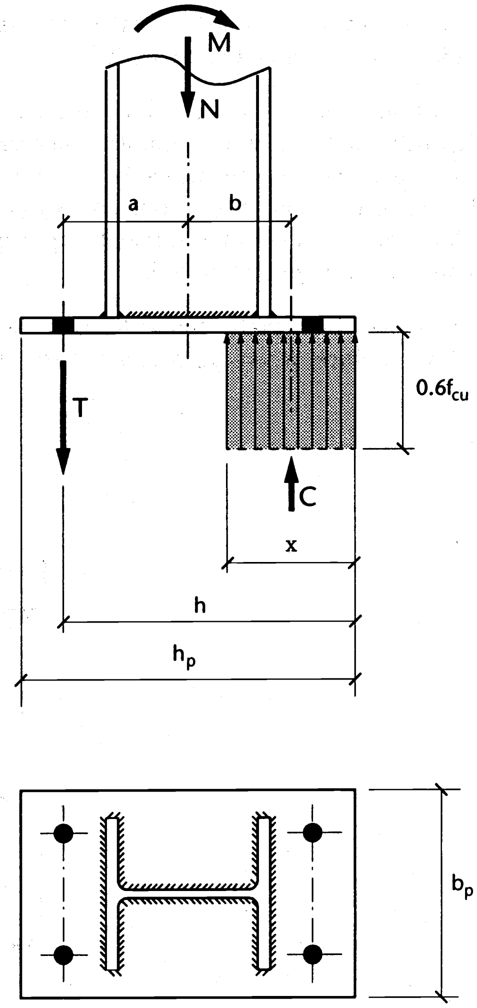
Note that the effective width is limited to the width of the column plus twice the cantilever L_1 (see Figure 6.7).

The quadratic equation is solved to determine X. (See figure 6.4)

Caution:

If the above equations do not give a sensible solution it could be because:

- *No tension required to resist the moment. (ie M ÷ N shows only a small eccentricity and the whole area of the base is in compression.)*
- *The base plate is not big enough to resist the imposed forces, ie the wrong trial size has been chosen.*



b_p = breadth of base plate

h_p = length of base plate

h = length from tension bolts to compression edge

$0.6f_{cu}$ = design bearing stress on concrete *(An upper limit of $15N/mm^2$ is recommended unless there will be special control over the placing of the bedding material.)*

X = length of compressive stress block

Figure 6.4 Base dimensions and compression block

STEP 2 DESIGN BASE PLATE THICKNESS

Plate bending on either the tension side or the compression side may govern.

Both sides must be investigated and the required plate thickness is the larger value resulting from these checks.

(a) Compression Side Bending

Projecting portion of base as a cantilever:
(Figure 6.5)

t_p = required base plate thickness

$$= \sqrt{\frac{4m_c}{p_{yp}}} \qquad (6.7)$$

where:

m_c = moment per mm width applied to plate from stress block

$$= 0.6f_{cu}\frac{e^2}{2}$$

p_{yp} = design strength of plate

e = $L_1 - 0.8s_w$

L_1 = cantilever length of base plate (see Figure 6.5)

s_w = weld size

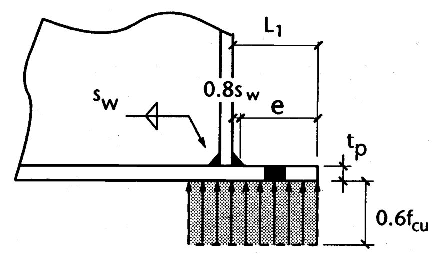


Figure 6.5 Uniform pressure on cantilever

Stiffened bases

In the case of stiffened bases (and occasionally unstiffened bases with a low axial force) the situation may occur when the width, X, of the stress block is smaller than the outstand, L_1 as shown in Figure 6.6. In these cases the value of m_c should be calculated as below and used in equation (6.7) to calculate t_p.

$$m_c = 0.6\,f_{cu}\,X\,(e - \frac{X}{2})$$

Note:
This approach is conservative because two-way spanning has been neglected.

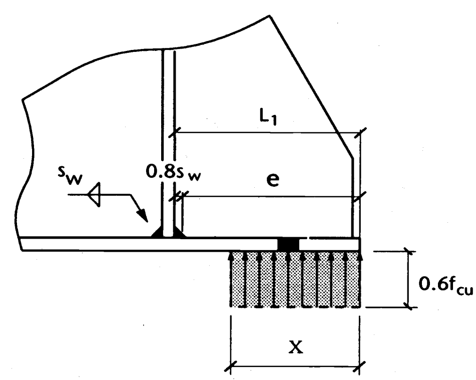


Figure 6.6 Uniform pressure on part of cantilever

Base plate spanning between column flanges

If the compressive stress block needs to extend into the area between column flanges, the effective cantilever cannot be more than L_1 without increasing plate thickness. The stress block therefore changes from a rectangular area to a 'T' shaped area around the flange and web of the column as shown in Figure 6.7.

This changes the position of the centroid to that of a 'T' section and necessitates the recalculation of the equilibrium equations of STEP 1 to re-establish C and T.

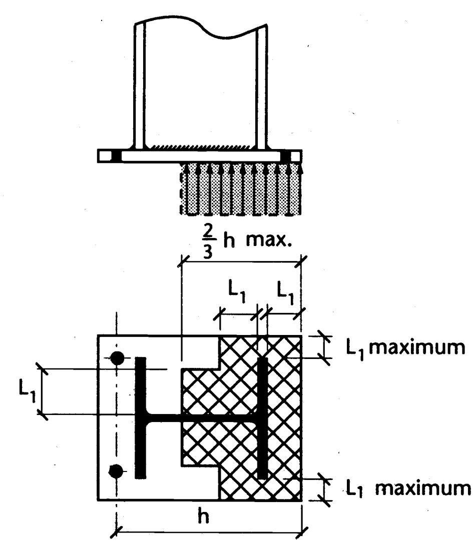


Figure 6.7 'T' shaped stress block

STEP 2	DESIGN BASE PLATE THICKNESS (CONTINUED)

(b) Tension Side Bending

With precautions taken as figure 6.8 to ensure that bending across corners of plate is avoided, the required plate thickness to resist bolt tension is based on a calculation for a pure cantilever, with no prying assumed.

Note: Plate bending across the corners may only be avoided by ensuring bolts are positioned within lines 45° from the corner of the column flange. (See Figure 6.8)

t_p = required base plate thickness

$$= \sqrt{\frac{4m_T}{p_{yp}\, b_p}}$$

Where,

m_T = $T \times m$

p_{yp} = design strength of plate

m = $L_1 - k - 0.8s_w$

L_1 = cantilever length of base plate

s_w = weld size

k = edge distance

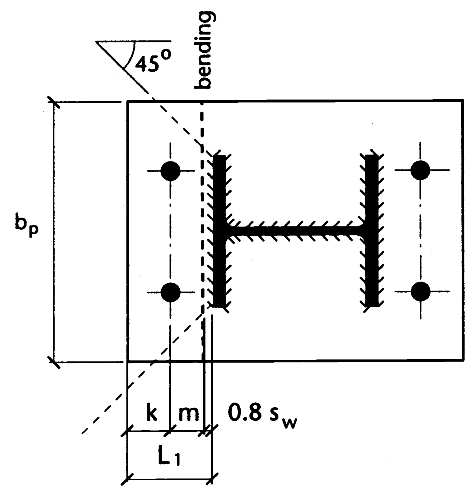


Figure 6.8 Plate bending on tension side

STEP 3 HOLDING DOWN BOLTS AND ANCHORAGE

HOLDING DOWN BOLTS

Force T in the row of bolts resisting tension is assumed to be shared equally among all bolts in the row.

The force per bolt should not exceed the value given in Table 6.7.

If it proves impractical to accommodate sufficient bolts of reasonable size, it is necessary either to:

- increase base plate dimensions (return to Step 1)

or

- add a second row of bolts behind the flange, if space permits

See Step 5 for the transfer of shear force from the base plate to the concrete.

Suggested practical limits of bolt spacing are given in Table 6.8. These minimum spacings apply in both directions. The minimum edge distances in the concrete should be of the same order, with reinforcement passing around and, where practical, between the bolts.

Table 6.7 Tensile capacity of H D bolts

Bolt size	Enhanced Tension Capacity P_t'(kN) 8.8	4.6
M20	137	59.5
M24	198	85.7
M30	314	136

Note: See appendix IV for derivation of bolt strengths

Table 6.8 Suggested minimum practical bolt spacing for H D bolts (mm)

	M20	M24	M30
Adjustable H D Bolts (using sleeves)	120	150	180
Non-adjustable HD Bolts *accurately held in position during concreting*	100	120	150

ANCHORAGE TO THE CONCRETE

Normally the objective is to ensure that the anchorage is as strong as the bolt that is used. See also discussion on bolt tension in Section 6.3.

The anchorage may be developed either by bond along the embedded length or, more commonly, by bearing via an anchor plate at the end of the bolt.

Bond along the embedded length

Where bond is relied upon, the bolt can be regarded as a reinforcing bar. To avoid unduly high strains, the bolt design strength should be limited to 400 N/mm².

From BS 8110 clauses 3.12.8.3 and 3.12.8.4, the basic requirement is:

$$f_b \leq f_{bu}$$

where:

f_b = anchorage bond stress

$= \dfrac{T}{n \times \pi \times d \times L}$

T = total tension force in the H D bolts

n = number of H D bolts on the tension side

d = H D bolt diameter

L = anchorage length (See Figure 6.10)

f_{bu} = design ultimate anchorage bond stress

$= 0.28\sqrt{f_{cu}}$

f_{cu} = concrete cube strength.

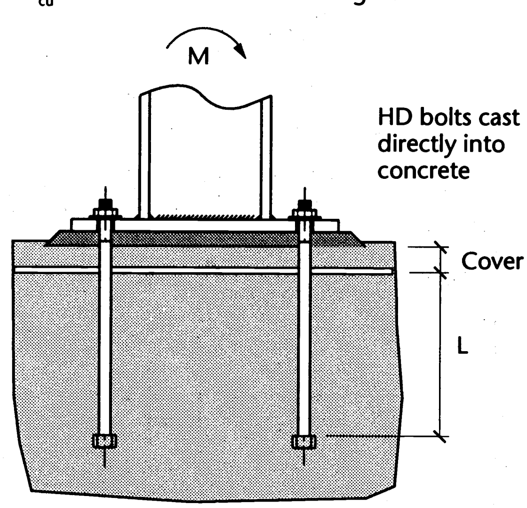


Figure 6.9
Anchorage bond length

STEP 3 HOLDING DOWN BOLTS AND ANCHORAGE (CONTINUED)

Anchor plates

An approximate rule for individual square anchor plates is (see table 6.3 on page 88):

8.8 bolts	•	5d	x 5d	x 0.8d thick
4.6 bolts	•	5d	x 5d	x 0.6d thick
		where d = bolt size		

If combined anchor plates are made to serve two or more bolts, a similar area should be provided symmetrically disposed about each bolt location.

Although traditional methods have been based on pull out of cones, it is recommended that checks should be based on a reinforced concrete analogy with the concrete section being checked for punching shear in accordance with BS 8110, clause 3.7.7. However, the procedure can only be used for reinforced concrete bases.

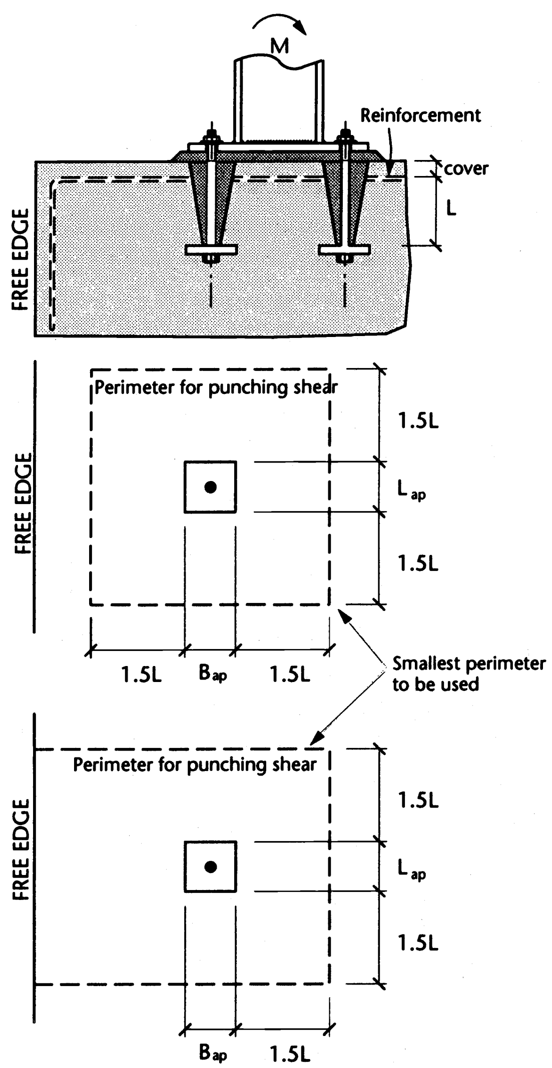


Figure 6.10 Punching shear perimeter for a single bolt

In BS8110, punching shear is considered at a rectangular perimeter 1.5L outside the loaded area. (See Figure 6.11) The perimeter may also be reduced by proximity to a free edge.

If bolts are placed such that their perimeters overlap, they should be checked as a group with the rectangular perimeter being at 1.5L around the group. (See Figure 6.12)

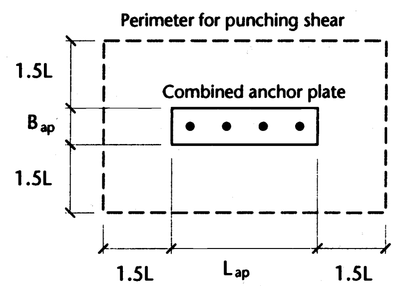


Figure 6.11 Punching shear perimeter for a bolt group

Basic requirement is:

$$f_v \leq v_c$$

where:

f_v = average shear stress over effective depth

$$= \frac{T}{P \times L}$$

T = total tension force in the bolts being considered within the perimeter

P = perimeter for punching shear (see Fig. 6.10)

L = effective depth of the H D bolt

L_{ap} = length of anchor plate

B_{ap} = width of anchor plate

v_c = design concrete shear stress obtained from table 3.9 of BS 8110 or alternatively:

$$v_c = \frac{0.79}{1.25} \times \left[\frac{100A_s}{P \times L}\right]^{1/3} \times \left[\frac{400}{L}\right]^{1/4} \times \left[\frac{f_{cu}}{25}\right]^{1/3}$$

where:

A_s = area of tension reinforcement in the base which includes all tensile reinforcement which passes through the zone within the perimeter and extends at least one effective depth, L or 12 bar diameters beyond on either side.

f_{cu} = concrete cube strength or 40N/mm² if lower.

If the term $\left[\frac{100A_s}{P \times L}\right]$ is < 0.15, use 0.15

and if the term $\left[\frac{400}{L}\right]$ is < 1, use 1.

STEP 4 **SHEAR TRANSFER TO CONCRETE**

In principle, shear may be transferred between the base plate and concrete in three ways:

- By friction. Available resistance of 0.3C may be assumed.
- In bearing, between the shafts of the bolts and the concrete surrounding them.
- Directly, either by setting the base plate in a shallow pocket which is filled with concrete or by providing a shear key welded to the underside of the plate. A minimum practical size - say grout space plus 50 mm into the concrete - is often ample.

In practice, most moment connections are able to rely on friction except where outlined below.

If high shear is combined with low moment and low axial compression, or if there is axial tension, friction may not suffice. In these circumstances it is safest to provide a direct shear connection (the third of the options listed above).

The second route, using the bolts to resist shear, can be effective but is difficult to depend on when the bolts are grouted in sleeves.

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 concrete of 3d and an average bearing stress of 2 f_{cu}.

When this approach is used, all bolts must be completely surrounded by reinforcement and bolts whose centre is less than 6d from the edge of the concrete in the direction of loading should not be considered.

$$H = n_s P_{ss} + n_t P_{ts}$$

where:

H	=	the design horizontal shear force
n_s	=	number of bolts in the non-tension zone
n_t	=	number of bolts in the tension zone
d	=	bolt diameter
P_{ss}	=	the shear capacity of a single bolt in the non-tension zone which is the lesser of: p_sA_s for bolt shear or $dt_p\, p_b$ for bolt bearing on the base plate or $6d^2f_{cu}$ for bolt bearing on the concrete
P_{ts}	=	the shear capacity of a single bolt in the tension zone which is the lesser of: $0.4\, p_s A_s$ for bolt shear or $dt_p\, p_b$ for bolt bearing on the base plate or $6d^2f_{cu}$ for bolt bearing on the concrete
p_s	=	the shear strength of the bolt
A_s	=	the shear area of the bolt (taken as the tensile area)
f_{cu}	=	lower cube strength of concrete or bedding material
p_b	=	the bearing strength of the baseplate (p_{bs} from Table 33 of BS 5950:Pt1).

STEP 5 WELDS - BASE PLATE TO COLUMN SHAFT

Welds between base plates and columns are sized in the same way as those between end plates and beams. Usually compression predominates and it is economical to ensure direct bearing between an accurately sawn column end and the base plate bearing surface. The weld can them be sized for tension and shear. Since moments are usually reversible, it is common to specify a single weld size all round (see Figure 6.12) and a minimum 8mm FW is appropriate for plate thicknesses up to 30mm.

Tension flange welds

The welds should be designed to carry a force which is the lesser of:

(a) The tension capacity of the flange,

$$= B_c \times T_c \times p_y$$

(b) The force in the tension flange,

$$= \frac{M}{D_c - T_c} - N \times \frac{A_f}{A_c}$$

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

Compression flange welds

As noted above, it is preferable to ensure direct bearing between an accurately sawn column end and the base plate bearing surface. Guidance on the necessary tolerances for bearing fit can be found in the *NSSS*. [10] If necessary the top surface of the base plate slab will have to be machined to achieve this.

If a bearing fit cannot be assumed then the weld must be designed to carry the lesser of:

(a) The crushing capacity of the flange, (see STEP 2B in Section 2)

$$= B_c \times T_c \times p_y$$

(b) The force in the compression flange,

$$= \frac{M}{D_c - T_c} + N \times \frac{A_f}{A_c}$$

where:

- M = design moment
- N = axial force in the column (+ve for compression)
- D_c = overall depth of column section
- T_c = column flange thickness
- A_f = area of the column flange = $B_c \times T_c$
- B_c = column flange width
- A_c = column cross-sectional area.

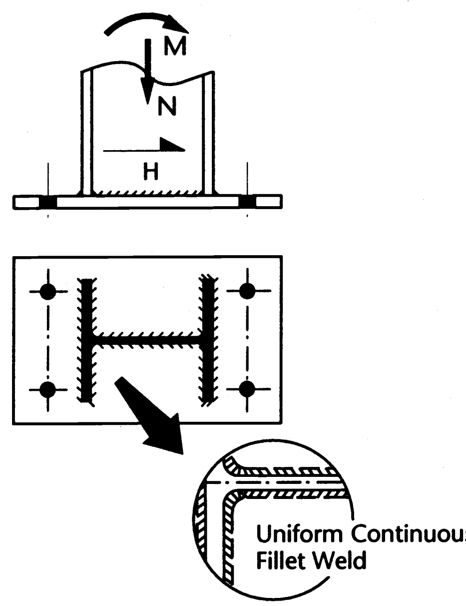


Figure 6.12 Base plate welds

Web welds

As already stated the weld is normally made the same size around the whole perimeter of the column section, but in cases where moments and axial loads are small compared with horizontal shears, a check on the web portions, considered to carry all shear, may be necessary.

The capacity of the column web welds for horizontal shear forces should be taken as:

$$P_{sw} = 2 \times 0.7 \times s_w \times p_w \times L_{ws}$$

where:

- s_w = fillet weld leg length
- p_w = design strength of fillet weld (BS 5950 Table 36)
- L_{ws} = length of web welds between fillets.

6.10 COLUMN BASE WORKED EXAMPLE

In the following example an unstiffened base plate is to be designed. The column is to bear on a reinforced concrete base.

The design uses a bearing strength of 18N/mm² for C30 concrete so special control of the grouting operation must be exercised as advised in section 6.3.

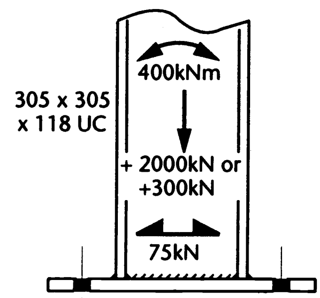
CALCULATION SHEET BCSA	Job: *Moment Connections*		Sheet *1 of 4*
	Title: *Column Base worked example*		
	Client: *SCI/BCSA Connections Group*		
	Calcs by: *RS*	Checked by: *DGB*	Date: *Apr 95*

Design an unstiffened column base for the column shown:

The base connection is to carry the combinations of overturning moment, axial load and shear forces indicated.

All forces are at ultimate limit state.

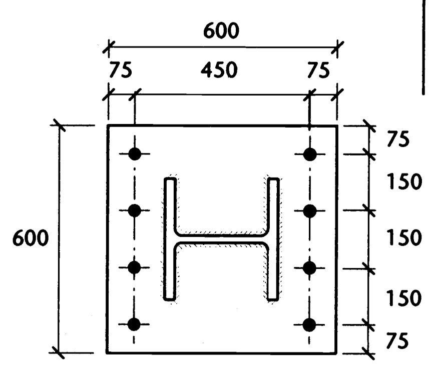
The foundation is to be in C30 concrete.



PRELIMINARY SIZING

A first guess can be obtained from Table 6.5 giving a 600 x 600 x 50 base plate with 4 M24 8.8 bolts each side. An axial load/moment of 300kN/400kNm and 2000kN/400kNm compares favourably with 250kN/430kNm and 2000kN/577kNm shown in the table (for C25 concrete).

Try base plate as shown:



(1) MOMENT PLUS MAXIMUM AXIAL FORCE:

M = ±400kNm, Axial force + 2000kN

Check whether there is tension in the bolts

First suppose there is no tension in the bolts:

$$b = \frac{M}{N} = \frac{400 \times 10^3}{2000} = 200\text{mm}$$

Distance to edge of compressive stress block

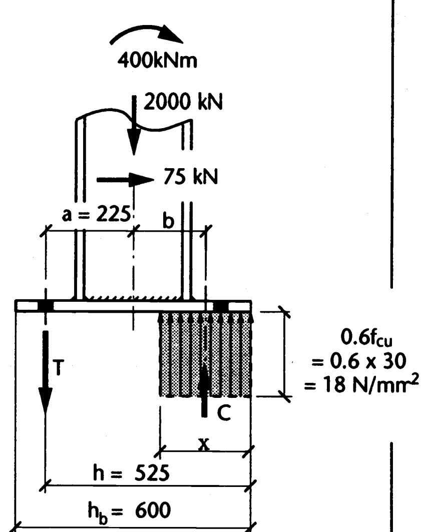
$$\frac{X}{2} = \frac{600}{2} - 200 = 100\text{mm}$$

Compression $= 2 \times 100 \times 600 \times 18\text{N/mm}^2 \times 10^{-3}$

$= 2160\text{kN} \geq 2000\text{kN} \therefore \text{OK}$

and no tension in the bolts

$C = 2000\text{kN}$ and $T = 0$



STEP 1

Title Column Base worked example	Sheet 2 of 4

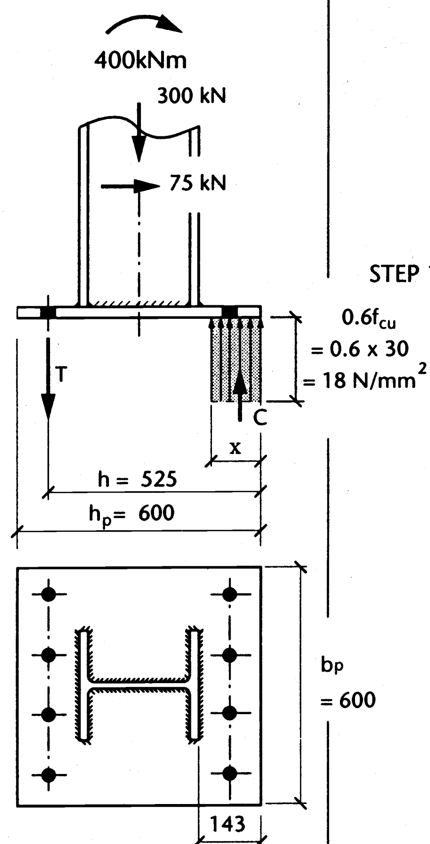
Required design stress = $\dfrac{2000}{600 \times 200}$ = $16.7\ N/mm^2$

$e = L_1 - 0.8s_w = 143 - (0.8 \times 10) = 135mm$ (assuming 10FW)

$m_c = \dfrac{16.7 \times 135^2}{2} = 152178$ Nmm per mm width

(2) MOMENT PLUS MINIMUM AXIAL FORCE:

M = ± 400kNm, Axial force + 300kN



BOLT TENSION AND CONCRETE COMPRESSION STEP 1

$$M = 0.6\,f_{cu}\,b_p X\left(h - \frac{X}{2}\right) - N\left(h - \frac{h_p}{2}\right)$$

substituting values results in the quadratic equation:

$$X^2 - 1050X + (86.5 \times 10^3) = 0$$

where $ax^2 + bx + c = 0$, $\quad x = \dfrac{-b \pm \sqrt{b^2 - 4ac}}{2a}$

Solving for X gives: $X = 90.7mm$

substituting into the equations for C and T gives:

$C = 18 \times 600 \times 90.7 \times 10^{-3} = 980kN$

and $T = 980 - 300 = 680kN$

$m_c = 18 \times 90.7 \times \left(135 - \dfrac{90.7}{2}\right)$

$= 146362$ Nmm per mm width

BASE PLATE THICKNESS STEP 2

The required plate thickness is the larger value resulting from (a) or (b) below

(a) Compression side bending

Required base plate thickness,

$t_p = \sqrt{\dfrac{4m_c}{p_{yp}}}$ where m_c is maximum moment/mm width from (1) & (2) above

$m_c = 152178$ Nmm per mm width

Therefore, $t_p = \sqrt{\dfrac{4 \times 152178}{255}} = 48.8mm$

Title *Column Base worked example*	Sheet *3 of 4*

(b) Tension side bending

Required base plate thickness,

$$t_p = \sqrt{\frac{4m_T}{p_{yp}\ b_p}}$$

$$m_T = T \times m$$

$$m = L_1 - k - 0.8s_w = 143 - 75 - (0.8 \times 10) = 60\text{mm (assuming 10FW)}$$

Hence, $m_T = 680 \times 60 \times 10^3 = 40.8 \times 10^6\text{Nmm}$

Therefore, $t_p = \sqrt{\frac{4 \times 40.8 \times 10^6}{255 \times 600}} = 32.7\text{mm}$

Larger plate thickness from (a) and (b) is 48.8mm, therefore ***Use 50mm plate.***

HOLDING DOWN BOLTS AND ANCHORAGE

STEP 3

Holding down bolts

Force T is assumed to be shared equally between all the bolts in the tension row:

Force per bolt $= \frac{680}{4} = 170\text{kN} < 198\text{kN}$ M24 8.8 bolts at 150mm crs.satisfactory — Table 6.7

Anchorage to concrete

Use anchor plates and check the concrete base for punching shear in accordance with BS 8110.

Anchor plate size (8.8 bolts) $= 5d \times 5d \times 0.8d = 120 \times 120 \times 20\text{mm}$

Assume an effective depth of the holding down bolts, $L = 450 - 50$
(50mm cover to reinforcement)
$= 400\text{mm}$

The perimeter for punching shear check will encompass the group of four bolts:
(it is assumed that there are no free edges which would reduce the perimeter)

$$P = (12 \times L) + P_{ap}$$

where,

$$P_{ap} = \text{total perimeter of anchor plates}$$

$$= 2\left[(3 \times 150) + (2 \times 60)\right] + (2 \times 120) = 1380\text{mm}$$

Therefore, $P = (12 \times 400) + 1380 = 6180\text{mm}$

Average shear stress

$$f_v = \frac{T}{P \times L}$$

$$= \frac{680 \times 10^3}{6180 \times 400} = 0.28\text{N/mm}^2$$

Assuming an area of tension reinforcement less than or equal to 0.15%

Design concrete shear stress, v_c taken from table 3.9 of BS 8110

$$v_c = 0.35\text{N/mm}^2 > 0.28\text{N/mm}^2 \text{ OK}$$

Provide 4 M24 8.8 holding down bolts

Overall embedment depth in the concrete (excluding the grout beneath the base plate) is 450mm.

Title *Column Base worked example*	Sheet 4 of 4

SHEAR TRANSFER TO CONCRETE

STEP 4

Check if the horizontal shear is transferred by friction.

Available shear resistance = $0.3 \times C\,(min) = 0.3 \times 300 = 90kN > 75kN$

O.K.

WELDS - BASE PLATE TO COLUMN

STEP 5

Tension flange weld

Force in the tension flange welds is the lesser of:

(a) The tension capacity of the flange $= B_c \times T_c \times p_y = 306.8 \times 18.7 \times 265 \times 10^{-3} = 1520kN$

(b) The force in the tension flange $= \dfrac{M}{D_C - T_C} - N \times \dfrac{A_F}{A_C}$

$$= \frac{400 \times 10^3}{314 - 18.7} - 300 \times \frac{5737}{15000} = 1240kN$$

Therefore, Weld force per mm $= \dfrac{1240}{(2 \times 306.8) - 11.9} = 2.06kN/mm$

Weld throat required at $215N/mm^2$ $= \dfrac{2.06 \times 10^3}{215} = 9.6mm$

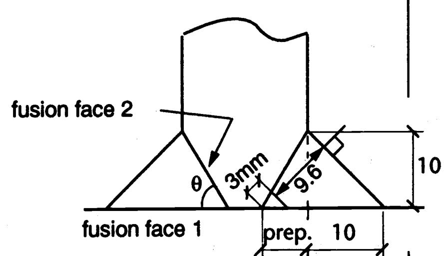
If based on a 10mm superimposed fillet weld:

preparation $= \sqrt{2 \times (9.6 + 3)^2} - 10 = 7.8mm$

length of fusion face 1 $= 10 + 7.8 - 3 = 14.8mm$

length of fusion face 2 $= \sqrt{7.8^2 + 10^2} - 3 = 9.7mm$

$\theta = \tan^{-1}(10 / 7.8) = 52°, > 45°$, OK



Fusion face 1. tensile force $= 2.06kN/mm$

tensile stress $= \dfrac{2.06 \times 10^3}{14.8} = 139N/mm^2 < 265N/mm^2$, OK

Fusion face 2. tensile force $= 2.06 \times \cos 52° = 1.27kN/mm$

shear force $= 2.06 \times \sin 52° = 1.62kN/mm$

tensile stress $= \dfrac{1.27 \times 10^3}{9.7} = 131N/mm^2 < 265N/mm^2$, OK

shear stress $= \dfrac{1.62 \times 10^3}{9.7} = 167N/mm^2$

allowable $= 0.7 \times 265 = 185N/mm^2 > 167N/mm^2$, OK

Provide partial penetration butt welds (8mm preparation) with 10mm superimposed fillet welds.

Compression flange weld

Assuming bearing contact, nominal welds only are required. However, since the moment is reversible, the tension weld must be made to both flanges.

Web welds

Assuming bearing contact to transfer the axial force, then by inspection 8mm fillet weld is adequate for the applied shear of 75kN.

Provide 8mm fillet weld both sides of the web.

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APPENDICES

APPENDIX I WORKED EXAMPLE - BOLTED END PLATE
USING THE RIGOROUS METHOD

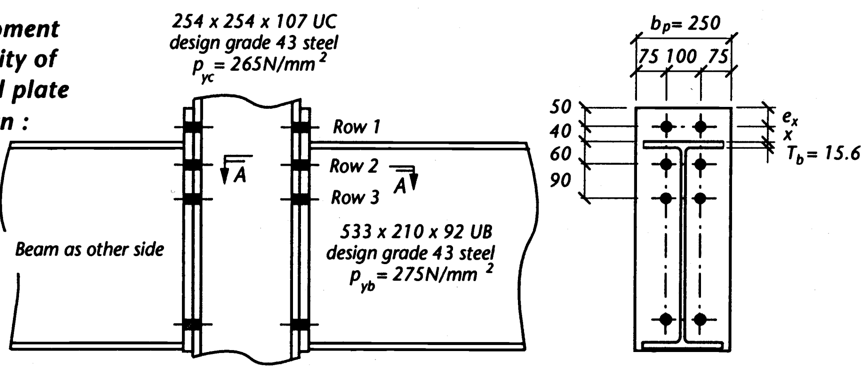
This example has been made using the full rigorous procedures in Section 2.8. It also includes the design of the various stiffeners covered in the procedures but, in order to show the use of stiffeners, the example has been changed where necessary to illustrate the application of each type.

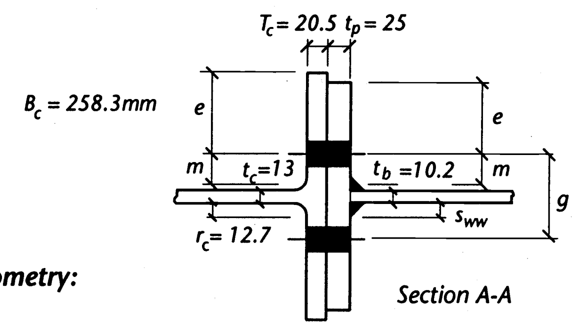
The calculation sheets are numbered consecutively and show:

Calculation Sheets 1 - 13	Calculation of the moment and shear capacity of a typical standard **Extended End-Plate** type connection of a 533 x 210 x 92 UB beam connecting to a 254 x 254 x 107 UC column using three tensile bolt rows.
Calculation Sheets 14 - 16	The column section is 254 x 254 x 107 UC, as in the previous example, but the compression force is increased to show the use of **Compression Stiffeners.**
Calculation Sheets 17 - 22	The column section is assumed to be 254 x 254 x 73 UC to show the use of **Column Flange Backing Plates.**
Calculation Sheets 23 - 24	The column section is assumed to be 254 x 254 x 73 UC to show the use of **Tension Rib Stiffeners.**
Calculation Sheets 25 - 26	The column section is 254 x 254 x 107 UC, as in the first example, but the web panel shear is increased to show the use of **Supplementary Web Plates.**
Calculation Sheets 27 - 28	The column section is assumed to be a 686 x 254 x 125 UB with the web panel shear force increased to show the use of **Morris Stiffeners.**

CALCULATION SHEET BCSA	Job: *Moment Connections*		Sheet: *1 of 28*
	Title: *Worked example for a bolted end plate using the rigorous method*		
	Client: *SCI/BCSA Connections Group*		
	Calcs by: *N C*	Checked by: *AM*	Date: *May 95*

Calculate the moment and shear capacity of the extended end plate connection shown :





660 x 250 x 25 end plate
design grade 43 steel

p_{yp} = 265 kN/mm^2

M24 8.8 Bolts

Shaded black = tensile bolts

Shaded grey = shear bolts

Connection geometry:

Column side:

$$m = \frac{g}{2} - \frac{t_c}{2} - 0.8r_c = 50 - 6.5 - 10.2 = 33.3\text{mm}$$

$$e = \frac{B_c}{2} - \frac{g}{2} = \frac{(258.3 - 100)}{2} = 79.2\text{mm}$$

n = smallest of e (column flange), e (end plate) or $1.25m$ (column flange)

$= 79.2, 75, 1.25 \times 33.3 = 41.6\text{mm}$

Beam side:

$$m = \frac{g}{2} - \frac{t_b}{2} - 0.8s_{ww} = 50 - 5.1 - 6.4 = 38.5\text{mm (assume 8FW)}$$

$$e = \frac{b_p}{2} - \frac{g}{2} = \frac{(250 - 100)}{2} = 75\text{mm}$$

n = smallest of e (column flange), e (end plate) or $1.25m$ (end plate)

$= 79.2, 75, 1.25 \times 38.5 = 48.1\text{mm}$

For the end plate extension only:

$$m_x = X - 0.8s_{wf} = 40 - 9.6 = 30.4\text{mm (assume 12FW)}$$

$$e_x = 50\text{mm}$$

n_x = smaller of e_x or $1.25m_x$ $= 1.25 \times 30.4 = 38\text{mm}$

Title	Worked example for a bolted end plate using the rigorous method	Sheet	2 of 28

POTENTIAL RESISTANCE OF BOLTS IN TENSION ZONE

STEP 1

BOLT ROW 1

Column flange bending

STEP 1A

Calculate effective length of T-stub. The bolt row is not influenced by a stiffener or a free end. From tables 2.5 and 2.4, L_{eff} is the minimum of:

$$2\pi m = 2 \times \pi \times 33.3 = 209mm \; *$$

or $$4m + 1.25e = (4 \times 33.3) + (1.25 \times 79.2) = 232mm$$

Min. values indicated thus *

Calculate M_p for the column flange.

$$M_p = \frac{L_{eff} \times T_c^2 \times p_{yc}}{4} = \frac{209 \times 20.5^2 \times 265 \times 10^{-3}}{4} = 5819kNmm$$

Find the critical failure mode. This is the minimum of the following three formulae:

Mode 1: $$P_r = \frac{4M_p}{m} = \frac{4 \times 5819}{33.3} = 699kN$$

Mode 2: $$P_r = \frac{2M_p + n\Sigma P'_t}{m+n} = \frac{(2 \times 5819) + (41.6 \times 2 \times 198)}{33.3 + 41.6} = 375kN \; *$$

375	1

Value inserted in worksheet on page115

Mode 3: $$P_r = \Sigma P_t' = 2 \times 198 = 396kN$$

Column web tension

STEP 1B

$$P_t = L_t \times t_c \times p_{yc}$$

L_t is the tensile length of web assuming a spread of load of 1:1.73 from the bolts.

$$L_t = \frac{g}{2} \times 1.73 \times 2 = \frac{100}{2} \times 1.73 \times 2 = 173mm$$

$$P_t = 173 \times 13.0 \times 265 \times 10^{-3} = 596kN$$

596	2

End plate bending

STEP 1A

Calculate effective length of T-stub. Row 1 is situated in the extension of the end plate. From tables 2.5 and 2.4, L_{eff} is the minimum of:

$$\frac{b_p}{2} = \frac{250}{2} = 125mm \; *$$

or $$2m_x + 0.625e_x + g/2 = (2 \times 30.4) + (0.625 \times 50) + 100/2 = 142mm$$

or $$2m_x + 0.625e_x + e = (2 \times 30.4) + (0.625 \times 50) + 75 = 167mm$$

or $$4m_x + 1.25e_x = (4 \times 30.4) + (1.25 \times 50) = 184mm$$

or $$2\pi m_x = 2 \times \pi \times 30.4 = 191mm$$

Calculate M_p for the end plate.

$$M_p = \frac{L_{eff} \times t_p^2 \times p_{yp}}{4} = \frac{125 \times 25^2 \times 265 \times 10^{-3}}{4} = 5176kNmm$$

Title *Worked example for a bolted end plate using the rigorous method*	Sheet 3 of 28

Find the critical failure mode. This is the minimum of the following 3 formulae.

STEP 1A

Mode 1: $P_r = \dfrac{4M_p}{m_x} = \dfrac{4 \times 5176}{30.4} = 681kN$

Mode 2: $P_r = \dfrac{2M_p + n_x \Sigma P_t'}{m_x + n_x} = \dfrac{(2 \times 5176) + (38.0 \times 2 \times 198)}{30.4 + 38.0} = 371kN$ *

| 371 | 3 |

Mode 3: $P_r = \Sigma P_t' = 2 \times 198 = 396kN$

Beam web tension

STEP 1B

Since row 1 is in the extension, beam web tension does not apply.

| N/A | 4 |

Triangular limit

STEP 1C

The triangular limit does not apply to row 1 of an extended end plate

| N/A | 5 |

Hence, the potential resistance of row 1, P_{r1} is the smallest of the values from boxes 1 to 4

i.e. 375kN, 596kN, 371kN

Therefore the Potential Resistance of row 1, P_{r1} = **371kN**

| 371 | 6 |

BOLT ROW 2

Row 2 alone

Column flange bending

STEP 1A

P_r is calculated as for row 1.

Therefore, P_r = 375KN

| 375 | 7 |

Column web tension

STEP 1B

As before, P_t = 596kN

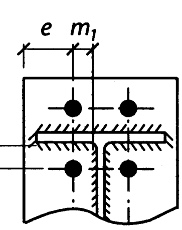
| 596 | 8 |

End plate bending

STEP 1A

Calculate effective length of T-stub. Row 2 is below the beam flange of an extended end plate. From tables 2.5 and 2.4, L_{eff} is given by:

Min{Max{pattern ii, pattern iii},pattern i}



Pattern (ii): $4m + 1.25e = (4 \times 38.5) + (1.25 \times 75) = 248mm$

Pattern (iii): αm_1 α is obtained from figure 2.16 using the following parameters:

$m_1 = m = 38.5mm$

$m_2 = 60 - 15.6 - (0.8 \times 12) = 34.8mm$

$\lambda_1 = \dfrac{m_1}{m_1 + e} = \dfrac{38.5}{38.5 + 75} = 0.34$

$\lambda_2 = \dfrac{m_2}{m_1 + e} = \dfrac{34.8}{38.5 + 75} = 0.31$

the chart shows $\alpha = 2\pi$

$\therefore \alpha m_1 = 2\pi \times 38.5 = 242mm$

the maximum of patterns (ii) and (iii) = 248mm

Pattern (i): $2\pi m = 2 \times \pi \times 38.5 = 242mm$ *

$M_p = \dfrac{L_{eff} \times t_p^2 \times p_{yp}}{4} = \dfrac{242 \times 25^2 \times 265 \times 10^{-3}}{4} = 10020kNmm$

Title	Worked example for a bolted end plate using the rigorous method	Sheet	4 of 28

Mode 1: $P_r = \frac{4M_p}{m} = \frac{4 \times 10020}{38.5} = 1041kN$

Mode 2: $P_r = \frac{2M_p + n\Sigma P_t'}{m+n} = \frac{(2 \times 10020) + (48.1 \times 2 \times 198)}{38.5 + 48.1} = 451kN$

Mode 3: $P_r = \Sigma P_t' = 2 \times 198 = 396kN^*$ — 396 | 9

Beam web tension — STEP 1B

Row 2 is situated below the beam flange, the underside of which is only 44mm from the bolt row. This would place the flange within the tensile length and therefore beam web tension can be discounted. — N/A | 10

Rows 1 + 2 combined

Column flange bending — STEP 1A

Calculate effective length of T–stub.
Neither row is influenced by stiffeners or free edges. From tables 2.6 and 2.4, L_{eff} for the group is given by:

$$L_{eff} = 2\left(\frac{ii}{2} + \frac{p}{2}\right)$$

$$L_{eff} = 2 \times (2m + 0.625e + p/2) = 2 \times [(2 \times 33.3) + (0.625 \times 79.2) + (100/2)]$$

$$= 332mm$$

Hence, $M_p = \frac{L_{eff} \times T_c^2 \times p_{yc}}{4} = \frac{332 \times 20.5^2 \times 265 \times 10^{-3}}{4} = 9243kNmm$

Critical failure mode:

Mode 1: $P_r = \frac{4M_p}{m} = \frac{4 \times 9243}{33.3} = 1110kN$

Mode 2: $P_r = \frac{2M_p + n\Sigma P_t'}{m+n} = \frac{(2 \times 9243) + (41.6 \times 4 \times 198)}{33.3 + 41.6} = 687kN^*$ — 687 | 11

Mode 3: $P_r = \Sigma P_t' = 4 \times 198 = 792kN$

P_r for row 2 is taken as the minimum from modes 1 to 3 minus P_{r1}

Therefore, $P_r = 687 - 371 = 316kN$ — 316 | 15

Column web tension — STEP 1B

$$P_t = L_t \times t_c \times p_{yc}$$

$$L_t = \left[\frac{g}{2} \times 1.73 \times 2\right] + p = \left[\frac{100}{2} \times 1.73 \times 2\right] + 100 = 273mm$$

$$P_{t(1+2)} = 273 \times 13.0 \times 265 \times 10^{-3} = 940kN$$ — 940 | 12

For row 2, $P_t = P_{t(1+2)} - P_{r1} = 940 - 371 = 569kN$ — 569 | 16

End plate bending — STEP 1A

Not applicable. (see Figure 2.14) — N/A | 13; N/A | 17

Title *Worked example for a bolted end plate using the rigorous method*	Sheet *5 of 28*

Beam web tension — STEP 1B

Not applicable. (see Figure 2.17) — [N/A | 14]

Triangular limit — [N/A | 18]

Since the colummn flange thickness does not exceed 21.9mm (Table 2.7), the triangular limit does not apply. — [N/A | 19]

The potential resistance for row 2, P_{r2} is the smallest of the values from boxes 7 to 10 and 15 to 19

i.e. 375kN, 596kN, 396kN, 316kN, 569kN

Therefore the Potential Resistance of row 2, P_{r2} = ***316kN*** — [316 | 20]

BOLT ROW 3

Row 3 alone

Column flange bending — STEP 1A

P_r is calculated as for row 1.

Therefore, P_r = *375kN* — [375 | 21]

Column web tension — STEP 1B

P_t = *596kN* — [596 | 22]

End plate bending — STEP 1A

Calculate effective length of T-stub. The bolt row is not influenced by a stiffener or a free end. From tables 2.5 and 2.6, L_{eff} is the minimum of:

$2\pi m = 2 \times \pi \times 38.5 = 242mm$ *

or $4m + 1.25e = (4 \times 38.5) + (1.25 \times 75) = 248mm$

Therefore P_r is the same as for row 2 alone,

P_r = *396kN* — [396 | 23]

Beam web tension — STEP 1B

$$P_t = L_t \times t_b \times p_{yb}$$

$$L_t = \frac{g}{2} \times 1.73 \times 2 = \frac{100}{2} \times 1.73 \times 2 = 173mm$$

$$P_t = 173 \times 10.2 \times 275 \times 10^{-3} = 485kN$$

[485 | 24]

Rows 2 + 3 combined

Column flange bending — STEP 1A

Calculate effective length of T–stub. Neither row is influenced by stiffeners or free edges. From tables 2.6 and 2.4, L_{eff} for the group is given by:

$$L_{eff} = 2\left(\frac{ii}{2} + \frac{p}{2}\right)$$

$$L_{eff} = 2 \times (2m + 0.625e + P/2) = 2 \times [(2 \times 33.3) + (0.625 \times 79.2) + (90/2)] = 322mm$$

$$\text{Hence } M_p = \frac{L_{eff} \times T_c^2 \times p_{yc}}{4} = \frac{322 \times 20.5^2 \times 265 \times 10^{-3}}{4} = 8965kNmm$$

Title	*Worked example for a bolted end plate using the rigorous method*	Sheet	*6 of 28*

Critical failure mode:

Mode 1: $$P_r = \frac{4M_p}{m} = \frac{4 \times 8965}{33.3} = 1077\text{kN}$$

Mode 2: $$P_r = \frac{2M_p + n\Sigma P_t'}{m+n} = \frac{(2 \times 8965) + (41.6 \times 4 \times 198)}{33.3 + 41.6} = 679\text{kN}^*$$ 679 | 25

Mode 3: $$P_r = \Sigma P_t' = 4 \times 198 = 792\text{kN}$$

$$\therefore \; P_r = (\text{min. modes 1 to 3}) - P_{r2} = 679 - 316 = 363\text{kN}$$ 363 | 29

Column web tension

$$P_t = L_t \times t_c \times p_{yc}$$

$$L_t = \left[\frac{g \times 1.73 \times 2}{2}\right] + p = \left[\frac{100 \times 1.73 \times 2}{2}\right] + 90 = 263\text{mm}$$

$$P_{t(2+3)} = 263 \times 13 \times 265 \times 10^{-3} = 906\text{kN}$$ 906 | 26

For row 3, $$P_t = P_{t(2+3)} - P_{r2} = 906 - 316 = 590\text{kN}$$ 590 | 30

End plate bending

STEP 1A

Calculate effective length of T-stub. Row 2 is adjacent to a beam flange. Row 3 is not influenced by a stiffener or a free edge. From tables 2.6 and 2.4, L_{eff} is given by:

$$\text{Max}\left\{\frac{ii}{2}, \left(iii - \frac{ii}{2}\right)\right\} + \frac{p}{2} + \frac{ii}{2} + \frac{p}{2}$$

$$\text{i.e. } L_{eff} = \frac{4m + 1.25e}{2} + \frac{4m + 1.25e}{2} + p$$

$$= 4m + 1.25e + p$$

$$= (4 \times 38.5) + (1.25 \times 75) + 90 = 338\text{mm}$$

$$\text{or } L_{eff} = \left(\alpha m_1 - \frac{(4m + 1.25e)}{2}\right) + \frac{p}{2} + \frac{4m + 1.25e}{2} + \frac{p}{2}$$

$$= \alpha m_1 + p \quad \text{and } \alpha \text{ (as for row 2 alone)} = 2\pi$$

Page 109

$$= (2 \times \pi \times 38.5) + 90 = 332\text{mm}$$

$$\text{hence } L_{eff} = 338\text{mm}$$

and, $$M_p = \frac{L_{eff} \times t_p^2 \times p_{yp}}{4} = \frac{338 \times 25^2 \times 265 \times 10^{-3}}{4} = 13995\text{kNmm}$$

Mode 1: $$P_r = \frac{4M_p}{m} = \frac{4 \times 13995}{38.5} = 1454\text{kN}$$

Mode 2: $$P_r = \frac{2M_p + n\Sigma P_t'}{m+n} = \frac{(2 \times 13995) + (48.1 \times 4 \times 198)}{38.5 + 48.1} = 763\text{kN}^*$$ 763 | 27

Mode 3: $$P_r = \Sigma P_t' = 4 \times 198 = 792\text{kN}$$

P_r for row 3 is taken as the minimum from modes 1 to 3 minus P_{r2}

Therefore, $$P_r = 763 - 316 = 447\text{kN}$$ 447 | 31

Title	Worked example for a bolted end plate using the rigorous method	Sheet	7 of 28

Beam web tension — STEP 1B

Not applicable. (stiffener (beam flange) is within the tensile length L_t)

N/A [28]
N/A [32]

Rows 1 + 2 + 3 combined

Column flange bending — STEP 1A

Calculate effective length of T-stub. The group is not influenced by a stiffener or free end. From tables 2.6 and 2.4, L_{eff} is given by (see typical example in table 2.6 – but note p varies):

$$L_{eff} = 4m + 1.25e + p_{1-2} + p_{2-3}$$

$$= (4 \times 33.3) + (1.25 \times 79.2) + 100 + 90 \quad = \quad 422\text{mm}$$

Hence, $$M_p = \frac{L_{eff} \times T_c^2 \times p_{yc}}{4} = \frac{422 \times 20.5^2 \times 265 \times 10^{-3}}{4} \quad = \quad 11749\text{kNmm}$$

Mode 1: $$P_r = \frac{4M_p}{m} = \frac{4 \times 11749}{33.3} \quad = \quad 1411\text{kN}$$

Mode 2: $$P_r = \frac{2M_p + n\Sigma P_t'}{m+n} = \frac{(2 \times 11749) + (41.6 \times 6 \times 198)}{33.3 + 41.6} \quad = \quad 974\text{KN}\ *$$

974 [33]

Mode 3: $$P_r = \Sigma P_t' = 6 \times 198 \quad = \quad 1188\text{kN}$$

P_r for row 3 is taken as the minimum from modes 1 to 3 minus P_{r1} minus P_{r2}

Therefore, $$P_r = 974 - 371 - 316 \quad = \quad 287\text{kN}$$

287 [37]

Column web tension — STEP 1B

$$P_t = L_t \times t_c \times p_{yc}$$

$$L_t = \left[\frac{g}{2} \times 1.73 \times 2\right] + p_{1-2} + p_{2-3}$$

$$= \left[\frac{100}{2} \times 1.73 \times 2\right] + 100 + 90 \quad = \quad 363\text{mm}$$

$$P_{t\,(1+2+3)} = 363 \times 13.0 \times 265 \times 10^{-3} \quad = \quad 1251\text{kN}$$

1251 [34]

For row 3, $$P_t = P_{t(1+2+3)} - P_{r1} - P_{r2} = 1251 - 371 - 316 \quad = \quad 564\text{kN}$$

564 [38]

End plate bending — STEP 1A

Not applicable. (see Figure 2.14)

N/A [35]
N/A [39]

Title	*Worked example for a bolted end plate using the rigorous method*	Sheet	*8 of 28*

Beam web tension

STEP 1B

Not applicable.(stiffener (beam flange) is within the tensile length L_t)

N/A	36
N/A	40

Triangular limit

Since the column flange thickness does not exceed 21.9mm (Table 2.7), the triangular limit does not apply.

N/A	41

The potential resistance for row 3, P_{r3} is the smallest of the values from boxes 21 to 24, 29 to 32 and 37 to 41.

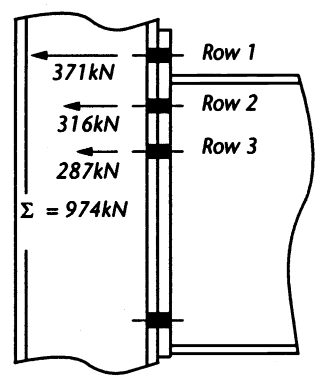
i.e. 375kN, 596kN, 396kN,485kN; 363kN, 590kN, 447kN; 287kN, 564kN

Therefore the Potential Resistance of row 3, P_{r3} = **287kN**

287	42

Distribution of bolt forces

Step 1 has produced the distribution of potential bolt forces shown:



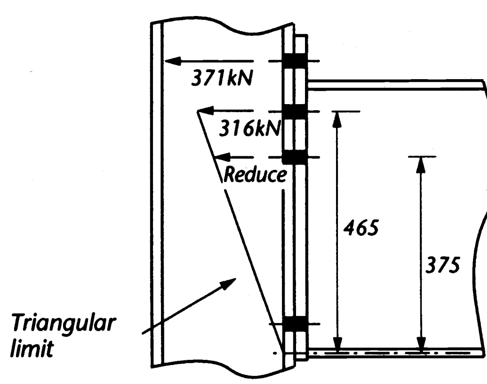
$$\Sigma P_{ri} = 974kN$$

STEP 1C

If both T_c and t_p had been too thick for full plastic distribution (see eq 2.5 and 2.6), the force distribution would have been limited as follows (shown for illustration purposes only).

By similar triangles, P_{r3} for row 3 is limited by:

$$P_{r3} = 316 \times \frac{375}{465}$$
$$= 255kN$$



Title	Sheet
Worked example for a bolted end plate using the rigorous method	*9 of 28*

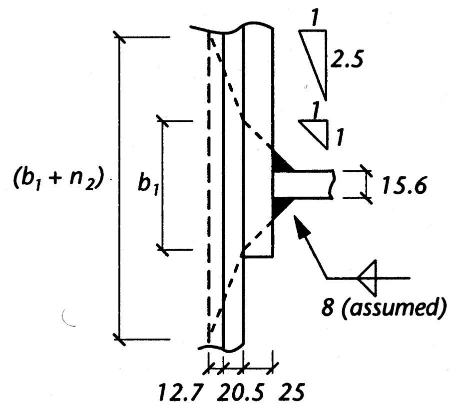
Worksheet for Step 1 : Tension Zone

Row	Column Side: STEP 1A Flange Bending	Column Side: STEP 1 B Web Tension	Beam Side: STEP 1A Plate Bending	Beam Side: STEP 1B Web Tension	STEP 1C Triangular Limit	Potential Resistance
1	*Resistance of Row 1*					least of boxes 1 to 4 gives
	375 [1]	596 [2]	371 [3]	N/A [4]	N/A [5]	P_{r1} = 371 [6]
2	*Resistance of row 2 alone:*					
	375 [7]	596 [8]	396 [9]	N/A [10]		
	Resistance of rows (1 + 2) combined:					least of boxes: 7 to 10 and 15 to 18 gives
	687 [11]	940 [12]	N/A [13]	N/A [14]		
	Deduct box 6:					
	316 [15]	569 [16]	N/A [17]	N/A [18]	N/A [19]	P_{r2} = 316 [20]
3	*Resistance of row 3 alone:*					
	375 [21]	596 [22]	396 [23]	485 [24]		
	Resistance of combined rows (2+3):					
	679 [25]	906 [26]	763 [27]	N/A [28]		
	Deduct box 20:					least of boxes 21 to 24 and 29 to 32 and 37 to 41 gives
	363 [29]	590 [30]	447 [31]	N/A [32]		
	Resistance of combined rows (1+2+3):					
	974 [33]	1251 [34]	N/A [35]	N/A [36]		
	Deduct sum of boxes 6 & 20:					
	287 [37]	564 [38]	N/A [39]	N/A [40]	N/A [41]	P_{r3} = 287 [42]
4	*Resistance of row 4 alone:*					
	[43]	[44]	[45]	[46]		
	Resistance of combined rows (3+4):					
	[47]	[48]	[49]	[50]		
	Deduct box 42:					
	[51]	[52]	[53]	[54]		
	Resistance of combined rows (2+3+4):					
	[55]	[56]	[57]	[58]		
	Deduct sum of boxes 20 & 42:					least of boxes 43 to 46 and 51 to 54 and 59 to 62 and 67 to 71 gives
	[59]	[60]	[61]	[62]		
	Resistance of combined rows (1+2+3+4):					
	[63]	[64]	[65]	[66]		
	Deduct sum of boxes 6, 20 & 42:					
	[67]	[68]	[69]	[70]	[71]	P_{r4} = [72]

Title	*Worked example for a bolted end plate using the rigorous method*	Sheet	*10 of 28*

RESISTANCE OF THE COLUMN WEB & BEAM FLANGE IN THE COMPRESSION ZONE

The compressive resistance, P_c is the minimum of the following three values:



(1) Column web crushing (bearing) STEP 2A

$P_c = (b_1 + n_2) \times t_c \times p_{yc}$

$b_1 = 15.6 + (2 \times 8) + (2 \times 25)$

$= 81.6\text{mm}$

$n_2 = [\,2.5 \times (20.5 + 12.7)\,] \times 2$

$= 166\text{mm}$

$\therefore P_c = (81.6 + 166) \times 13.0 \times 265 \times 10^{-3} \quad = \quad 853\text{kN}^{*}$

(2) Column web buckling STEP 2A

$P_c = (b_1 + n_1) \times t_c \times p_c$

$b_1 = 81.6\text{mm}$

$n_1 = \text{depth of column} = 266.7\text{mm}$

p_c is obtained from table 27(c) of BS 5950 using:

$$\lambda = \frac{2.5d}{t_c} = \frac{2.5 \times 200.3}{13.0} = 38.5$$

$\therefore p_c = 233\text{N/mm}^2 \quad (\text{when } p_{yc} = 265\text{N/mm}^2)$

and, $P_c = (81.6 + 266.7) \times 13.0 \times 233 \times 10^{-3} \quad = \quad 1055\text{kN}$

(3) Beam flange crushing (bearing) STEP 2B

The resistance of the beam flange in compression is given by:

$P_c = 1.4 \times p_{yb} \times T_b \times B_b$

$= 1.4 \times 275 \times 15.6 \times 209.3 \times 10^{-3} \quad = \quad 1257\text{kN}$

Therefore the resistance in the compression zone, $P_c = 853\text{kN}$ — $P_c = 853\text{kN}$

RESISTANCE OF THE COLUMN WEB PANEL IN SHEAR **STEP 3**

The resistance of the column web panel in shear, P_v is given by:

$P_v = 0.6 \times p_{yc} \times A_v$

$A_v = t_c \times D_c = 13.0 \times 266.7 = 3467\text{mm}^2$

Therefore, $P_v = 0.6 \times 265 \times 3467 \times 10^{-3} \quad = 551\text{kN}$ — $P_v = 551\text{kN}$

Title *Worked example for a bolted end plate using the rigorous method*	Sheet *11 of 28*

CALCULATION OF MOMENT CAPACITY

STEP 4

Force distribution

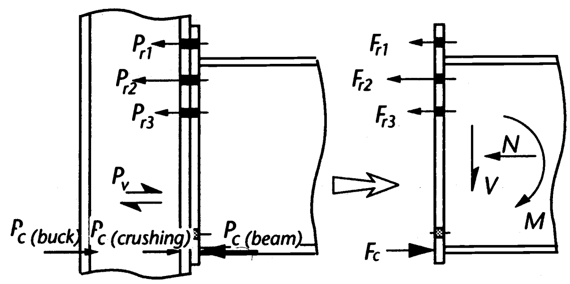
The potential resistances must now be translated into a set of internal forces (in horizontal equilibrium) from which the moment capacity of the connection can be calculated.

Horizontal equilibrium is satisfied by:

$$\Sigma F_{ri} + N = F_c$$

where F_c is the smallest of:

$$\Sigma P_{ri} + N = 974kN \quad (N = 0)$$
$$P_c = 853kN$$
$$P_v = 551kN$$



The web panel shear capacity P_v, would be the critical value in a one sided connection. Horizontal equilibrium would be achieved by reducing the sum of the bolt row forces, commencing by reducing the lowest row. If the moment capacity of the connection was then found to be insufficient, web strengthening could be provided, or a revised connection configuration chosen.

In this example of a two sided connection with equal and opposite applied moments, the column web panel shear is zero, and P_v is not critical.

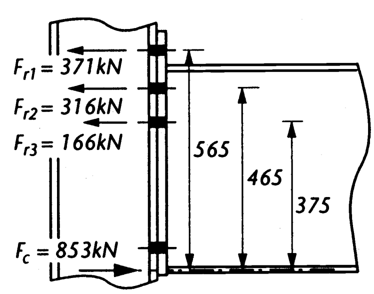
Since web panel shear is zero in this example the compressive resistance P_c is critical. In order to satisfy horizontal equilibrium, ΣF_{ri} must equal F_c of 853kN. The values of P_{ri} must be reduced in order to achieve equilibrium

$\Sigma P_{ri} = 371 + 316 + 287 \qquad = 974kN$

$\therefore$ Reduce ΣP_{ri} by a total of $974 - 853 \qquad = 121kN.$

Starting with the lowest row and working up, all of the 121kN can be taken from row 3, leaving $F_{r3} = 287 - 121 = 166kN$

This results in the following internal horizontal force distribution:



Moment capacity

The moment capacity of the connection is:

$$M_c = \Sigma (F_{ri} \times h_i)$$
$$= (371 \times 0.565) + (316 \times 0.465) + (166 \times 0.375) \qquad = \mathbf{419kNm}$$

Moment Capacity Mc = 419kNm

Title	*Worked example for a bolted end plate using the rigorous method*	Sheet	*12 of 28*

DESIGN FOR VERTICAL SHEAR FORCE

STEP 5

The vertical shear capacity of the connection is:

$$P_v = n_s P_{ss} + n_t P_{ts}$$

Bolt capacity table is on page 221

P_{ss} is the shear capacity of a single bolt in the shear zone and is the lesser of:

$$p_s A_s = 132\text{kN} \; *$$

$$dt_p p_b = 24 \times 25 \times 460 \times 10^{-3} = 276\text{kN}$$

$$dT_c p_b = 24 \times 20.5 \times 460 \times 10^{-3} = 226\text{kN}$$

P_{ts} is the shear capacity of a single bolt in the tension zone and is the lesser of:

$$0.4 p_s A_s = 0.4 \times 132 = 53\text{kN} \; *$$

$$dt_p p_b = 24 \times 25 \times 460 \times 10^{-3} = 276\text{kN}$$

$$dT_c p_b = 24 \times 20.5 \times 460 \times 10^{-3} = 226\text{kN}$$

Therefore, $P_v = (2 \times 132) + (6 \times 53) = \mathbf{582kN}$

Shear Capacity, P_v = 582kN

STEP 6 ***Stiffener Designs are illustrated by the examples on sheets 14 to 28***

WELD DESIGN

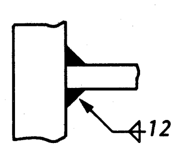
STEP 7

Tension flange welds

Provide a full strength fillet weld.

$$\text{leg length} = \frac{T_b}{2 \times 0.7} = \frac{15.6}{2 \times 0.7} = 11.1\text{mm}$$

Use 12mm FW.



Compression flange weld

Provide a bearing fit and provide nominal fillet welds. ***Use 8mm FW.***

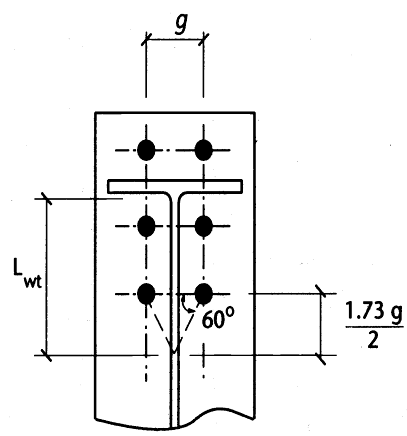
Title *Worked example for a bolted end plate using the rigorous method*	Sheet *13 of 28*

Web welds

For webs up to 11.3mm an 8mm fillet weld each side provides full strength and can be carried down the full web depth. The following is provided for illustration purposes.

(1) Tension Zone

Tension in the bottom bolt row is considered as dispersed at an angle of 60° thus:



$$L_{wt} = (60 - T_b - r_b) + 90 + (1.73 \times 100/2)$$

$$= (60 - 15.6 - 12.7) + 90 + 87 \quad = \quad 209\text{mm}$$

Leg length of fillet welds providing full strength $= \dfrac{t_b}{2 \times 0.7} = \dfrac{10.2}{2 \times 0.7} = 7.3\text{mm}$

Use 8mm FW.

(2) Shear Zone

If the 8mm fillet is continued for the full depth, the capacity of the beam web weld for vertical shear is given by:

$$P_{sw} = 2 \times a \times p_w \times L_{ws}$$

$$L_{ws} = D_b - 2(T_b + r_b) - L_{wt}$$

$$= 533.1 - 2(15.6 + 12.7) - 209 \quad = 268\text{mm}$$

$$\therefore p_{sw} = 2 \times (0.7 \times 8) \times 215 \times 268 \times 10^{-3} \quad = 645\text{kN}$$

Therefore, with a 8mm fillet weld in the shear zone providing a resistance of 645kN, the shear capacity of the connection is limited by the shear resistance of the bolts, 582kN (sheet 12).

Mar. 97 Revision: *tension and shear zone modified*

CALCULATION SHEET BCSA	Job: Moment Connections	Sheet: 14 of 28
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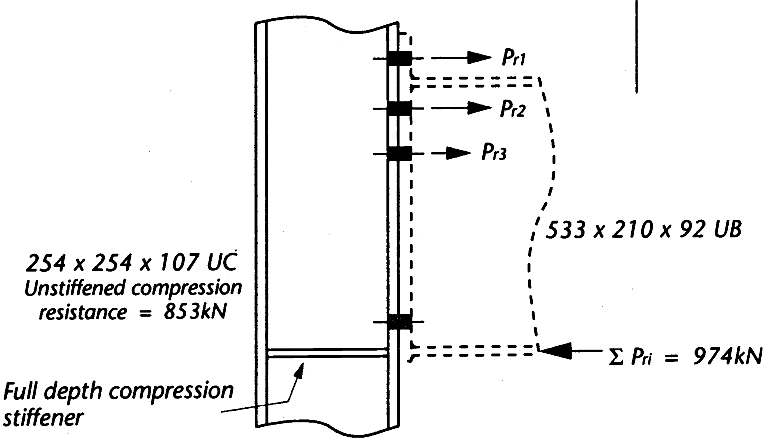
	Calcs by: RS / Checked by: AM	Date: May 95

STEP 6A

DESIGN OF COMPRESSION STIFFENERS

Design full depth compression stiffeners for the column used in the design example - sheets 1-13.

They must resist the sum of the potential bolt row resistances derived in STEP 1 on sheet 8 and as shown:



Basic requirement is:

$$P_c \geq \Sigma P_{ri}$$

where: P_c = the potential compression resistance of the stiffened column

= lesser of formulae (2.11), (2.12) or (2.13) from STEP 6A (See page 32)

ΣP_{ri} = sum of the potential bolt row resistances from sheet 8 (STEP 1C)

= 974kN

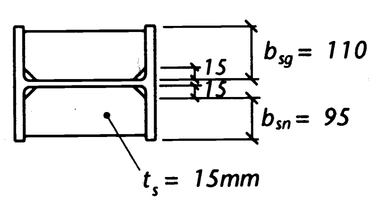
Make a first guess for the stiffener thickness.
Although ΣP_{ri} (974kN) is only slightly greater than the unstiffened compression resistance of the column (853kN), by inspection the 80% rule will govern. It is required that:

$$A_{sn} \geq \frac{\Sigma P_{ri} \times 0.8}{p_{ys}}$$

$$\text{ie } A_{sn} \geq \frac{974 \times 10^3 \times 0.8}{275}$$

$$A_{sn} \geq 2833\text{mm}^2$$

Try 2/110 x 15 stiffeners thus:
(Grade 43)



$$A_{sn} = 2 \times 95 \times 15 = 2850\text{mm}^2 \quad \text{OK}$$

Title *Worked example for a bolted end plate using the rigorous method*	Sheet *15 of 28*

Stiffener buckling

$P_{c\,buckling} = (A_w + A_{sg}) \times p_c$

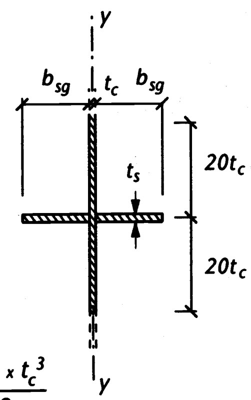
where: A_w = *Allowable column web buckling area* $= 40t_c \times t_c$

$= (40 \times 13.0) \times 13.0 = 6760mm^2$

A_{sg} = *gross area of stiffeners* $= 2 \times b_{sg} \times t_s$

$2 \times 110 \times 15 = 3300mm^2$

p_c *is obtained from table 27(c) of BS 5950 Pt1 using:*



$$\lambda = \frac{0.7 \times L}{r_y}$$

L = *length of stiffener* $= D_c - 2T_c = 226mm$

r_y = *radius of gyration of effective area shown opposite*

$$= \sqrt{\frac{I}{A}}$$

$$I = \text{Second moment of area} = \frac{t_s \times (2\,b_{sg} + t_c)^3}{12} + \frac{40\,t_c \times t_c^3}{12}$$

$$= \frac{15 \times (2 \times 110 + 13.0)^3}{12} + \frac{40 \times 13.0 \times 13.0^3}{12}$$

$= 15.91 \times 10^6 mm^4$

A = *area of effective section* $= A_w + A_{sg}$

$= 6760 + 3300 = 10060mm^2$

$$\therefore\ r_y = \sqrt{\frac{15.91 \times 10^6}{10060}} = 40mm$$

$$\lambda = \frac{0.7 \times 226}{40} = 4$$

Hence, from table 27(c) with p_{yc} *of* $265N/mm^2$ $(T_c > 16mm)$

$p_c = 265N/mm^2$

$P_{c\,buckling} = 10060 \times 265 \times 10^{-3} = 2666kN > 974kN$ O.K

Stiffener/column web crushing

$P_{c\,crushing} = [A_{sn} \times p_y] + [(b_1 + n_2) \times t_c \times p_y]$

where: A_{sn} = *net area of stiffeners in contact with column flange*

$= 2 \times b_{sn} \times t_s = 2 \times 95 \times 15 = 2850mm^2$

p_y = *lower design strength of column or stiffener*

$= 265N/mm^2$

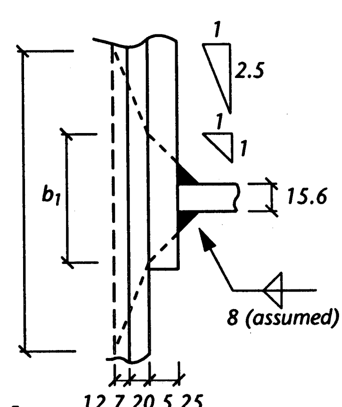
b_1 = *stiff bearing*

$= 15.6 + (2 \times 8) + (2 \times 25) = 81.6mm$

$n_2 = [\,2.5 \times (20.5 + 12.7)\,] \times 2 = 166mm$

$\therefore\ P_{c\,crushing} = [2850 \times 265 \times 10^{-3}] + [(81.6 + 166) \times 13.0 \times 265 \times 10^{-3}]$

$= 1608kN > 974kN$ O.K



Mar. 97 Revision: A_w calculation modified

Title *Worked example for a bolted end plate using the rigorous method*	Sheet *16 of 28*

Weld design

Welds to Flanges:

It is usual for the stiffeners to be fitted for bearing. ***Therefore use: 6mm fillet welds.***
(If not fitted - Use full strength welds)

Welds to web

Design welds for 974kN

Effective weld length, $L_w = 4 \times L_{sn}$

L_{sn} = *net stiffener length*
(i.e. Distance between column flanges minus snipes)

$= D_c - 2T_c -$ *corner snipes*

$= 266.7 - (2 \times 20.5) - (2 \times 15) = 196\text{mm}$

$L_w = 4 \times 196 = 784\text{mm}$

Force per mm in weld, $F_w = \dfrac{974}{784} = 1.24\text{kN/}$ *mm*

Use 10mm fillet welds
(1.5kN/mm)

weld capacities on page 224

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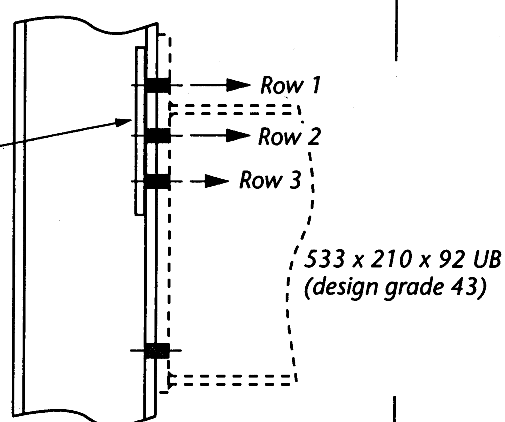
DESIGN OF COLUMN FLANGE BACKING PLATES

STEP 6B

The addition of a column flange backing plate to the column enhances the column flange bending resistance, however they only assist when Mode 1 - Complete flange yielding - is critical. This type of stiffening will only provide effective reinforcement to columns with relatively thin flanges.

Column flange backing plates

Calculate the moment capacity of a 254 x 254 x 73 UC column in the connection shown when column flange backing plates are used. All details not shown are as sheet 1.



Use plates as follows:

Width b_{bp} = *120mm*

Thickness t_{bp} = *15mm*

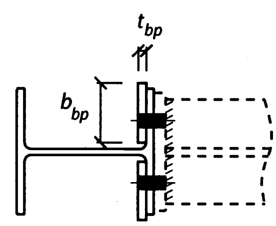
Length — *to be not less than two bolt diameters each end of the tension bolts, and not less than the effective length of the equivalent Tee-Stub for the bolt group.*

thus L_{eff} *to be:* $\geq p_{1-2} + p_{2-3} + (4 \times d)$

$\geq 100 + 90 + (4 \times 24)$

≥ 286mm,

and ≥ 428mm (sheet 20)



Using the same procedure as before (sheet 1)

Column side:

$$m = \frac{g}{2} - \frac{t_c}{2} - 0.8r_c = 50 - \frac{8.6}{2} - 0.8 \times 12.7 = 35.5\text{mm}$$

$$e = \frac{B_c}{2} - \frac{g}{2} = \frac{(254 - 100)}{2} = 77\text{ mm}$$

n = *smallest of: e(column flange), e(end plate) or 1.25m(end plate)*

= *77, 75, 1.25 x 35.5* = *44.4mm*

Other connection details are as shown on design sheet 1

Title	Worked example for a bolted end plate using the rigorous method	Sheet	18 of 28

CALCULATE BOLT ROW RESISTANCES

M_p per mm = $t^2 p_y/4$

For the end plate, t_p = 25mm, p_y = 265N/mm^2 $\therefore$ M_p/mm = 41.41kNmm/mm

For the column flange, T_c = 14.2mm, p_y = 275N/mm^2 $\therefore$ M_p/mm = 13.86kNmm/mm

For the backing Plates t_{bp} = 15mm, p_y = 275N/mm^2 $\therefore$ M_p/mm = 15.47kNmm/mm

The effective lengths of the equivalent T-stubs and potential resistances for the beam side are as calculated previously on design sheets 2 to 5. Effective lengths of the equivalent T-stubs for the lighter column must be calculated and resistances checked.

BOLT ROW 1

Column flange bending with backing plate

As sheet 2, L_{eff} is the minimum of:

$2\pi m = 2 \times \pi \times 35.5 = 223\text{mm}^*$

or $4m + 1.25e = (4 \times 35.5) + (1.25 \times 77) = 238\text{mm}$

M_p for the column flange: = 223×13.86 = 3091kNmm

M_{bp} for the backing plate: = 223×15.47 = 3450kNmm

Find the critical failure mode:

Mode 1:

$$P_r = \frac{4M_p + 2M_{bp}}{m}$$

$$P_r = \frac{(4 \times 3091) + (2 \times 3450)}{35.5} = 543\text{kN}$$

Mode 2:

$$P_r = \frac{2M_p + n(\Sigma P_t')}{m+n}$$

$$= \frac{(2 \times 3091) + (44.4 \times 2 \times 198)}{35.5 + 44.4} = 297\text{kN}^*$$

297	1

Value inserted in worksheet on page128

Mode 3:

$$P_r = \Sigma P_t' = 2 \times 198 = 396\text{kN}$$

Column web tension (see sheet 2) P_t = $173 \times 8.6 \times 275 \times 10^{-3}$ = 409kN | 409 | 2 |

End plate bending (sheet 3) P_r = 371kN | 371 | 3 |

Beam web tension (sheet 3) N/A | N/A | 4 |

Triangular limit (sheet 3) N/A | N/A | 5 |

Therefore the Potential Resistance of row 1, P_{r1} = **297kN** | 297 | 6 |

Title	Sheet
Worked example for a bolted end plate using the rigorous method	19 of 28

BOLT ROW 2:

Row 2 alone:

Column flange with backing plate, as above.	P_r	=	297kN	297 | 7
Column web tension (as above)	P_t	=	409kN	409 | 8
End plate bending (sheet 4)	P_r	=	396kN	396 | 9
Beam web tension (sheet 4)			N/A	N/A | 10

Rows 1 + 2 combined.

Column flange bending: (see sheet 4)

For the two rows acting in combination, L_{eff} is obtained from tables 2.6 and 2.4.

$$L_{eff} = 2 \times (2m + 0.625e + p/2)$$
$$= 2 \times [(2 \times 35.5) + (0.625 \times 77) + (100/2)]$$
$$= 338\text{mm}$$

M_p for the column flange = 338×13.86 = 4685kNmm

M_{bp} for the backing plate = 338×15.47 = 5229kNmm

Critical failure mode:

Mode 1:

$$P_r = \frac{(4 \times 4685) + (2 \times 5229)}{35.5} = 822\text{kN}$$

Mode 2:

$$P_r = \frac{(2 \times 4685) + (44.4 \times 4 \times 198)}{35.5 + 44.4} = 557\text{kN}^*$$

Mode 3:

$$P_r = \Sigma P_t' = 4 \times 198 = 792\text{kN}$$ 557 | 11

For row 2, P_r = 557 – 297 = 260kN 260 | 15

Column web tension (see sheet 4)

$P_{t(1+2)}$ = $273 \times 8.6 \times 275 \times 10^{-3}$ = 646kN 646 | 12

For row 2, P_t = 646 – 297 = 349kN 349 | 16

End plate bending

(see sheet 4) not applicable N/A | 13 N/A | 17

Beam web tension

(see sheet 5) not applicable N/A | 14 N/A | 18

Triangular limit

Since the end plate and the combined flange / backing plate both exceed 21.9mm, the triangular limit will apply. N/A | 19

No modification is required to row 2 in an extended end plate, as row 2 sets the limit. (see figure 2.18)

The potential resistance for row 2, P_{r2} is the least of the values from the boxes 7 to 10 and 15 to 19.

i.e. 297kN, 409kN, 396kN, 260kN, 349kN

Therefore the Potential Resistance of Row 2 P_{r2} = **260kN** 260 | 20

Title	*Worked example for a bolted end plate using the rigorous method*	Sheet	*20 of 28*

BOLT ROW 3:

Row 3 alone

Column flange with backing plate, as above. P_r = 297kN | 297 | 21 |

Column web tension (as above) P_t = 409kN | 409 | 22 |

End plate bending (sheet 5) P_r = 396kN | 396 | 23 |

Beam web tension (sheet 5) P_r = 485kN | 485 | 24 |

Rows 2 + 3 combined.

Column flange bending (see sheet 5)

L_{eff} *obtained from tables 2.6 and 2.4* = $2 \times (2m + 0.625e + p/2)$

$= 2 \times [(2 \times 35.5) + (0.625 \times 77) + (90/2)]$

$= 328mm$

M_p *for the column flange* = 328×13.86 = 4546kNmm

M_{bp} *for the backing plate* = 328×15.47 = 5074kNmm

Critical failure mode

Mode 1:

$$P_r = \frac{(4 \times 4546) + (2 \times 5074)}{35.5} = 798kN$$

Mode 2:

$$P_r = \frac{(2 \times 4546) + (44.4 \times 4 \times 198)}{35.5 + 44.4} = 554kN^*$$

| 554 | 25 |

Mode 3:

$$P_r = \Sigma P_t' = 4 \times 198 = 792kN$$

For row 3, $P_r = 554 - 260 = 294kN$ | 294 | 29 |

Column web tension (see sheet 6)

$P_{t(2+3)} = 263 \times 8.6 \times 275 \times 10^{-3} = 622kN$ | 622 | 26 |

For row 3, $P_t = 622 - 260 = 362kN$ | 362 | 30 |

End plate bending (sheet 6)

For row 3, $P_r = 763 - 260 = 503kN$ | 763 | 27 |

| 503 | 31 |

Beam web tension

(see sheet 7) *not applicable* | N/A | 28 |

Rows 1 + 2 + 3 combined. | N/A | 32 |

L_{eff} *is obtained from tables 2.6 and 2.4*

(see sheet 7) $L_{eff} = 4m + 1.25e + p_{1-2} + p_{2-3}$

$= (4 \times 35.5) + (1.25 \times 77) + 100 + 90$

$= 428mm$

M_p *for the column flange* = 428×13.86 = 5932kNmm

M_{bp} *for the backing plate* = 428×15.47 = 6621kNmm

Title	Sheet
Worked example for a bolted end plate using the rigorous method	*21 of 28*

Critical failure mode

Mode 1:

$$P_r = \frac{(4 \times 5932) + (2 \times 6621)}{35.5} = 1041\text{kN}$$

Mode 2:

$$P_r = \frac{(2 \times 5932) + (44.4 \times 6 \times 198)}{35.5 + 44.4} = 809\text{kN}^{*}$$

809 | 33

Mode 3:

$$P_r = 6 \times 198 = 1188\text{kN}$$

P_r for row 3 is taken as minimum for modes 1 to 3 minus P_{r1} minus P_{r2}

For row 3, P_r = 809 – 297 – 260 = 252kN

252 | 37

Column web tension (see sheet 7)

$$P_{t(1+2+3)} = 363 \times 8.6 \times 275 \times 10^{-3} = 859\text{kN}$$

For row 3, P_t = 859 – 297 – 260 = 302kN

859 | 34

302 | 38

End plate bending

(see sheet 7) *not applicable*

N/A | 35

N/A | 39

Beam web tension

(see sheet 8) *not applicable*

N/A | 36

N/A | 40

Triangular limit

Triangular limit applies, with the limiting value of:

$$\frac{260 \times 375}{465} = 210\text{kN}$$

210 | 41

The potential resistance for row 3, P_{r3} is the smallest of the values from boxes 21 to 24, 29 to 32 and 37 to 41. i.e. 297kN, 409kN, 396kN, 485kN; 294kN, 362kN, 503kN; 252kN, 302kN, 210kN.

Therefore the potential resistance for row 3, P_{r3} = 210kN

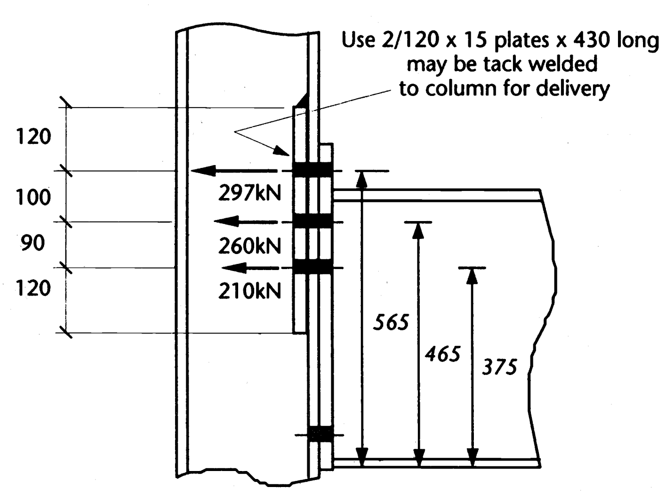
210 | 42

Step 1 has produced the distribution of potential bolt forces shown below.

Assuming that neither the web panel shear capacity or the resistance in the compression zone limit the development of these forces (strengthening may be required), then

Moment capacity = $297 \times 0.565 + 260 \times 0.465 + 210 \times 0.375$

= *367kNm*



Title	Worked example for a bolted end plate using the rigorous method	Sheet	22 of 28

Worksheet for Step 1 : Tension Zone

Row	Column Side STEP 1A Flange Bending	Column Side STEP 1 B Web Tension	Beam Side STEP 1A Plate Bending	Beam Side STEP 1B Web Tension	STEP 1C Triangular Limit	Potential Resistance
1	*Resistance of Row 1*					least of boxes 1 to 4 gives
	297 [1]	409 [2]	371 [3]	N/A [4]	N/A [5]	P_{r1} = 297 [6]
2	*Resistance of row 2 alone:*					
	297 [7]	409 [8]	396 [9]	N/A [10]		
	Resistance of rows (1 + 2) combined:					least of boxes: 7 to 10 and 15 to 18 gives
	557 [11]	646 [12]	N/A [13]	N/A [14]		
	Deduct box 6:					
	260 [15]	349 [16]	N/A [17]	N/A [18]	N/A [19]	P_{r2} = 260 [20]
3	*Resistance of row 3 alone:*					
	297 [21]	409 [22]	396 [23]	485 [24]		
	Resistance of combined rows (2+3):					
	554 [25]	622 [26]	763 [27]	N/A [28]		
	Deduct box 20:					
	294 [29]	362 [30]	503 [31]	N/A [32]		least of boxes 21 to 24 and 29 to 32 and 37 to 41 gives
	Resistance of combined rows (1+2+3):					
	809 [33]	859 [34]	N/A [35]	N/A [36]		
	Deduct sum of boxes 6 & 20:					
	252 [37]	302 [38]	N/A [39]	N/A [40]	210 [41]	P_{r3} = 210 [42]
4	*Resistance of row 4 alone:*					
	[43]	[44]	[45]	[46]		
	Resistance of combined rows (3+4):					
	[47]	[48]	[49]	[50]		
	Deduct box 42:					
	[51]	[52]	[53]	[54]		
	Resistance of combined rows (2+3+4):					
	[55]	[56]	[57]	[58]		
	Deduct sum of boxes 20 & 42:					least of boxes 43 to 46 and 51 to 54 and 59 to 62 and 67 to 71 gives
	[59]	[60]	[61]	[62]		
	Resistance of combined rows (1+2+3+4):					
	[63]	[64]	[65]	[66]		
	Deduct sum of boxes 6, 20 & 42:					
	[67]	[68]	[69]	[70]	[71]	P_{r4} = [72]

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	Calcs by: DGB · Checked by: AM	Date: May 94

DESIGN OF TENSION RIB STIFFENERS

STEP 6C

The addition of a tension rib stiffener to a beam or column section may be:

(1) *to enhance the column flange bending resistance (STEP 1A) by increasing the effective length of the equivalent T–stubs for the bolt rows adjacent to the stiffener.*

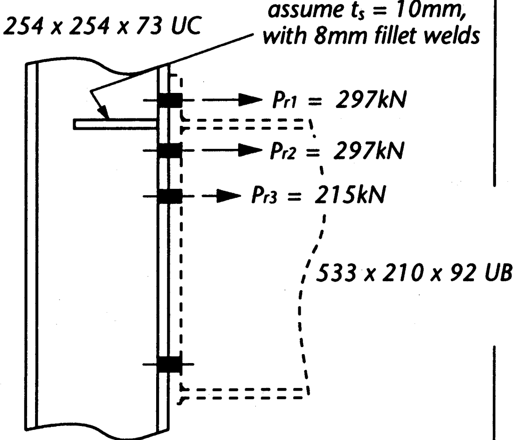
(2) *to enhance the column web tension resistance (STEP 1B) by effectively preventing a web tension failure.*

Stiffener Design

Design the tension rib stiffener shown. The potential bolt row resistances have been calculated using the procedures of section 2.8, including the effect of the stiffener. For the purposes of this example, it is assumed that the full bolt row potential forces can be developed

i.e. $F_{ri} = p_{ri}$

All details not shown are as sheet 1.



The recommended sizing parameters are:

$$b_{sg} \geq \frac{0.75\,(B_c - t_c)}{2}, \text{ and}$$

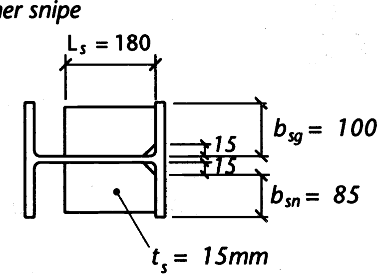
$$L_s \geq 1.8 b_{sg}$$

hence:

$$b_{sg} \geq \frac{0.75\,(254.0 - 8.6)}{2} = 92\text{mm} \quad \text{say} \quad 100\text{mm}$$

$$L_s \geq 1.8 \times 100 = 180\text{mm} \quad \text{say} \quad 180\text{mm}$$

the stiffeners will have a 15 x 15 corner snipe



Title *Worked example for a bolted end plate using the rigorous method*	Sheet *24 of 28*

Stiffener Net Area

The net area of the stiffener, A_{sn} must not be less than the values from formulae (2.17) and (2.18).

Web Tension

basic requirement is: $$A_{sn} \geq \frac{(F_{r1} + F_{r2})}{p_y} - (L_t \times t_c) \quad .... (2.17)$$

where:

F_{r1} = 297kN

F_{r2} = 297kN

p_y = 275N/mm^2 *(the lesser design strength of the stiffener and column)*

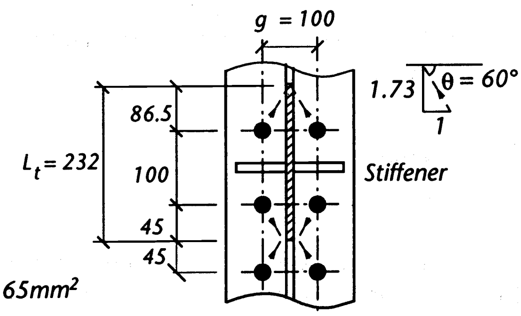
L_t = *available length of web*

$= (1.73 \times \frac{g}{2}) + p_{1\text{-}2} + \frac{p_{2\text{-}3}}{2}$

$= (1.73 \times \frac{100}{2}) + 100 + 45$

$= 86.5 + 100 + 45$

$= 232mm$



Hence, $$A_{sn} \geq \frac{[(297 + 297) \times 10^3]}{275} - (232 \times 8.6) = 165mm^2$$

Flange Bending

basic requirement is:

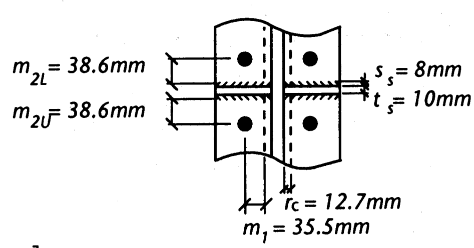
$$A_{sn} \geq \frac{m_1}{p_y}\left[\frac{F_{r1}}{(m_1 + m_{2U})} + \frac{F_{r2}}{(m_1 + m_{2L})}\right] \quad .... (2.18)$$

$$m_1 = \frac{g}{2} - \frac{t_c}{2} - 0.8r_c = \frac{100}{2} - \frac{8.6}{2} - 0.8 \times 12.7 = 35.5mm$$

with the stiffeners placed centrally between bolts:

$$m_{2U} = m_{2L} = \frac{p_{1\text{-}2}}{2} - \frac{t_s}{2} - 0.8s_s$$

$$= \frac{100}{2} - \frac{10}{2} - 0.8 \times 8 = 38.6mm$$



Hence, $$A_{sn} \geq \frac{35.5}{275}\left[\frac{297 \times 10^3}{(35.5 + 38.6)} + \frac{297 \times 10^3}{(35.5 + 38.6)}\right] = 1035mm^2$$

Therefore the net area of both stiffeners must be at least 1035mm^2

and, $A_{sn} = 2\,(b_{sn} \times t_s)$

giving, $$t_s \geq \frac{1035}{2 \times b_{sn}} \geq \frac{1035}{2 \times 85} = 6.1mm$$

Therefore use 2/100 x 10 stiffeners x 180mm long.

Weld Design

Use full strength fillet welds, i.e. $$\frac{t_s}{2 \times 0.7} = \frac{10}{2 \times 0.7} = 7.1mm.$$

Adopt 8mm fillet welds.

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	Calcs by: RS · Checked by: AM	Date: May 95

STEP 6D

DESIGN OF COLUMN SUPPLEMENTARY WEB PLATES

Supplementary web plates provide:

an increase in web tension resistance of :
- *50% for a plate on one side of the web*
- *100% for a plate on both side of the web*

an increase in web crushing and buckling resistance of :
- *50% for a plate on one side of the web*
- *100% for a plate on both side of the web*

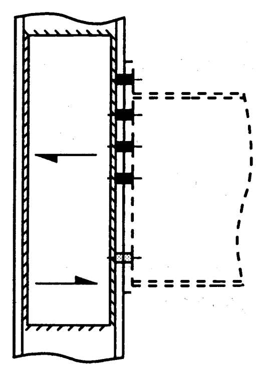
an increase in web panel shear resistance :
- *approximately 75% (no increase for two plates)*

Worked Example

The example considered on Calculation Sheet 1 is to be used, assuming a beam connection to the column on one side only. The column web panel will be strengthened, to avoid the web panel shear capacity limiting the moment resistance of the connection.

254 x 254 x 107 UC
Design grade 43

Panel shear capacity required = 853 kN



Try one supplementary web plate as follows.

Design grade = 43 (as column)

Breadth b_s = depth between fillets (d) say 200mm

Thickness t_s = not less than column web thickness (13.0 mm) say 15mm

Length required, $L_s \geq g + L_c + \dfrac{D_c}{2}$

where: g = gauge of bolts = 100mm

L_c = length of end plate = 660mm

D_c = depth of column = 266.7mm

Length $L_s \geq 100 + 660 + \dfrac{266.7}{2}$

= 893mm say L_s = 900mm

Title	Worked example for a bolted end plate using the rigorous method	Sheet 26 of 28

Column panel shear

$P_v = 0.6 \times p_{yc} \times A_v$

where: p_{yc} = design strength of the column = $265 N/mm^2$

A_v = shear area of the column web and SWP combined

$= t_c \times (D_c + b_s) = 13.0 \times (266.7 + 200) = 6067\ mm^2$

$P_v = 0.6 \times 265 \times 6067 \times 10^{-3} = 965\ kN > 853$ OK

Welds

Horizontal Welds

Fillet welds of leg length equal to the plate thickness t_s = 15mm fillet weld

Vertical Welds

Fillet welds of leg length equal to the plate thickness t_s = 15mm fillet weld.

(if the supplementary web plate was provided to increase web tension resistance, 'fill in' welds (Figure 2.34) would be required.)

Plug welds

Required if b_s exceeds $37t_s$ (design grade 43) or $33t_s$ (design grade 50).

$37t_s = 37 \times 15 = 555 > 200$

Therefore plug welds not required.

Adopt 15mm fillet welds all round

Note: the provision of a supplementary web plate will also improve the web tension, crushing and buckling resistance of the column.

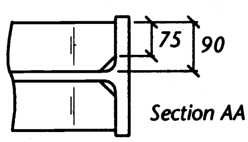
CALCULATION SHEET BCSA	Job: Moment Connections	Sheet: 27 of 28
	Title: Worked example for a bolted end plate using the rigorous method	
	Client: SCI/BCSA Connections Group	
	Calcs by: RS / Checked by: AM	Date: May 95

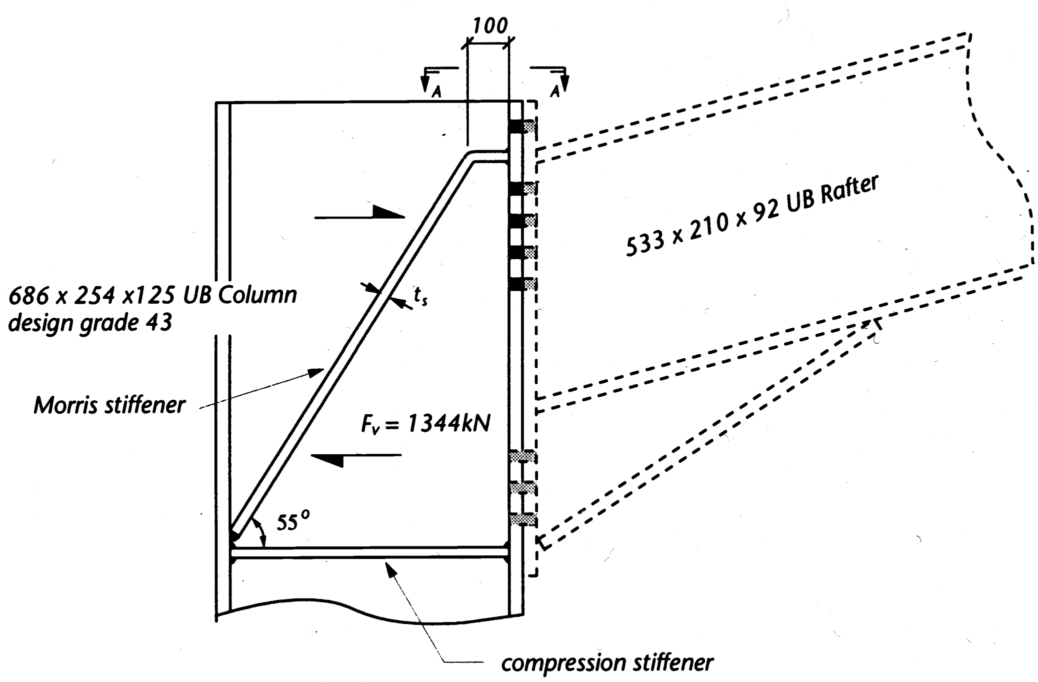
DESIGN OF MORRIS STIFFENER

STEP 6E

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 frequently found to be necessary in addition to compression stiffeners.

The portal connection shown will be considered, with the web panel shear force assumed to be 1344 kN.





Shear capacity of a 686 x 254 x 125 UB is 1261 kN < 1344 kN

Therefore provide Morris stiffeners.

Title	Worked example for a bolted end plate using the rigorous method	Sheet	28 of 28

Web Panel Shear

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

$$A_{sg} \geq \frac{(F_v - P_v)}{p_y \cos\theta} \quad \ldots\ldots (2.20)$$

where:

A_{sg} = $2 \times b_{sg} \times t_s$

b_{sg} = net width of stiffener (see figure 2.35)

t_s = thickness of stiffener

F_v = the applied shear force (see STEP 3)

P_v = the resistance of the unstiffened column web panel (see STEP 3)

p_y = lower design strength of stiffener or column

θ = angle of stiffener from horizontal

$$\frac{F_v - P_v}{p_y \cos\theta} = \frac{(1344 - 1261) \times 10^3}{265 \times \cos 55} = 546 \text{ mm}^2$$

then:

$$t_s \geq \frac{546}{2 \times b_{sg}} = \frac{546}{2 \times 90} = 3.0\text{mm}$$

Adopt 10 thick x 90 Morris stiffeners.

Note: the design of compression stiffeners is illustrated on calculation sheets 14 to 16.

Welds

To column flange:

Provide full strength fillet welds, i.e. $\frac{t_s}{2 \times 0.7} = \frac{10}{2 \times 0.7} = 7.1\text{mm.}$

To column web:

Provide nominal fillet welds

Adopt 8mm fillet welds to flanges and web

APPENDIX II BOLTED END PLATE CONNECTIONS

Background to the design method

Essentially the design method put forward in Section 2 is that of EC3 Annex J, which treats the beam-to-column connection in substantially more detail than previous codes. However, the method has been adapted for use with BS 5950 in recognition of the fact that it will be some years before designs are routinely based on EC3.

A hierarchy of authority

With the notable exception of its bolt tension values (see appendix IV), BS 5950 has been regarded as the "highest authority" for the purpose of detail design checks. Where BS 5950 does not lay down a design rule, EC3 is followed. New rules have been formulated where the aspect in question is not adequately covered by either of these standards.

This policy has been followed by the task group despite the knowledge that the published EC3 Annex J is deficient in a number of respects and a thoroughgoing revision is in train. Some inviting improvements to the design procedure have, for the time being, been foregone simply because it was judged desirable to respect "official" source material as closely as possible.

It is envisaged that once the revised Annex J is published this design guide will itself be revised; whether this revision retains allegiance to BS 5950 or represents a total conversion to EC3 will have to be decided at the time.

The choice of symbolism

In order to retain familiarity and compatibility (so far as possible) with BS 5950, the task group decided **not** to adopt the ISO symbols used in Eurocodes in this design guide. However, there are several situations in which new symbols or subscripts are required, and the preference has been to follow the EC3 pattern for these.

Inevitably, the result is a hybrid system which may serve as a stopgap but should be replaced by EC3 symbols as soon as the industry is ready to accept the transition.

Exceptions to published standards

Exceptions to the policy of giving preference to published standards have only been made where it would have been absurd or dangerous to conform. However, if technical arguments alone had prevailed the task group might have been inclined to depart from the published standards more freely.

In the interest of those returning to the subject in the future, notes (1) to (15) on the following pages set out examples of the more questionable provisions which the present design method either incorporates (reluctantly) or departs from (giving reasons).

(1) EC3 - Rotational stiffness formula

The formula in the current Annex J of EC3 for the rotational stiffness of an end plate connection does not give accurate predictions, and designers are warned against reliance on it in critical cases. In practice this is mainly of consequence to semi-rigid elastic analysis, connections for which are outside the scope of this design guide.

(2) EC3 - The "Rigid" criterion

Connections are required to be "Rigid" if their frame is to be analysed in the conventional elastic way. EC3 specifies a rotational stiffness which must be achieved for a connection to qualify. Figure 6.9.8 of EC3 summarises the requirement, which is relative to the rotational stiffness of the beam and varies strikingly between braced and unbraced frames. In view of the difficulty of predicting rotational stiffness (see above) this numerical approach is somewhat academic, but it is worth remarking that EC3's numerical requirement is substantially more demanding than that of BS 5950.

(3) EC3 - Influence of high compressive stress in the column

EC3 prescribes reduction factors in two areas: the flange in the tension zone (clause J.3.4.1(3)) and the web (crushing only) in the compression zone (clause J.3.5.1(1)). In practical cases these provisions are marginal in their effect and at the same time tedious to apply. The task group has decided to omit them.

(4) EC3 - Yield patterns

There are several unsatisfactory aspects to the way the effective lengths of equivalent T-stubs are calculated in the current Annex J of EC3. Clearly, the circular yield pattern involves no prying and bears no comparison with the T-stub model from which the formulae for Modes 1 and 2 are derived. (Expressions (2.1) (2.2) and (2.3) on page 18 of this design guide.) In fact, an effective length of $2\pi m$ gives the correct answer with the Mode 1 formula but is meaningless with the Mode 2 formula. (It would have been simpler and less confusing to regard it as a separate Mode O.)

The second major difficulty is that the α-chart used for bolt rows adjacent to stiffeners or flanges is artificially limited to $\alpha = 2\pi$ because of the circular yield pattern. When another bolt row is present, a combined yield pattern will almost invariably govern. However no physically coherent yield pattern can be deduced from the given expressions. As a result the strength calculation will often be unnecessarily conservative.

There are also omissions of yield patterns which could govern in certain circumstances, which is of course unconservative. These have been remedied in the present design method, but otherwise effective lengths are calculated according to the published EC3. It is with some reluctance that the promising new approach of the current draft revision of Annex J, with its extended α-chart, has not been adopted in this manual.

(5) EC3 - Web tension

EC3 prescribes that the effective area for web tension checks should be based on the same "effective length of equivalent T-stub" as used for the bolt tension calculation. Since edge distance is an important influence on T-stub length, but can only be a minor one on the way the web responds to tension, this approach seems questionable. In the present design method it is replaced by a simple geometrical rule for the unstiffened web, and a somewhat intuitive apportionment where stiffeners are present.

There is also a procedural improvement; web tension (both on the column side and the beam side) is calculated row by row, alongside the flange and end plate bending (see worksheet on page 26) rather than separately later (as in step (6) of EC3's procedure J.3.1).

(6) EC3 - Limit to plastic distribution of bolt row forces - Procedure J.3.1

"Step (4)" in EC3 Procedure J.3.1 represents an impasse for many, if not most, practical connection designs. It has the effect of limiting end plate thickness in relation to bolt size and strength, to ensure that mode 3 is avoided, except where the connection is designed to be full strength. It was introduced to avoid the situation in which outer bolts fail before inner rows have developed their full contribution to the plastic bolt force distribution. Instead, the present design method, included under Step 1C, imposes the triangular limit to bolt force distribution.

(7) EC3 - Limit to plastic distribution of bolt row forces - Procedure J.3.2

This "alternative" to the plastic bolt force distribution is not to be confused with the triangular limit of the present design method. It has more in common with the traditional approach to multi-bolt row connections, and is open to the same objections.

(8) BS 5950 - Column web buckling

BS 5950: Part1, clause 4.5.2.1, provides, in column web buckling checks, for the effective area being based on the sum of stiff bearing length plus the column depth. It is included in STEP 2A of the procedures. EC3's expression (5.79), based on recent tests, is more conservative in prescribing the square root of the sum of the squares of these quantities.

(9) EC3 - Beam flange compression

The current EC3 Annex J omits explicit reference to beam flange compression resistance, giving the impression that no check need be made. Particularly when axial compression acts in the beam, this would be untenable. On the other hand, to limit the flange to yield stress is overcautious and prevents many full strength connections from being designed as such. The task group's rule (Step 2B, page 28) is a compromise.

(10) EC3 - Two sided connections

The current EC3 Annex J fails to confront the issue of the competing demands that two opposing beams make of the column web panel. It is an inescapable fact that the strength available to one side, and even the stiffness perceived by that side, will be influenced by the magnitude and direction of the moment applied on the other. It is anticipated that this problem will be addressed in the revised Annex J. Meanwhile, this design guide offers no magic solution. The problem is a real one for designers of unbraced frames, who must guard against counting the web panel twice and must avoid alternating plasticity in that zone.

(11) BS 5950 - Interaction of bolt tension and shear

STEP 5 of the procedures, dealing with bolts in both shear and tension, adopts BS 5950: Part1 clause 6.3.6.3. It allows bolts subject to full design load in tension to retain 40% of their regular shear capacity. EC3's corresponding interaction formula, in expression (6.6), only permits such bolts to contribute $28\frac{1}{2}$% of design shear resistance. Both formulae represent straight-line simplifications of a more-or-less elliptical interaction plot based on tests. BS 5950's relative under-conservatism needs to be viewed alongside its generally more conservative bolt values. Any future revision of the present design method which increases design tension in bolts (see appendix IV) should prompt a re-examination of this question.

(12) BS 5950 - Compression stiffeners the "80%" rule

Compression stiffeners are designed to resist 80% of the force applied by the beam flange, following BS 5950: Part1, clause 4.5.4.2. While it must be expected that such a stiffener will tend to attract the greater part of the flange force (in preference to the column web) the "80%" rule appears unduly onerous. Nevertheless it has been incorporated into STEP 6A of the connection design procedures.

(13) EC3 - Stiffener design

EC3 is short of detail on the mechanics of stiffeners, whether for web reinforcement or to increase flange bending resistance. Clause J.2.3.3 suggests that it was envisaged that stiffeners would generally be sized to match the beam flanges, but this is often impractical and usually unnecessary.

The stiffener design rules presented under Step 6 (p32 et seq) are by and large of the task group's own devising. They respect statics and, where it makes a prescription, BS 5950. Rib (i.e. discontinuous) and Morris stiffeners, neither of which are covered in EC3, are included.

(14) EC3 - Supplementary web plates

Design rules for supplementary web plates in EC3 have been adopted in this design guide but the task group was unwilling to accept the degree of strain hardening that is implied in the "neck" between the flange and a pair of fillet-welded SWPs. Conversely, some of the other rules seem quite cautious, for example where a second plate adds no further resistance to shear.

To be safe, it is recommended under step 6D that the welds down the sides of the SWP should be of the "fill-in" type (EC3 calls them "butt" welds) if the purpose of the SWP is to improve web tension resistance.

(15) EC3 - Flange to end plate weld

EC3 clause J.3.4.4(6) requires welds between flanges and end plates to be overdesigned by up to 70%. This is because weld failure is brittle and must be avoided, even when (as commonly occurs) the other components of the connection overperform by such a margin.

The task group has taken the view that in general the recommendations given under Step 7 (page 39) will suffice. It should be noted that full strength welds are prescribed for the ductile wind-moment connections of table 3.2.

APPENDIX III MATHEMATICAL DERIVATION OF ALPHA CHART

This section gives the information required for the mathematical derivation of the value of α which is used in the calculation of the effective lengths of equivalent T-stubs when the bolt row being considered is adjacent to a flange or stiffener. The formulae have been determined from a curve fitting exercise, and are an approximation for the curves shown on page 23.

The value of α depends on the magnitude of λ_1 and λ_2 which are calculated from the connection geometry as shown on pages 22 and 23. These values of λ_1 and λ_2 are substituted into the six formulae, F1 to F6 given below.

α is found by satisfying one of the conditional statements opposite.

Conditional statements

1. If $\lambda_1 \leq$ F1 then, $\alpha = 2\pi$
2. If $\lambda_1 \geq$ F2 then, $\alpha = 4.45$
3. If F1 $< \lambda_1 <$ F2 then,

   (a) If $\lambda_2 \geq 0.45$
   then, α = F3 but $\leq 2\pi$

   (b) If $(0.2768\,\lambda_1 + 0.14) \leq \lambda_2 < 0.45$
   then, α = F4 but $\leq 2\pi$

   (c) If $(1.2971\,\lambda_1 - 0.7782) \leq \lambda_2 < (0.2768\,\lambda_1 + 0.14)$
   then, α = F5 but $\leq 2\pi$

   (d) If $\lambda_2 < (1.2971\,\lambda_1 - 0.7782)$
   then, α = F6 but $\leq 2\pi$

Formulae

$$F1 = 0.99477448 - 2.45848503\,\lambda_2 + 3.15497168\,\lambda_2^2 - 2.23017434\,\lambda_2^3 + 0.52850212\,\lambda_2^4$$

$$F2 = 1.04213142 - 0.85759182\,\lambda_2 + 1.15828063\,\lambda_2^2 - 0.79910192\,\lambda_2^3 + 0.21398139\,\lambda_2^4$$

Values of the coefficients for formulae F3 to F6 are given in tabular form below:

	F3	F4	F5	F6
constant	8.130283	1.245666	- 86.505200	- 226.979097
λ_1	4.488295	39.333003	478.588870	1095.760732
λ_2	- 3.441231	- 3.580332	79.430092	- 12.1186777
λ_1^2	-16.699661	- 55.940605	- 935.102794	- 1848.467314
λ_2^2	4.657641	40.544586	- 329.854733	717.104423
$\lambda_1 \lambda_2$	- 6.802532	- 55.343570	- 68.228567	- 264.307024
λ_1^3	8.747474	21.049463	809.056164	1369.007748
λ_2^3	- 1.197675	-33.001768	531.672952	- 2120.516058
$\lambda_1 \lambda_2^2$	- 1.227359	2.792410	252.193252	- 69.105002
$\lambda_1^2 \lambda_2$	8.318217	44.062493	- 44.242644	195.697905
λ_1^4			--254.659837	- 381.685783
λ_2^4			- 605.622885	2562.146768

APPENDIX IV 8.8 BOLTS
ENHANCED TENSILE STRENGTH

The recommended design tension values in this publication for 8.8 bolts are based on 560N/mm^2 over the tensile stress area, which is just under 25% greater than the 450N/mm^2 ("inclusive of prying") of BS 5950: Part 1, Table 32. While the figure of 560N/mm^2 is, in the final analysis, a committee decision, it may be rationalised as follows:

- The current 8.8 bolt supplied to BS 3692 has an ultimate tensile strength (U.T.S) of 785N/mm^2 'guaranteed minimum'. U.T.S is a more appropriate basis than yield stress (which becomes unidentifiable at higher strength grades).

- It is considered that a 'material factor' of 1.25 should be applied to components such as bolts.

  This is not because individual bolts are especially variable in strength; rather it recognises the uneven distribution of force between bolts in common jointing situations and perhaps also an 'importance factor' in the sense that these small components exert a disproportionate influence on the overall safety of the structure they connect.

- A further reduction factor of 0.9 is applied to allow for the possibility that thread stripping at the nut will prevent the bolt and nut assembly from achieving the strength of the bolt.

Hence,

$$785 \times \left[\frac{0.9}{1.25}\right] = 565, \text{ say } 560\text{N/mm}^2$$

It should, in due course, be possible to recommend a higher design strength when conversion to new bolt standards (ISO 4014, BS EN 24014 etc.) is complete. This is because the guaranteed minimum U.T.S is higher (828N/mm^2 instead of 785N/mm^2) and nut geometry is modified to reduce the risk of premature thread stripping.

Bolts up to M24 specified as HSFG bolts to BS 4395 are available now with both these advantages, and could be used at higher design strengths (say 600N/mm^2), without needing to be preloaded.

There is no technical reason why the higher performance of these bolts should not be taken advantage of, but it was felt that an already confused situation could be made more so if their (temporary) superiority were to be given prominence in this publication.

CAPACITY TABLES
AND DIMENSIONS FOR DETAILING

Contents

BOLTED END PLATE CONNECTIONS

Notes on use of the Beam-to-Column Capacity Tables.

BEAM TABLES

Bolt row forces F_{r1}, F_{r2} etc

The bolt row forces shown are the maximum available on the beam side of the connection, calculated using the procedures in Section 2.8. The standard end plate thicknesses adopted in the tables exceed the limit for full plastic force distribution quoted in Table 2.7, and therefore the triangular limit has been imposed in all cases. The sum of the bolt row forces, ΣF_r, is limited to the compression flange resistance. If the compression flange resistance governs, the bolt row forces have been reduced, starting at the lowest row; the maximum values are shown in brackets below the reduced values.

ΣF_r

ΣF_r is the sum of the maximum available bolt row forces, in tension, on the beam side when no axial forces are present.

Beam P_c

Beam P_c is the capacity of the beam compression flange. This is based on the smaller of the beam width or end plate width.

Moment capacity

The moment capacity is that of the beam side of the connection, calculated from the quoted bolt row forces.

Beam M_{cx}

M_{cx} is the moment capacity of the beam section.

Bolt shear capacity

The bolt shear capacities shown are the vertical shear capacity of each bolt row in the tension zone, and of each row in the shear zone. Bolt rows in the tension zone are shown in black. Bolt rows in the shear zone are shown in grey.

Mini haunches

The bolt row forces are based on the lightest section in the serial size(s) covered.

The haunch flange and web are taken as being the same steel grade as that of the beam section

The minimum haunch depth used in the tables is one third of the section depth. Moment capacities are quoted for increasing depths of haunch, until M_{cx} of the beam is exceeded, or the maximum haunch depth is reached.

The maximum haunch depth is taken as $D_b - T_b - r_b - 10$.

It is recommended that the mini haunch is made with the flange sloping at an angle of 30° to the beam flange. The capacity of the beam web in compression should be checked in accordance with Section 2, STEP 8.

If the haunch is built up from plates, the web plate should be of similar thickness to the beam web.

Min. thickness of haunch flange

The minimum thickness shown for the haunch flange is that required for the flange to develop ΣF_r, based on the width of the flange being equal to the beam width, or the end plate width if this is narrower. It is calculated using an angle of 30° between beam and haunch flange. The haunch flange width should be at least as wide as the end plate or beam flange (whichever is narrower).

If, after matching with the column resistance, ΣF_r is reduced from the quoted value, the minimum thickness of the haunch flange may be reduced in proportion to the reduction in ΣF_r.

Welds

Forces and capacities quoted in the tables are based on welds to design grade 43 beams being made with E43 electrodes to BS 639, and welds to design grade 50 beams being made with E51 electrodes to BS 639; all in accordance with Table 36 of BS 5950 Part 1.

It should be noted that the calculation of forces and capacities take account of the weld sizes shown. Weld sizes should not be changed without re-checking the connection.

Tension flange weld

The weld size is designated as a fillet weld (e.g. 8FW), a partial penetration butt weld with a superimposed 10mm fillet (e.g. 10FW+6pp), or a full penetration butt weld (FPB). The fillet welds and partial penetration welds are required around the whole flange to the junction with the root and beam web. The size is calculated to provide a capacity which is not less than the maximum force from the top three bolt rows in an extended end plate, and the maximum force from the top two rows in a flush end plate, or equal to the full strength of the flange.

The sizes indicated for partial penetration butt welds with superimposed fillets take account of a 3mm loss of penetration at the root of the weld. The detail is shown in Figure C1. The shear and tension stresses on the fusion faces have been checked in accordance with STEP 7 of the procedures in Section 2.

In many cases, a manual check using the actual bolt row forces may obviate the requirement for the partial penetration weld, but the fillet size quoted should not be reduced without re-checking the connection.

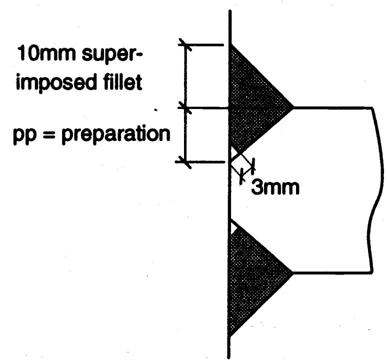


Figure C1
Partial penetration welds with superimposed fillets

Web weld

A continuous fillet weld each side of the web of the leg length shown provides a capacity not less than that of the beam web.

Compression flange weld

In the extended and flush end plate cases bearing contact between the compression flange and end plate is assumed, and nominal fillet welds are therefore prescribed. These weld sizes will also be satisfactory if bearing is provided in the haunched case. If a bearing fit is not specified, the welds between flange and end plate should be calculated manually.

The end plate should extend a minimum of the plate thickness plus weld leg length below the flange as shown in Figure C2. (The column tables are not valid if this recommendation is infringed)

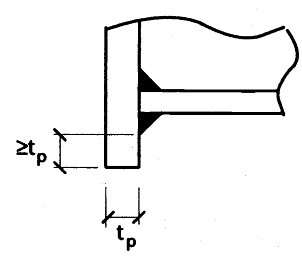


Figure C2
End plate extension below compression flange

Axial forces in the beam

The tables assume axial force in the beam is zero. If present, the axial force should be considered to act at the centre of compression and the applied moment modified accordingly (Section2, STEP 4). Worked examples on pages 148 and 149 show how the tables may be used to design the connection when axial force is present in the beam. Increased capacity with axial loads may be obtained when the rigorous method is applied to both the beam and the column.

Axial compression reduces the applied moment, and increases the force in the compression flange. This may require the bolt row forces to be reduced, starting with the lowest row if the beam compression flange capacity was (or becomes) limiting.

Axial tension increases the applied moment, and reduces the force in the compression flange. If, with no axial load, the compression flange is limiting, then the reduction in force will allow an increase in bolt row forces, up to the maximum values available (shown in brackets in the tables).

The minimum flange thickness to the mini haunch is based on ΣF_r as explained on page 142. However, in the presence of axial compression, the minimum thickness should be increased in proportion to the actual flange force, compared to ΣF_r quoted.

COLUMN TABLES

Bolt row forces F_{r1}, F_{r2} etc

The bolt row forces are the maximum acceptable on the column side of the connection calculated using the procedures in Section 2.8 for up to six rows of bolts.

The bolt row forces are calculated assuming 90mm vertical pitch. They are slightly conservative in an extended end plate detail where the second row is 100mm below the first row.

Compression capacity P_c

P_c is the maximum acceptable compression force on the column flange. Crushing resistance and buckling resistance are calculated using the stiff bearing lengths shown in Figure C3, and the lesser resistance is quoted.

Where the compressive force exceeds the resistance quoted, a pair of compression stiffeners may be provided, designed in accordance with Section 2, STEP 6A.

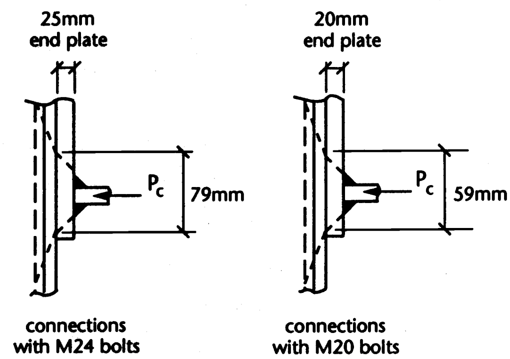


Figure C3
Stiff bearing lengths used in capacity calculations

Web panel shear capacity, P_v

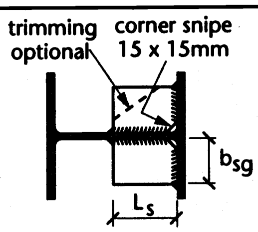
P_v is the shear capacity of the column web and supplementary web plate when shown. Web panel shear must take account of beams connecting onto both column flanges and the direction of their respective moments. See Section 2, STEP 3.

Tension stiffeners

A pair of rib stiffeners can be used to increase the resistance in the tension zone where either web tension or flange bending is limiting. The quoted bolt row forces in the Rib Stiffened column tables take this into account.

Stiffeners may be sized in accordance with Table T1, or designed in accordance with STEP 6C

Table T1 Dimensions for tension stiffeners



Stiffeners	Weld
10mm	8FW
12mm	10FW

Serial Size	M24			M20		
	Width b_{sg}	Length L_s	Thickness t_s	Width b_{sg}	Length L_s	Thickness t_s
356 x 368	100	180	10	75	150	10
305 x 305	100	180	10	75	150	10
254 x 254	100	180	10	75	150	10
203 x 203	80	150	12	75	150	10

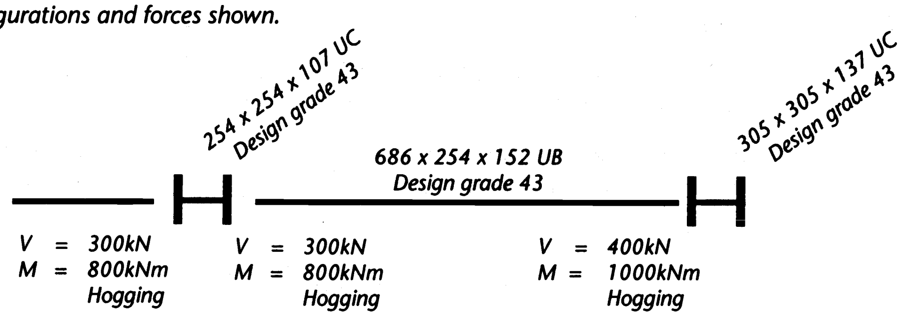
Supplementary web plate

A supplementary web plate can be used to increase web tension, shear and compression resistance. The Web Stiffened column tables take account of the increased web panel shear capacity and compression capacity available when a supplementary web plate, of the same grade steel as the column, is provided on one side only of the sections listed. The supplementary web plate should be sized in accordance with Section 2, STEP 6D. In the standard details shown, the web plate is not required to increase web tension resistance, and "fill in" welds are not required.

Worked Examples Using the Capacity Tables

DESIGN EXAMPLE 1

Design connections for the configurations and forces shown.



Left hand connection.

Select 686 UB series beam table (Page 151) and try the unstiffened column (Page182)

From the beam table, choose the connection type which will develop a moment of at least 800kNm.

An extended end plate connection for a 686 x 254 x 152 has a capacity of 925kNm.

Tabulate the potential bolt row forces from the beam side of the connection alongside the potential bolt row forces from the column side of the connection, then choose the minimum force for each row and calculate the moment capacity by multiplying the bolt row forces by the respective lever arms.

Tension

Row No	Beam side (page 151)	Column side (page 182)	Minimum		Lever arm		Moment capacity
1	364kN	375kN	364kN	x	0.72m	=	262kNm
2	396kN	304kN	304kN	x	0.62m	=	189kNm
3	338kN	287kN	287kN	x	0.53m	=	152kNm
4	280kN	287kN	280kN	x	0.44m	=	123kNm
5	223kN	287kN	223kN	x	0.35m	=	78kNm
6	165kN	287kN	165kN	x	0.26m	=	43kNm
		TOTALS	**1623kN**				**847kNm**

∴ Provide a standard extended end plate connection to the beam

Compression — *Compressive force on column is 1623 kN, but the unstiffened column compression capacity is only 845kN (page 182).*

∴ Provide a pair of compression stiffeners designed in accordance with Section 2 STEP 6A

Vertical Shear — *Applied shear is 300 kN.*

End plate thickness = 25mm Column flange thickness = 20.5mm

By inspection of page 221 for M24 8.8 bolts, bolt shear governs rather than bearing.

Bottom row dedicated to shear	*provides*	*264kN.*
Each tension row	*provides*	*106kN.*

Connection resistance = 264 + (6 x 106) = 900kN > 300kN, OK

Web Panel Shear — *Equal moments on each side result in zero applied web panel shear. (Note the unstiffened column resistance is 551kN.)*

Welds — *Weld to End Plate as shown in table on page 151*

Provide:

Tension Flange	-	**10FW+9pp**
Web	-	**10FW**
Compression Flange (bearing fit specified)	-	**8FW**

DESIGN EXAMPLE 1 (Continued)

Right hand connection.

The 686 UB series beam table on page 151 shows that a mini haunch 230mm deep will develop a moment of 1130kNm, but it must be checked against the column side.

The potential bolt row forces are tabulated in a similar manner as shown for the left hand connection, and the accumulated moment capacity is also noted in order to check the number of tension rows needed, thus:

Tension

Row No.	Beam side (page 151)	Column side (page 182)	Minimum		Lever arm		Moment capacity per row	Moment capacity cumulative
1	396kN	394kN	394kN	x	0.85m	=	337kNm	337kNm
2	353kN	354kN	353kN	x	0.76m	=	269kNm	606kNm
3	311kN	301kN	301kN	x	0.67m	=	202kNm	808kNm
4	268kN	301kN	268kN	x	0.58m	=	156kNm	964kNm
5	226kN	301kN	226kN	x	0.49m	=	111kNm	1075kNm
6	183kN	301kN	183kN	x	0.4m	=	74kNm	1149kNm
		TOTALS 5 Rows	**1542kN**					**1075kNm**
		TOTALS 6 Rows	**1725kN**					**1149kNm**

The moment capacity for 5 rows will suffice.

∴ **Provide a standard 230 deep mini haunch connection to the beam with 5 rows of tension bolts.**

Compression

Compressive force on column is 1542 kN, but the unstiffened column resistance is 964kN (page 182).

∴ **Provide a pair of compression stiffeners designed in accordance with Section 2 STEP 6A**

Vertical Shear

Applied shear is 400 kN.

End plate thickness = 25mm *Column flange thickness = 21.7mm*
By inspection of page 221 for M24 8.8 bolts, bolt shear governs rather than bearing.
Bottom row dedicated to shear provides 264kN
Each tension row provides 106kN
Connection resistance = 264 + (5 x 106) = 794kN > 400kN OK

Web Panel Shear

The unstiffened web panel shear resistance is 703kN.

A supplementary web plate will only increase web panel shear resistance to 1244kN, whereas the applied web panel shear is 1542kN. A possible solution is to increase haunch depth with fewer bolt rows to reduce applied panel shear. A 400 deep haunch, 4 rows of tension bolts and supplementary web plate will be satisfactory.

∴ **Provide a 400 deep haunch to the beam, and a supplementary web plate to the column**

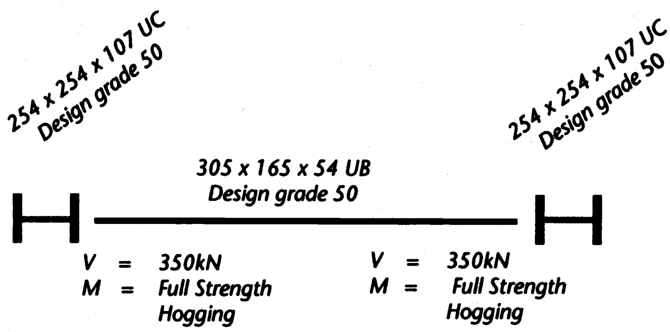
Welds

Welds to End Plate as shown in table on page 151

Provide	**Tension Flange**	-	**10FW+9pp**
	Web	-	**10FW**
	Compression Flange	-	**8FW**
	(bearing fit specified)		

DESIGN EXAMPLE 2

Provide a full strength connection for beam to column as shown:



Page 177 (305 UB series in design grade 50, M20 8.8 bolts) shows M_{cx} for a 305 x 165 x 54 UB = 299kNm.
By inspection of the tables a 280mm deep mini haunch is required.
The potential bolt forces an unstiffened column are obtained from page 185.
Comparing beam and column bolt forces:

Tension

Row No.	Beam side (page 177)	Column side (page 185)	
1	274kN	274kN	Inspection shows that the column side
2	226kN	274kN	forces are all equal or greater than the
3	179kN	274kN	beam side. ∴ **Beam side governs.**
ΣF_r	679kN		

As the beam side governs for each bolt row, the connection moment capacity may be taken directly from the table, ie 300kNm.

∴ **Provide a standard 280 deep mini haunch connection to the beam (design grade 50 steel)**

Compression *The minimum haunch flange thickness is 10mm.*

∴ **Provide a haunch cutting from the 305 x 165 x 54 beam section (T_b = 13.7mm).**

Compressive force on the column ΣF_r = 679 kN < 1008kN OK (page 185)

Vertical Shear *Applied shear is 350 kN.*
End plate thickness = 25mm Column flange thickness = 21.7mm
By inspection of page 221 for M20 8.8 bolts, bolt shear governs rather than bearing.
Bottom row dedicated to shear provides 184kN
Each tension row provides 74kN
Connection resistance = 184 + (3 x 74) = 406kN > 350kN OK

Panel Shear *680kN applied*
The unstiffened column resistance (page 185) = 717kN > 679kN OK

Welds *Welds as shown in table on page 177 (to be made with E51 electodes to BS 639).*

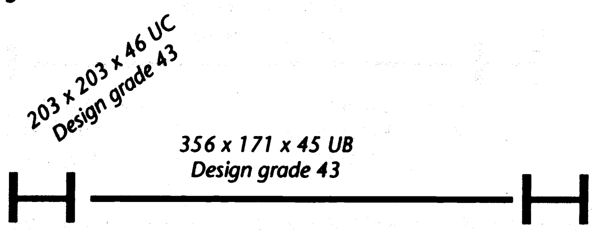
∴ **Provide:**

Tension Flange	-	**10FW**
Web	-	**6FW**
Compression Flange (bearing fit specified)	-	**8FW**

DESIGN EXAMPLE 3

In this example the bolt row forces on the beam side are adjusted to take account of the axial tension present in the beam. This approach will produce conservative results. Increased capacity may be obtained by following the rigorous approach.

Design a connection for the configuration and forces shown which include axial tension:



V = 100kN
M = 90kNm
T = 60kN (Axial tension)

Try an extended endplate M20 bolts (page 159) ΣF_r = 639kN.

Tension *Due to the axial tension, the sum of the bolt row forces ΣF_r will no longer be limited to the beam flange compression capacity. The bolt row forces may be increased, up to the maximum values shown (in brackets).*

ΣF_r may be increased to 639 + 60 = 699kN

By inspection of the table,

F_{r3} can be increased by 46kN from 142kN to 188kN (maximum).
F_{r4} can provide the remaininng 14kN.

Comparing beam and column bolt row forces :

Row No.	Beam (page 159)	Column (page 185)	Minimum		Lever Arm		Moment capacity
1	226kN	198kN	198kN	x	0.39m	=	77kNm
2	274kN	97kN	97kN	x	0.29m	=	28kNm
3	188kN	90kN	90kN	x	0.20m	=	18kNm
4	14kN	90kN	14kN	x	0.11m	=	2kNm
		TOTALS	**399kN**			Σ=	**125kNm**

Modified moment (STEP 4):

$M_m = 90 + (60 \times D_b/2) = 90 + (60 \times 0.176)$ = 101kNm < 125kNm OK
In practice, row 4 makes a very small contribution to the moment capacity, and could be omitted.
Omitting row 4, $\Sigma F_r = 198 + 97 + 90$ = 385kN

Compression *Resolving, the compression force, F_c = 385 – 60 = 325kN*

From page 185, P_c = 331kN > 325kN satisfactory

Panel Shear

From page 185, P_v = 245kN < 325kN unsatisfactory

Two solutions are possible without increasing the column weight or grade:

(a) provide shear stiffening.

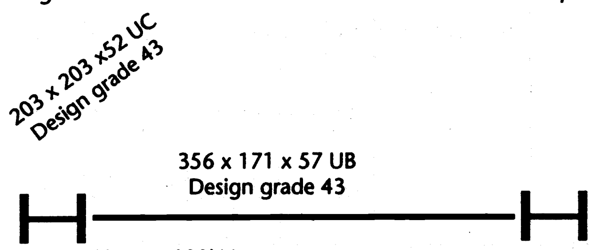
or ***(b) reduce ΣF_r and F_c by providing a haunch.***

Vertical Shear and welds are considered in similar manner to examples 1 and 2

DESIGN EXAMPLE 4

In this example the bolt row forces on the beam side are adjusted to take account of the axial compression present in the beam. This approach will produce conservative results. Increased capacity may be obtained by following the rigorous approach.

Design a connection for the configuration and forces shown which include axial compression:



V = 100kN
M = 150kNm
C = 100kN (Axial compression)

Try an extended endplate M20 bolts
(page 159) ΣF_r = *795kN*

Compression (beam side) *Axial compression contributes to the force in the compression flange,*

$P_{c(beam)}$ *(page 159)* = *861kN*

resolving, maximum total tension = 861 − 100 = 761kN

Tension ΣF_r *is limited to* 761kN

Thus the bolt row forces must be reduced by 795 − 761 = 34kN
This can all be deducted from row 4, ie F_{r4} = 105 − 34 = 71kN

Comparing beam and column bolt row forces :

Row No.	Beam (page 159)	Column (page 185)	Minimum		Lever Arm		Moment capacity per row	Moment capacity cumulative
1	226kN	211kN	211kN	x	0.39m	=	82kNm	82kNm
2	274kN	172kN	172kN	x	0.29m	=	50kNm	132kNm
3	190kN	118kN	118kN	x	0.20m	=	24kNm	156kNm
4	71kN	118kN	71kN	x	0.11m	=	8kNm	164kNm
		TOTALS 3 Rows	**501kN**					**156kNm**
		TOTALS 4 Rows	**572kN**					**164kNm**

The modified moment (STEP 4)

$M_m = 150 - (100 \times D_b/2) = 150 - (100 \times 0.179) = 132kNm < 156kNm$

The moment capacity for 3 rows will suffice

Compression (column side) *Resolving, the compression force,* F_c = 501 + 100 = 601kN

From page 185, P_c = 379kN < 601kN *unsatisfactory*

Panel Shear *From page 185,* P_v = 272kN < 601kN *unsatisfactory*

Two solutions are possible without increasing the column weight or grade:

(a)** **provide diagonal and compression stiffeners.

or** **(b)** **reduce ΣF_r ***and*** F_c ***by providing a haunch.***

Vertical Shear and welds are considered in similar manner to examples 1 and 2

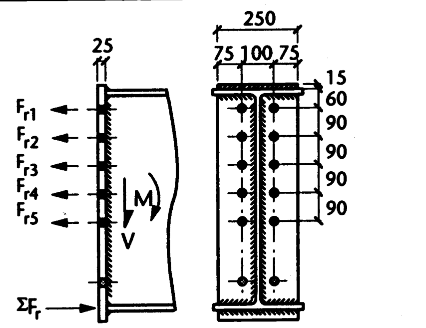
MOMENT CAPACITIES

762 x 267 UB
DESIGN GRADE 43

250 x 25 END PLATE - DESIGN GRADE 43

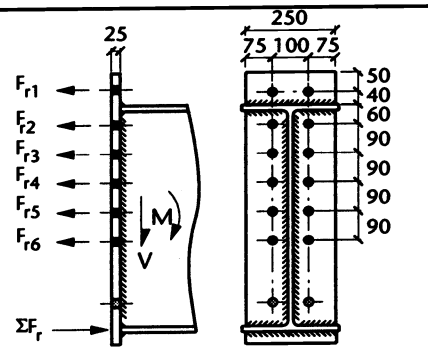
M24 8.8 BOLTS

FLUSH END PLATE



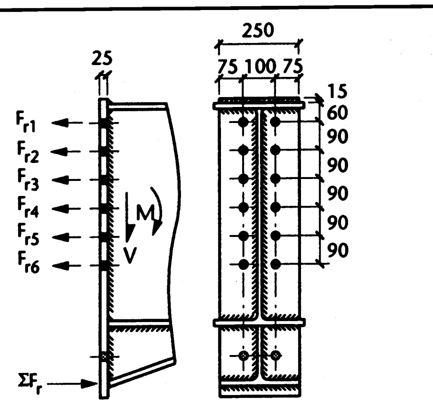
Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	F_{r5} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
762 x 267 x 197	396	345	294	243	191	1469	2356	**805**	1900	In tension zone = 106 Dedicated to shear = 264
173	396	344	293	241	190	1464	2003	**795**	1640	
147	396	344	292	240	188	1460	1623	**784**	1370	

EXTENDED END PLATE



Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	F_{r5} kN	F_{r6} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
762 x 267 x 197	364	396	345	294	243	191	1832	2356	**1095**	1900	In tension zone = 106 Dedicated to shear = 264
173	364	396	344	293	241	190	1828	2003	**1083**	1640	
147	364	396	344	292	227 (240)	0 (188)	1623	1623	**1004**	1370	

MINI HAUNCH - FOR ALL 762 x 267 UBs



Haunch Depth mm	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	F_{r5} kN	F_{r6} kN	ΣF_r kN	Moment Capacity kNm	Min. thickness Haunch Flange mm	Bolt Shear Capacity kN per row
260 *	396	358	320	283	245	207	1809	**1357**	23	In tension zone = 106 Dedicated to shear = 264
300	396	360	323	287	251	215	1832	**1444**	23	
350	396	361	327	292	258	223	1858	**1553**	24	
400	396	363	330	297	264	231	1882	**1664**	24	
450	396	365	330	302	270	239	1900	**1773**	24	
500	396	366	329	306	276	246	1917	**1882**	24	
550	396	367	328	309	280	251	1931	**1989**	25	

** minimum recommended haunch depth.*

WELDS

Serial Size	Tension Flange Extended mm	Tension Flange Flush mm	Web mm	Compression Flange in direct bearing mm
762 x 267 x 197	10FW+9pp	10FW	12FW	8FW
173	10FW+9pp	10FW	12FW	8FW
147	10FW+9pp	10FW	10FW	8FW

See: Notes - pages 142 - 143

Examples - pages 145 - 149

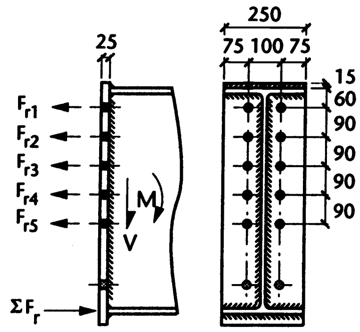
Column tables - pages 182 - 184

MOMENT CAPACITIES

686 x 254UB
DESIGN GRADE 43

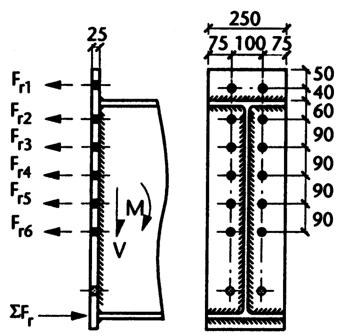
250 x 25 END PLATE - DESIGN GRADE 43
M24 8.8 BOLTS

FLUSH END PLATE



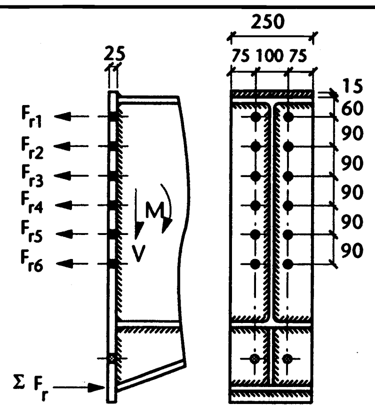
Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	F_{r5} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
686 x 254 x 170	396	339	281	224	166	1406	2198	**672**	1490	In tension zone = 106 Dedicated to shear = 264
152	396	338	280	223	165	1402	1948	**665**	1330	
140	396	338	280	222	164	1400	1762	**659**	1210	
125	396	338	279	221	162	1395	1503	**652**	1060	

EXTENDED END PLATE



Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	F_{r5} kN	F_{r6} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
686 x 254 x 170	364	396	339	281	224	166	1770	2198	**934**	1490	In tension zone = 106 Dedicated to shear = 264
152	364	396	338	280	223	165	1766	1948	**925**	1330	
140	364	396	338	280	222	164	1763	1762	**919**	1210	
125	371	396	338	279	119 (221)	0 (162)	1503	1503	**838**	1060	

MINI HAUNCH - FOR ALL 686 x 254 UBs



Haunch Depth mm	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	F_{r5} kN	F_{r6} kN	ΣF_r kN	Moment Capacity kNm	Min. thickness Haunch Flange mm	Bolt Shear Capacity kN per row
230 *	396	353	311	268	226	183	1737	**1130**	22	In tension zone = 106 Dedicated to shear = 264
250	396	354	313	271	230	188	1752	**1172**	22	
300	396	357	317	278	239	199	1786	**1279**	23	
350	396	359	321	284	247	210	1817	**1387**	23	
400	396	361	325	290	254	219	1845	**1497**	23	
450	396	362	329	295	261	228	1871	**1611**	24	

** minimum recommended haunch depth.*

WELDS

Serial Size	Tension Flange: Extended mm	Tension Flange: Flush mm	Web mm	Compression Flange in direct bearing mm
686 x 254 x 170	10FW+9pp	10FW	12FW	8FW
152	10FW+9pp	10FW	10FW	8FW
140	10FW+9pp	10FW	10FW	8FW
125	12FW	10FW	10FW	8FW

***See:* Notes - pages 142 - 143**

***Examples* - pages 145 - 149**

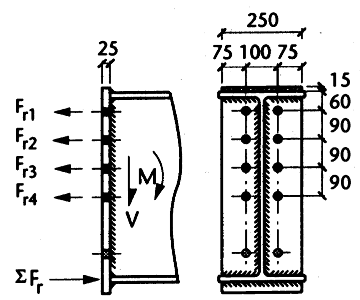
***Column tables* - pages 182 - 184**

MOMENT CAPACITIES

610 x 305 UB
DESIGN GRADE 43
M24 8.8 BOLTS

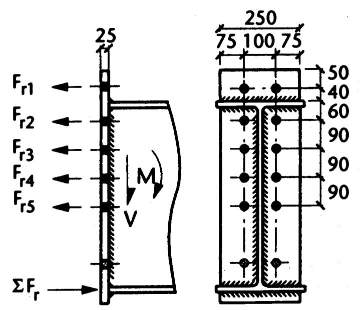
250 x 25 END PLATE - DESIGN GRADE 43

FLUSH END PLATE



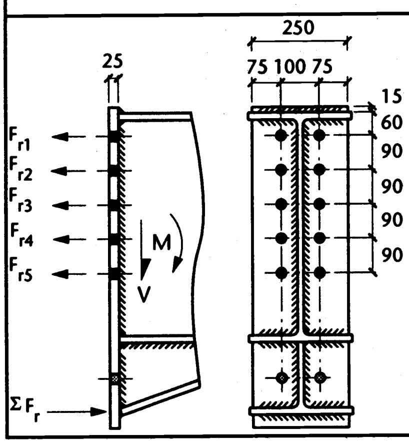
Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
610 x 305 x 238	396	332	268	204	1200	2912	536	1980	In tension zone = 106 Dedicated to shear = 264
179	396	331	265	200	1192	2189	519	1460	
149	396	330	264	198	1188	1827	510	1210	

EXTENDED END PLATE



Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	F_{r5} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kNper row
610 x 305 x 238	364	396	332	268	204	1564	2912	775	1980	In tension zone = 106 Dedicated to shear = 264
179	364	396	331	265	200	1556	2189	754	1460	
149	364	396	330	264	198	1552	1827	743	1210	

MINI HAUNCH - FOR ALL 610 x 305 UBs



Haunch Depth mm	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	F_{r5} kN	ΣF_r kN	Moment Capacity kNm	Min. thickness Haunch Flange mm	Bolt Shear Capacity kN per row
200 *	396	348	300	251	203	1498	882	19	In tension zone = 106 Dedicated to shear = 264
250	396	351	306	261	215	1529	972	19	
300	396	354	311	269	226	1556	1064	20	
350	396	356	316	276	236	1579	1157	20	
400	396	358	320	282	244	1601	1250	20	
450	396	360	324	288	252	1620	1344	21	
500	396	362	327	293	259	1637	1438	21	
560 $	396	364	327	299	266	1651	1548	21	

WELDS

Serial Size	Tension Flange Extended mm	Tension Flange Flush mm	Web mm	Compression Flange in direct bearing mm
610 x 305 x 238	10FW+9pp	10FW	10FW+8pp	8FW
179	10FW+9pp	10FW	10FW	8FW
149	10FW+9pp	10FW	10FW	8FW

** minimum recommended haunch depth.*

$ maximum recommended haunch depth.

***See:* Notes - pages 142 - 143**

Examples - pages 145 - 149

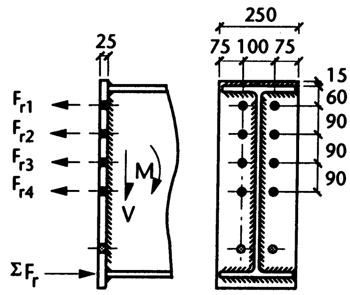
Column tables - pages 182 - 184

MOMENT CAPACITIES

610 x 229 UB
DESIGN GRADE 43
M24 8.8 BOLTS

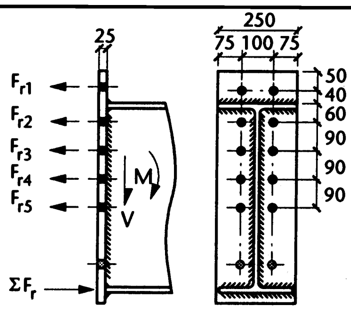
250 x 25 END PLATE - DESIGN GRADE 43

FLUSH END PLATE



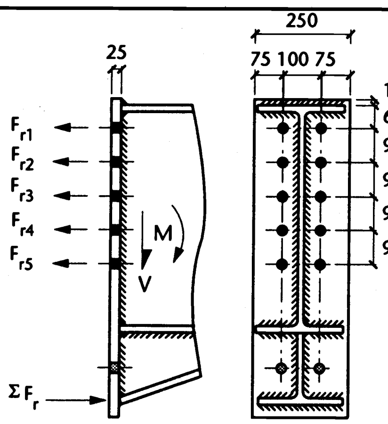
Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
610 x 229 x 140	396	331	265	200	1192	1887	519	1100	In tension zone = 106 Dedicated to shear = 264
125	396	330	264	199	1189	1665	514	973	
113	396	330	264	197	1187	1465	509	871	
101	396	329	263	196	1184	1297	503	794	

EXTENDED END PLATE



Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	F_{r5} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kNper row
610 x 229 x 140	364	396	331	265	200	1556	1887	754	1100	In tension zone = 106 Dedicated to shear = 264
125	335	396	330	264	199	1524	1665	729	973	
113	335	396	330	264	140 (197)	1465	1465	707	871	
101	371	396	329	200 (263)	0 (196)	1297	1297	665	794	

MINI HAUNCH - FOR ALL 610 x 229 UBs



Haunch Depth mm	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	F_{r5} kN	ΣF_r kN	Moment Capacity kNm	Min. thickness Haunch Flange mm	Bolt Shear Capacity kN per row
200 *	396	347	298	250	201	1492	868	21	In tension zone = 106 Dedicated to shear = 264
250	396	350	305	259	214	1524	959	21	
300	396	353	310	268	225	1552	1051	22	
350	396	356	315	275	234	1577	1147	22	

** minimum recommended haunch depth.*

WELDS

Serial Size	Tension Flange: Extended mm	Tension Flange: Flush mm	Web mm	Compression Flange in direct bearing mm
610 x 229 x 140	10FW+10pp	12FW	10FW	8FW
125	FPB	12FW	10FW	8FW
113	FPB	12FW	8FW	8FW
101	12FW	12FW	8FW	8FW

See: Notes - pages 142 - 143

Examples - pages 145 - 149

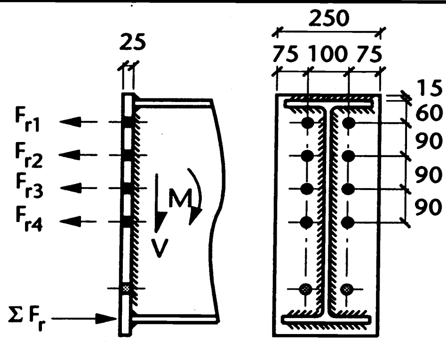
Column tables - pages 182 - 184

MOMENT CAPACITIES

250 x 25 END PLATE - DESIGN GRADE 43

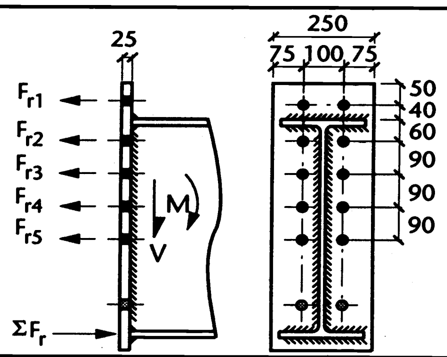
533 x 210 UB
DESIGN GRADE 43
M24 8.8 BOLTS

FLUSH END PLATE



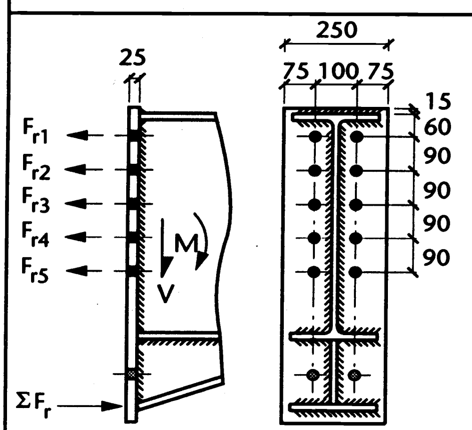
Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
533 x 210 x 122	396	321	246	170	1133	1674	418	849	In tension zone = 106 Dedicated to shear= 264
109	396	320	244	169	1129	1470	412	749	
101	396	320	244	168	1128	1356	410	694	
92	396	319	243	166	1124	1257	406	651	
82	396	319	242	104 (164)	1061	1061	389	566	

EXTENDED END PLATE



Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	F_{r5} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
533 x 210 x 122	335	396	321	246	170	1468	1674	610	849	In tension zone = 106 Dedicated to shear= 264
109	335	396	320	244	169	1464	1470	603	749	
101	335	396	320	244	62 (168)	1356	1356	579	694	
92	371	396	319	171 (243)	0 (166)	1258	1257	563	651	
82	364	396	301 (319)	0 (242)	0 (164)	1061	1061	499	566	

MINI HAUNCH - FOR ALL 533 x 210 UBs



Haunch Depth mm	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	F_{r5} kN	ΣF_r kN	Moment Capacity kNm	Min. thickness Haunch Flange mm	Bolt Shear Capacity kN per row
180 *	396	341	285	229	173	1423	702	22	In tension zone = 106 Dedicated to shear= 264
200	396	342	288	234	180	1440	737	22	
250	396	346	296	245	195	1478	826	22	
300	396	350	303	256	208	1512	916	23	

** minimum recommended haunch depth.*

WELDS

Serial Size	Tension Flange Extended mm	Tension Flange Flush mm	Web mm	Compression Flange in direct bearing mm
533 x 210 x 122	FPB	12FW	10FW	8FW
109	FPB	12FW	10FW	8FW
101	FPB	12FW	8FW	8FW
92	12FW	12FW	8FW	8FW
82	10FW	12FW	8FW	8FW

***See:* Notes - pages 142 - 143**

Examples - pages 145 - 149

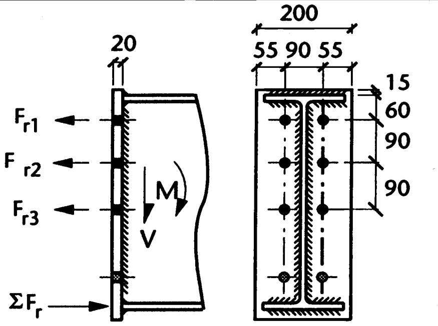
Column tables - pages 182 - 184

MOMENT CAPACITIES

457 x 191 UB
DESIGN GRADE 43

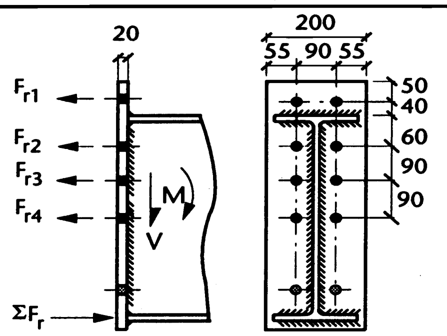
200 x 20 END PLATE - DESIGN GRADE 43
M20 8.8 BOLTS

FLUSH END PLATE



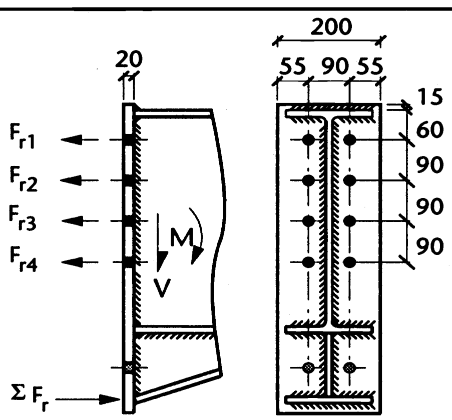
Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
457 x 191 x 98	274	212	150	636	1402	**207**	592	In tension zone = 74 Dedicated to shear = 184
89	274	212	149	635	1261	**205**	535	
82	274	211	148	633	1136	**203**	504	
74	274	211	148	632	1063	**201**	456	
67	274	210	147	631	929	**199**	405	

EXTENDED END PLATE



Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
457 x 191 x 98	226	274	212	150	862	1402	**319**	592	In tension zone = 74 Dedicated to shear = 184
89	226	274	212	149	861	1261	**316**	535	
82	230	274	211	148	863	1136	**316**	504	
74	230	274	211	148	862	1063	**314**	456	
67	226	274	210	147	857	929	**309**	405	

MINI HAUNCH - FOR ALL 457 x 191 UBs



Haunch Depth mm	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	ΣF_r kN	Moment Capacity kNm	Min. thickness Haunch Flange mm	Bolt Shear Capacity kN per row
150 *	274	228	182	136	821	**351**	13	In tension zone = 74 Dedicated to shear = 184
200	274	232	190	148	844	**401**	14	
250	274	235	197	158	864	**451**	14	
300	274	237	202	166	879	**501**	14	
350	274	237	207	174	891	**551**	15	
400	274	237	211	180	902	**602**	15	

** minimum recommended haunch depth.*

WELDS

Serial Size	Tension / Extended mm	Flange / Flush mm	Web mm	Compression Flange in direct bearing mm
457 x 191 x 98	10FW+7pp	10FW	10FW	8FW
89	10FW+7pp	10FW	8FW	8FW
82	12FW	10FW	8FW	8FW
74	12FW	10FW	8FW	8FW
67	10FW	10FW	8FW	8FW

See: Notes - pages 142 - 143

Examples - pages 145 - 149

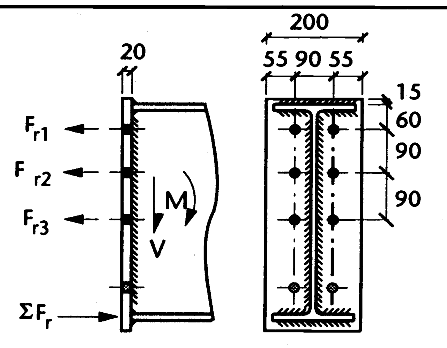
Column tables - pages 185 - 187

MOMENT CAPACITIES

457 x 152 UB
DESIGN GRADE 43

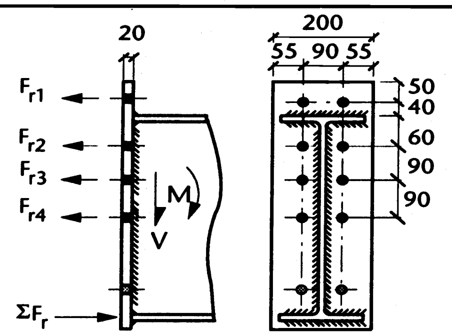
200 x 20 END PLATE - DESIGN GRADE 43
M20 8.8 BOLTS

FLUSH END PLATE



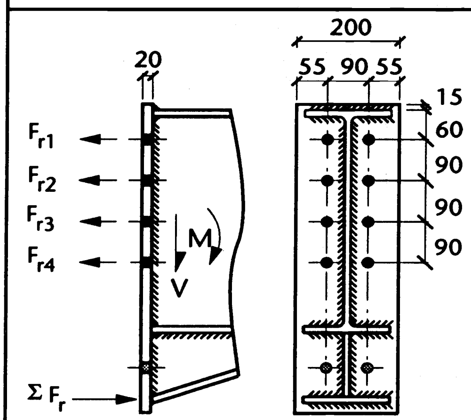
Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
457 x 152 x 82	274	212	149	635	1076	**205**	477	In tension zone = 74 Dedicated to shear = 184
74	274	211	148	634	963	**203**	430	
67	274	211	147	632	877	**201**	396	
60	274	210	147	631	783	**200**	353	
52	274	210	146	630	640	**197**	301	

EXTENDED END PLATE



Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
457 x 152 x 82	211	274	212	149	846	1076	**310**	477	In tension zone = 74 Dedicated to shear = 184
74	211	274	211	148	845	963	**307**	430	
67	230	274	211	147	862	877	**313**	396	
60	226	274	210	73 (147)	782	783	**294**	353	
52	222	274	143 (210)	0 (146)	640	640	**255**	301	

MINI HAUNCH - FOR ALL 457 x 152 UBs



Haunch Depth mm	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	ΣF_r kN	Moment Capacity kNm	Min. thickness Haunch Flange mm	Bolt Shear Capacity kN per row
150 *	274	228	181	135	818	**345**	17	In tension zone = 74 Dedicated to shear = 184
200	274	232	189	147	841	**394**	17	
250	274	232	196	157	859	**443**	17	
300	274	232	202	165	873	**493**	18	
350	274	232	206	173	885	**542**	18	
400	274	232	211	179	896	**592**	18	

** minimum recommended haunch depth.*

WELDS

Serial Size	Tension Flange: Extended mm	Tension Flange: Flush mm	Web mm	Compression Flange in direct bearing mm
457 x 152 x 82	FPB	12FW	8FW	8FW
74	FPB	12FW	8FW	8FW
67	12FW	12FW	8FW	8FW
60	10FW	10FW	6FW	8FW
52	8FW	8FW	6FW	6FW

See: *Notes* - *pages 142 - 143*

Examples - *pages 145 - 149*

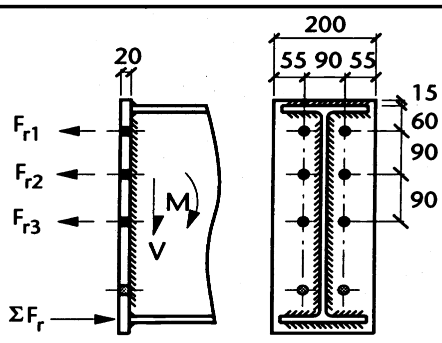
Column tables - *pages 185 - 187*

MOMENT CAPACITIES

406 x 178 UB
DESIGN GRADE 43
M20 8.8 BOLTS

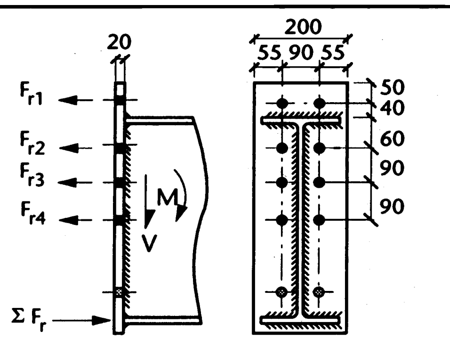
200 x 20 END PLATE - DESIGN GRADE 43

FLUSH END PLATE



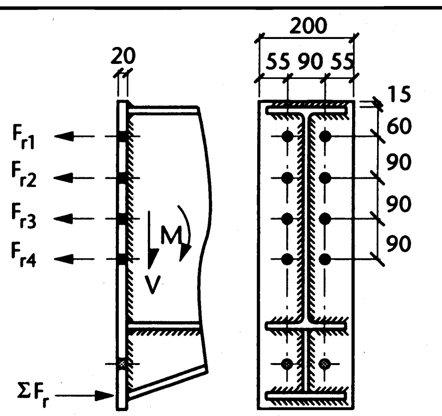
Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
406 x 178 x 74	274	203	131	608	1067	**168**	415	In tension zone = 74 Dedicated to shear = 184
67	274	202	130	606	984	**166**	370	
60	274	201	129	604	876	**164**	329	
54	274	201	128	603	745	**162**	289	

EXTENDED END PLATE



Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
406 x 178 x 74	230	274	203	131	837	1067	**270**	415	In tension zone = 74 Dedicated to shear = 184
67	230	274	202	130	836	984	**267**	370	
60	226	274	201	129	830	876	**264**	329	
54	222	274	201	48 (128)	745	745	**247**	289	

MINI HAUNCH - FOR ALL 406 x 178 UBs



Haunch Depth mm	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	ΣF_r kN	Moment Capacity kNm	Min. thickness Haunch Flange mm	Bolt Shear Capacity kN per row
130 *	274	221	168	116	779	**279**	14	In tension zone = 74 Dedicated to shear = 184
150	274	223	173	122	792	**298**	14	
200	274	228	182	136	821	**349**	14	
250	274	230	190	148	843	**399**	15	
300	274	232	196	158	860	**448**	15	

** minimum recommended haunch depth.*

WELDS

Serial Size	Tension Flange		Web	Compression Flange in direct bearing
	Extended	Flush		
	mm	mm	mm	mm
406 x 178 x 74	12FW	10FW	8FW	8FW
67	12FW	10FW	8FW	8FW
60	10FW	10FW	6FW	8FW
54	8FW	8FW	6FW	6FW

See: *Notes - pages 142 - 143*

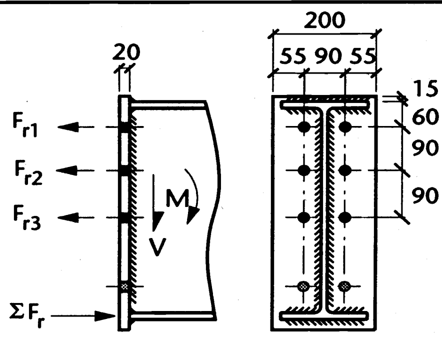
Examples - pages 145 - 149

Column tables - pages 185 - 187

MOMENT CAPACITIES

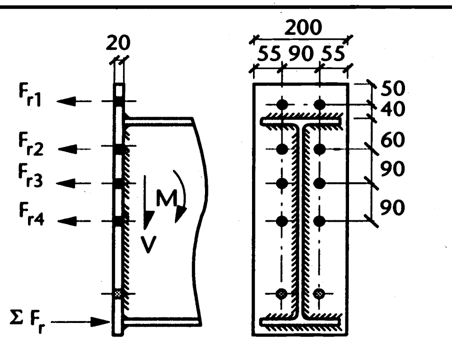
406 x 140 UB
DESIGN GRADE 43
200 x 20 END PLATE - DESIGN GRADE 43
M20 8.8 BOLTS

FLUSH END PLATE



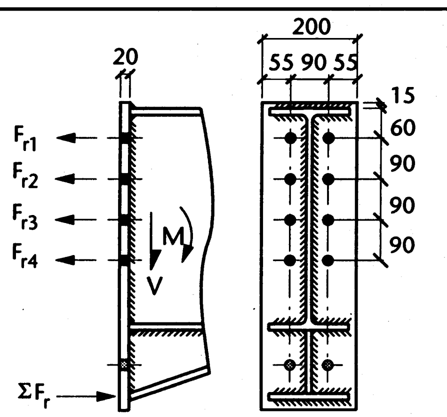
Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
406 x 140 x 46	274	201	128	602	614	**162**	245	In tension zone = 74
39	274	195 (200)	0 (126)	469	469	**139**	198	Dedicated to shear = 184

EXTENDED END PLATE



Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
406 x 140 x 46	222	274	118 (201)	0 (128)	614	614	**218**	245	In tension zone = 74
39	222	247 (274)	0 (200)	0 (126)	469	469	**178**	198	Dedicated to shear = 184

MINI HAUNCH - FOR ALL 406 x 140 UBs



Haunch Depth mm	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	ΣF_r kN	Moment Capacity kNm	Min. thickness Haunch Flange mm	Bolt Shear Capacity kN per row
130 *	274	221	166	113	773	**275**	17	In tension zone = 74
150	274	222	171	119	787	**293**	18	Dedicated to shear = 184
200	274	227	181	134	8168	**342**	18	
250	274	231	189	146	839	**391**	19	
300	274	231	195	156	856	**439**	19	

** minimum recommended haunch depth.*

WELDS

Serial Size	Tension Flange Extended mm	Tension Flange Flush mm	Web mm	Compression Flange in direct bearing mm
406 x 140 x 46	8FW	8FW	6FW	6FW
39	8FW	8FW	6FW	6FW

See: Notes - pages 142 - 143

Examples - pages 145 - 149

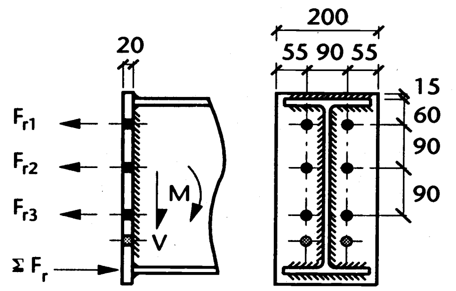
Column tables - pages 185 - 187

MOMENT CAPACITIES

356 x 171 UB
DESIGN GRADE 43
M20 8.8 BOLTS

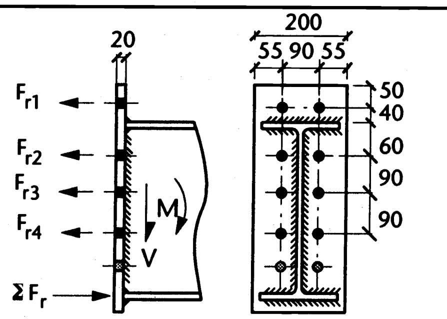
200 x 20 END PLATE - DESIGN GRADE 43

FLUSH END PLATE



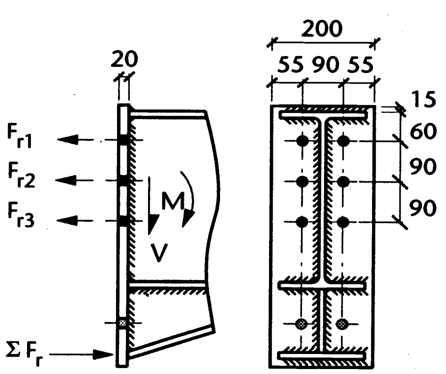
Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
356 x 171 x 67	274	191	107	572	1047	**133**	334	In tension zone = 74 Dedicated to shear = 184
57	274	190	105	569	861	**130**	278	
51	274	189	104	567	759	**129**	246	
45	274	188	102	564	639	**127**	213	

EXTENDED END PLATE



Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
356 x 171 x 67	230	274	191	107	802	1047	**224**	334	In tension zone = 74 Dedicated to shear = 184
57	226	274	190	105	795	861	**219**	278	
51	226	274	189	70 (104)	759	759	**213**	246	
45	222	274	142 (188)	0 (102)	639	639	**193**	213	

MINI HAUNCH - FOR ALL 356 x 171 UBs



Haunch Depth mm	F_{r1} kN	F_{r2} kN	F_{r3} kN	ΣF_r kN	Moment Capacity kNm	Min. thickness Haunch Flange mm	Bolt Shear Capacity kN per row
120 *	274	213	153	640	**213**	12	In tension zone = 74 Dedicated to shear = 184
150	274	217	161	652	**236**	12	
200	274	223	172	670	**274**	12	
250	274	228	182	684	**313**	12	
300	274	231	190	695	**352**	13	

** minimum recommended haunch depth.*

WELDS

Serial Size	Tension Flange: Extended mm	Tension Flange: Flush mm	Web mm	Compression Flange in direct bearing mm
356 x 171 x 67	12FW	10FW	8FW	8FW
57	10FW	10FW	6FW	8FW
51	10FW	10FW	6FW	6FW
45	8FW	8FW	6FW	6FW

See: Notes - pages 142 - 143

Examples - pages 145 - 149

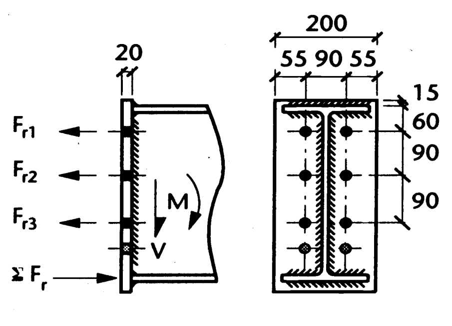
Column tables - pages 185 - 187

MOMENT CAPACITIES

356 x 127 UB
DESIGN GRADE 43

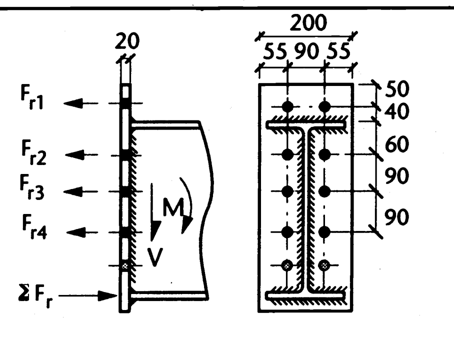
200 x 20 END PLATE - DESIGN GRADE 43
M20 8.8 BOLTS

FLUSH END PLATE



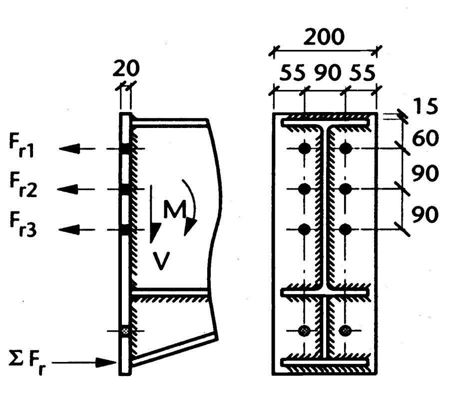
Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
356 x 127 x 39	274	188	57 (102)	519	519	122	180	In tension zone = 74 Dedicated to shear = 184
33	274	136 (187)	0 (100)	410	410	104	148	

EXTENDED END PLATE



Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
356 x 127 x 39	222	274	23 (188)	0 (102)	519	519	169	180	In tension zone = 74 Dedicated to shear = 184
33	222	188 (274)	0 (187)	0 (100)	411	411	139	148	

MINI HAUNCH - FOR ALL 356 x 127 UBs



Haunch Depth mm	F_{r1} kN	F_{r2} kN	F_{r3} kN	ΣF_r kN	Moment Capacity kNm	Min. thickness Haunch Flange mm	Bolt Shear Capacity kN per row
120 *	274	212	151	637	209	16	In tension zone = 74 Dedicated to shear = 184

** minimum recommended haunch depth.*

WELDS

Serial Size	Tension Flange Extended mm	Tension Flange Flush mm	Web mm	Compression Flange in direct bearing mm
356 x 127 x 39	8FW	8FW	6FW	6FW
33	8FW	8FW	6FW	6FW

See: Notes - pages 142 - 143

Examples - pages 145 - 149

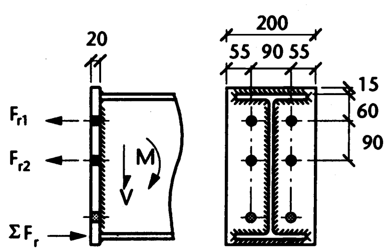
Column tables - pages 185 - 187

MOMENT CAPACITIES

305 x 165 UB
DESIGN GRADE 43
M20 8.8 BOLTS

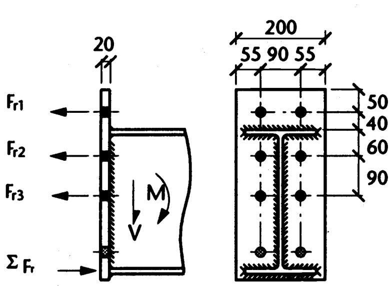
200 x 20 END PLATE - DESIGN GRADE 43

FLUSH END PLATE



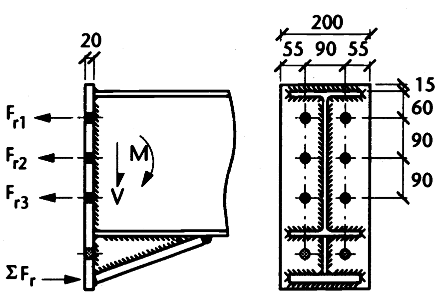
Serial Size	F_{r1} kN	F_{r2} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
305 x 165 x 54	274	173	447	880	93	232	In tension zone = 74 Dedicated to shear = 184
46	274	172	446	753	92	198	
40	274	171	445	648	91	172	

EXTENDED END PLATE



Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
305 x 165 x 54	226	274	173	673	880	171	232	In tension zone = 74 Dedicated to shear = 184
46	226	274	172	672	753	169	198	
40	222	274	152 (171)	648	648	163	172	

MINI HAUNCH - FOR ALL 305 x 165 UBs



Haunch Depth	F_{r1} kN	F_{r2} kN	F_{r3} kN	ΣF_r kN	Moment Capacity kNm	Min. thickness Haunch Flange mm	Bolt Shear Capacity kN per row
100 *	274	201	128	604	165	12	In tension zone = 74 Dedicated to shear = 184
130	274	207	140	621	187	12	
160	274	212	150	636	209	12	
190	274	216	158	647	227	12	
220	274	220	166	661	255	13	

** minimum recommended haunch depth.*

WELDS

Serial Size	Tension Flange Extended mm	Tension Flange Flush mm	Web mm	Compression Flange in direct bearing mm
305 x 165 x 54	10FW	10FW	6FW	8FW
46	10FW	10FW	6FW	6FW
40	8FW	8FW	6FW	6FW

See: Notes - pages 142 - 143

Examples - pages 145 - 149

Column tables - pages 185- 187

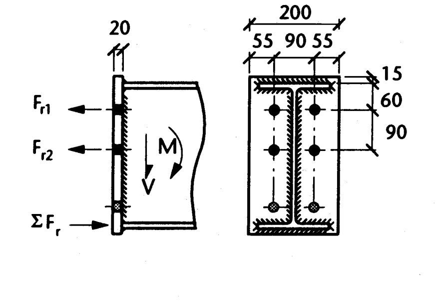
MOMENT CAPACITIES

305 x 127 UB
DESIGN GRADE 43

200 x 20 END PLATE - DESIGN GRADE 43

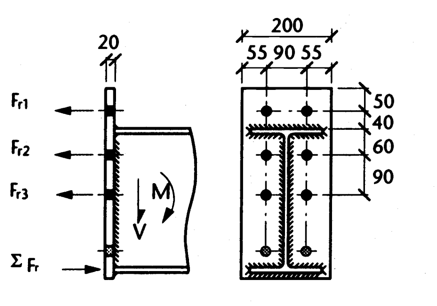
M20 8.8 BOLTS

FLUSH END PLATE



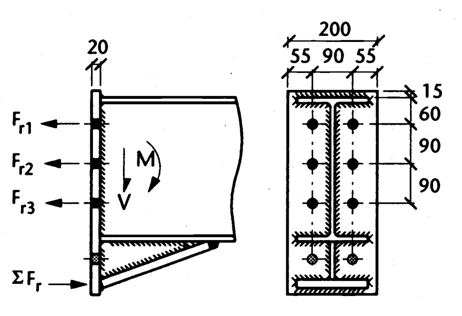
Serial Size	F_{r1} kN	F_{r2} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
305 x 127 x 48	274	173	447	675	**93**	194	In tension zone = 74 Dedicated to shear = 184
42	274	171	445	579	**92**	168	
37	274	171	445	509	**91**	149	

EXTENDED END PLATE



Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
305 x 127 x 48	226	274	173	673	675	**171**	194	In tension zone = 74 Dedicated to shear = 184
42	226	274	79 (171)	579	579	**155**	168	
37	222	274	13 (171)	509	509	**142**	149	

MINI HAUNCH - FOR ALL 305 x 127 UBs



Haunch Depth	F_{r1} kN	F_{r2} kN	F_{r3} kN	ΣF_r kN	Moment Capacity kNm	Min. thickness Haunch Flange mm	Bolt Shear Capacity kN per row
100 *	274	201	128	603	**163**	15	In tension zone = 74 Dedicated to shear = 184
130	274	207	140	621	**185**	16	
160	274	212	150	636	**207**	16	

** minimum recommended haunch depth.*

WELDS

Serial Size	Tension Flange Extended mm	Tension Flange Flush mm	Web mm	Compression Flange in direct bearing mm
305 x 127 x 48	10FW	10FW	8FW	8FW
42	10FW	10FW	6FW	8FW
37	8FW	8FW	6FW	6FW

See: Notes - pages 142 - 143

Examples - pages 145 - 149

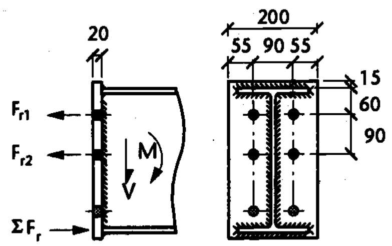
Column tables - pages 185 - 187

MOMENT CAPACITIES

305 x 102 UB
DESIGN GRADE 43
M20 8.8 BOLTS

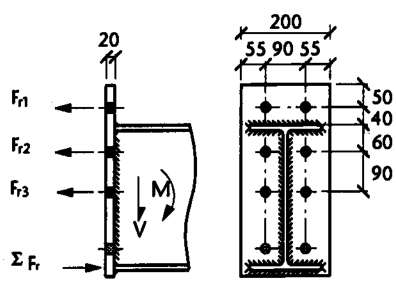
200 x 20 END PLATE - DESIGN GRADE 43

FLUSH END PLATE



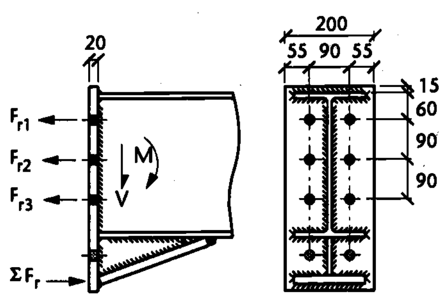
Serial Size	F_{r1} kN	F_{r2} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
305 x 102 x 33	274	152 (174)	426	426	92	132	In tension zone = 74 Dedicated to shear = 184
28	274	75 (173)	349	349	79	112	
25	266 (274)	0 (172)	266	266	64	92.4	

EXTENDED END PLATE



Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
305 x 102 x 33	222	204 (274)	0 (174)	426	426	128	132	In tension zone = 74 Dedicated to shear = 184
28	222	127 (274)	0 (173)	349	349	107	112	
25	219	47 (274)	0 (172)	266	266	86	92.4	

MINI HAUNCH - FOR ALL 305 x 102 UBs



Haunch Depth	F_{r1} kN	F_{r2} kN	F_{r3} kN	ΣF_r kN	Moment Capacity kNm	Min. thickness Haunch Flange mm	Bolt Shear Capacity kN per row
100 *	274	200	127	601	161	19	In tension zone = 74 Dedicated to shear = 184

** minimum recommended haunch depth.*

WELDS

Serial Size	Tension Flange Extended mm	Tension Flange Flush mm	Web mm	Compression Flange in direct bearing mm
305 x 102 x 33	8FW	8FW	6FW	6FW
28	8FW	8FW	6FW	6FW
25	6FW	6FW	6FW	6FW

See: Notes - pages 142 - 143

Examples - pages 145 - 149

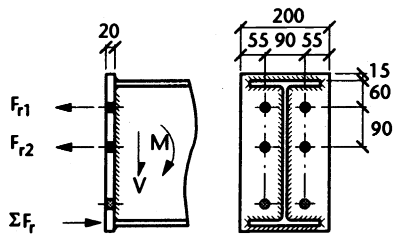
Column tables - pages 185 - 187

MOMENT CAPACITIES

254 x 146 UB
DESIGN GRADE 43
M20 8.8 BOLTS

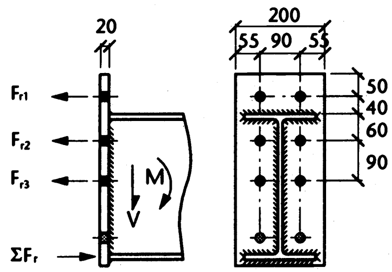
200 x 20 END PLATE - DESIGN GRADE 43

FLUSH END PLATE



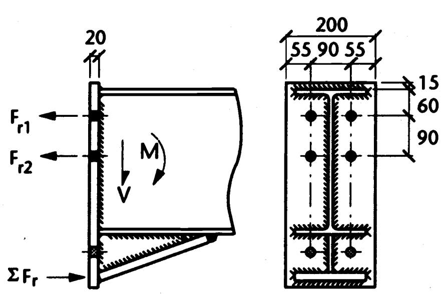
Serial Size	F_{r1} kN	F_{r2} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
254 x 146 x 43	274	146	420	720	**68**	156	In tension zone = 74 Dedicated to shear = 184
37	274	145	419	614	**67**	133	
31	274	142	416	484	**65**	109	

EXTENDED END PLATE



Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
254 x 146 x 43	226	274	146	646	720	**134**	156	In tension zone = 74 Dedicated to shear = 184
37	222	274	118 (145)	614	614	**129**	133	
31	222	262 (274)	0 (142)	484	484	**113**	108	

MINI HAUNCH - FOR ALL 254 x 146 UBs



Haunch Depth	F_{r1} kN	F_{r2} kN	F_{r3} kN	ΣF_r kN	Moment Capacity kNm	Min. thickness Haunch Flange mm	Bolt Shear Capacity kN per row
85 *	274	183	93	550	**116**	12	In tension zone = 74 Dedicated to shear = 184
100	274	188	102	564	**127**	12	
130	274	196	118	589	**148**	13	
160	274	204	132	609	**169**	13	

** minimum recommended haunch depth.*

WELDS

Serial Size	Tension Flange Extended mm	Tension Flange Flush mm	Web mm	Compression Flange in direct bearing mm
254 x 146 x 43	10FW	10FW	6FW	8FW
37	8FW	8FW	6FW	6FW
31	8FW	8FW	6FW	6FW

***See:* Notes - pages 142 - 143**

Examples - pages 145 - 149

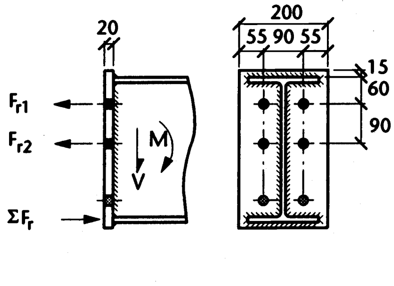
Column tables - pages 185 - 187

MOMENT CAPACITIES

254 x 102 UB
DESIGN GRADE 43
M20 8.8 BOLTS

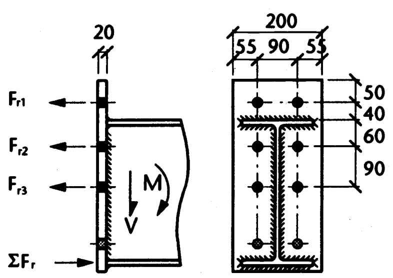
200 x 20 END PLATE - DESIGN GRADE 43

FLUSH END PLATE



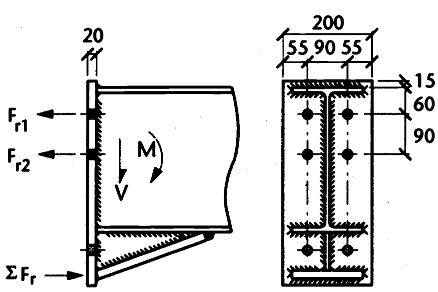
Serial Size	F_{r1} kN	F_{r2} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
254 x 102 x 28	274	119 (148)	393	393	66	97.4	In tension zone = 74 Dedicated to shear = 184
25	274	56 (146)	330	330	59	84.3	
22	266 (274)	0 (144)	266	266	51	71.6	

EXTENDED END PLATE



Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
254 x 102 x 28	222	171 (274)	0 (148)	393	393	99	97.4	In tension zone = 74 Dedicated to shear = 184
25	219	110 (274)	0 (146)	330	330	85	84.3	
22	219	47 (274)	0 (144)	266	266	73	71.6	

MINI HAUNCH - FOR ALL 254 x 102 UBs



Haunch Depth	F_{r1} kN	F_{r2} kN	F_{r3} kN	ΣF_r kN	Moment Capacity kNm	Min. thickness Haunch Flange mm	Bolt Shear Capacity kN per row
85	274	183	92	548	115	17	In tension zone = 74 Dedicated to shear = 184

WELDS

Serial Size	Tension Extended mm	Flange Flush mm	Web mm	Compression Flange in direct bearing mm
254 x 102 x 28	8FW	8FW	6FW	6FW
25	6FW	6FW	6FW	6FW
22	6FW	6FW	6FW	6FW

See: ***Notes - pages 142 - 143***

Examples - pages 145 - 149

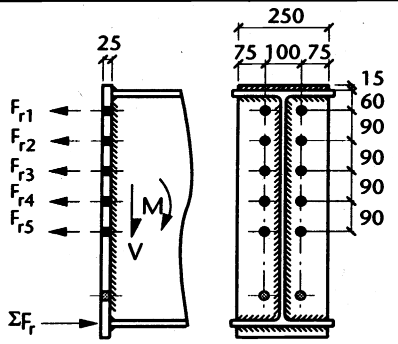
Column tables - pages 185 - 187

MOMENT CAPACITIES

762 x 267 UB
DESIGN GRADE 50

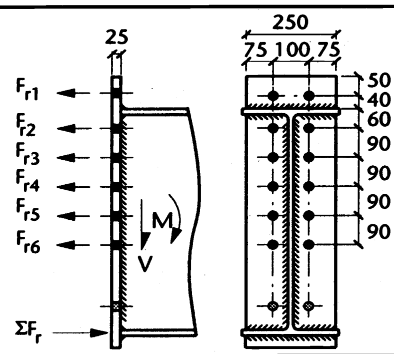
250 x 25 END PLATE - DESIGN GRADE 43
M24 8.8 BOLTS

FLUSH END PLATE



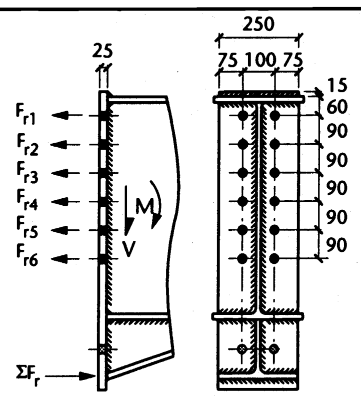
Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	F_{r5} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
762 x 267 x 197	396	345	294	243	191	1469	3067	**805**	2470	In tension zone = 106 Dedicated to shear = 264
173	396	344	293	241	190	1464	2608	**795**	2140	
147	396	344	292	240	188	1460	2113	**780**	1780	

EXTENDED END PLATE



Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	F_{r5} kN	F_{r6} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
762 x 267 x 197	364	396	345	294	243	191	1832	3067	**1095**	2470	In tension zone = 106 Dedicated to shear = 264
173	364	396	344	293	241	190	1828	2608	**1083**	2140	
147	364	396	344	292	240	188	1824	2113	**1070**	1780	

MINI HAUNCH - FOR ALL 762 x 267 UBs



Haunch Depth mm	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	F_{r5} kN	F_{r6} kN	ΣF_r kN	Moment Capacity kNm	Min. thickness Haunch Flange mm	Bolt Shear Capacity kN per row
350	396	361	327	292	258	223	1858	**1558**	18	In tension zone = 106 Dedicated to shear = 264
400	396	363	330	297	264	231	1882	**1669**	18	
450	396	365	330	302	270	239	1900	**1777**	19	
500	396	366	329	306	276	246	1917	**1888**	19	
550	396	367	328	309	281	252	1932	**1996**	19	
600	396	368	327	313	285	257	1946	**2104**	19	
650	396	369	326	313	289	262	1956	**2211**	19	
700	396	370	325	313	293	267	1964	**2317**	19	

WELDS

Serial Size	Tension Flange		Web	Compression Flange in direct bearing
	Extended mm	Flush mm	mm	mm
762 x 267 x 197	10FW+7pp	10FW	12FW	8FW
173	10FW+7pp	10FW	12FW	8FW
147	10FW+7pp	10FW	10FW	8FW

See: Notes - pages 142 - 143

Examples - pages 145 - 149

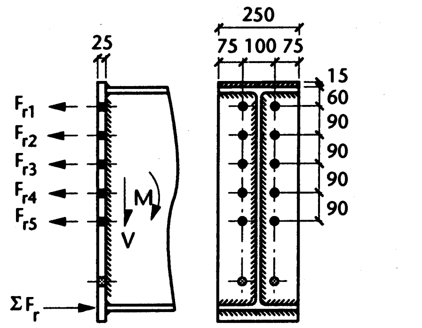
Column tables - pages 182 - 184

MOMENT CAPACITIES

686 x 254 UB
DESIGN GRADE 50
M24 8.8 BOLTS

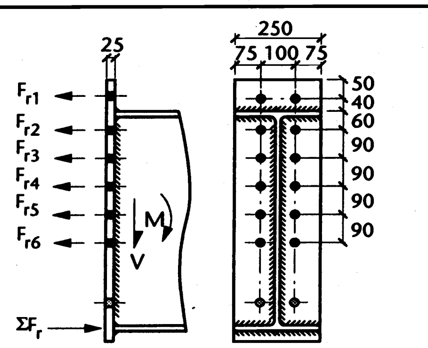
250 x 25 END PLATE - DESIGN GRADE 43

FLUSH END PLATE



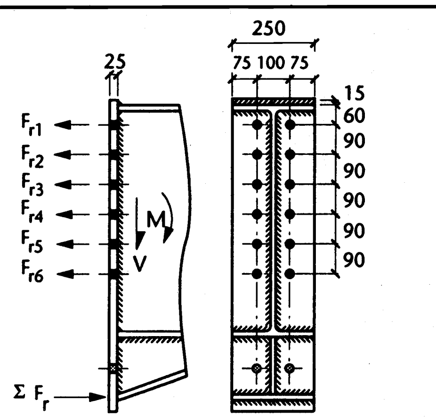
Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	F_{r5} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
686 x 254 x 170	396	339	281	224	166	1406	2862	672	1940	In tension zone = 106 Dedicated to shear = 264
152	396	338	280	223	165	1402	2536	665	1730	
140	396	338	280	222	164	1400	2294	659	1570	
125	396	338	279	221	162	1396	1956	652	1380	

EXTENDED END PLATE



Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	F_{r5} kN	F_{r6} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
686 x 254 x 170	364	396	339	281	224	166	1770	2862	934	1940	In tension zone = 106 Dedicated to shear = 264
152	364	396	338	280	223	165	1766	2536	925	1730	
140	364	396	338	280	222	164	1763	2294	919	1570	
125	371	396	338	279	221	162	1767	1956	916	1380	

MINI HAUNCH - FOR ALL 686 x 254 UBs



Haunch Depth mm	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	F_{r5} kN	F_{r6} kN	ΣF_r kN	Moment Capacity kNm	Min. thickness Haunch Flange mm	Bolt Shear Capacity kN per row
250	396	354	313	271	230	188	1752	**1172**	17	In tension zone = 106 Dedicated to shear = 264
300	396	357	317	278	239	199	1786	**1279**	18	
350	396	359	321	284	247	210	1817	**1387**	18	
400	396	361	325	290	254	219	1845	**1503**	18	
450	396	362	328	295	261	228	1870	**1612**	18	
500	396	364	326	300	268	235	1888	**1718**	19	
550	396	365	325	304	273	242	1905	**1826**	19	
600	396	367	323	308	278	249	1919	**1933**	19	
640 §	396	367	323	310	282	253	1932	**2021**	19	

§ maximum recommended haunch depth.

WELDS

Serial Size	Tension: Extended mm	Flange: Flush mm	Web mm	Compression Flange in direct bearing mm
686 x 254 x 170	10FW+7pp	10FW	12FW	8FW
152	10FW+7pp	10FW	10FW	8FW
140	10FW+7pp	10FW	10FW	8FW
125	12FW	10FW	10FW	8FW

See: Notes - pages 142 - 143

Examples - pages 145 - 149

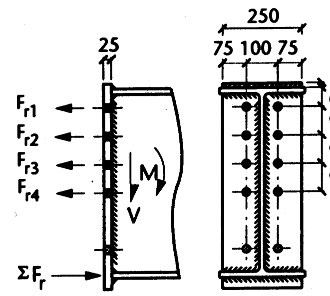
Column tables - pages 182 - 184

MOMENT CAPACITIES

610 x 305 UB
DESIGN GRADE 50
M24 8.8 BOLTS

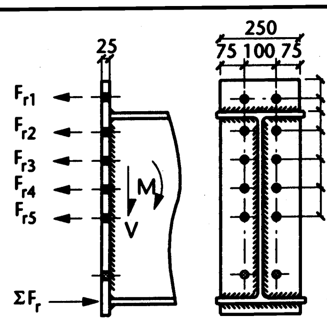
250 x 25 END PLATE - DESIGN GRADE 43

FLUSH END PLATE



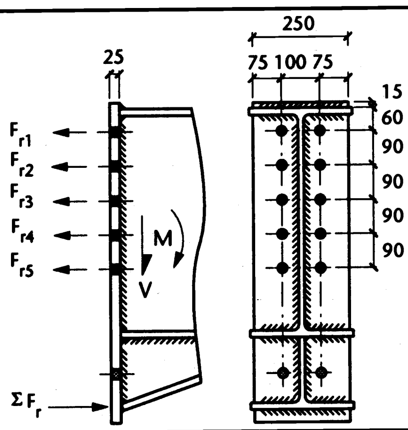
Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
610 x 305 x 238	396	332	268	204	1200	3792	536	2570	In tension zone = 106 Dedicated to shear = 264
179	396	331	265	200	1192	2850	519	1900	
149	396	330	264	198	1188	2379	510	1580	

EXTENDED END PLATE



Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	F_{r5} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
610 x 305 x 238	364	396	332	268	204	1564	3792	775	2570	In tension zone = 106 Dedicated to shear = 264
179	364	396	331	265	200	1556	2850	754	1900	
149	364	396	330	264	198	1552	2379	743	1580	

MINI HAUNCH - FOR ALL 610 x 305 UBs



Haunch Depth mm	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	F_{r5} kN	ΣF_r kN	Moment Capacity kNm	Min. thickness Haunch Flange mm	Bolt Shear Capacity kN per row
200 *	396	348	300	252	204	1499	885	15	In tension zone = 106 Dedicated to shear = 264
250	396	351	306	261	216	1530	976	15	
300	396	354	311	269	227	1557	1068	15	
350	396	356	316	276	236	1580	1160	16	
400	396	358	320	282	245	1601	1253	16	
450	396	360	324	288	252	1620	1347	16	
500	396	362	328	293	259	1638	1441	16	
560 $	396	364	327	299	267	1654	1555	16	

** minimum recommended haunch depth.*

$ maximum recommended haunch depth.

WELDS

Serial Size	Tension Extended mm	Flange Flush mm	Web mm	Compression Flange in direct bearing mm
610 x 305 x 238	10FW+7pp	10FW	10FW+8pp	8FW
179	10FW+7pp	10FW	10FW	8FW
149	10FW+7pp	10FW	10FW	8FW

See: Notes - pages 142 - 143

Examples - pages 145 - 149

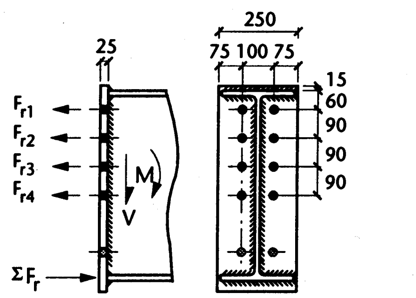
Column tables - pages 182 - 184

MOMENT CAPACITIES

610 x 229 UB
DESIGN GRADE 50

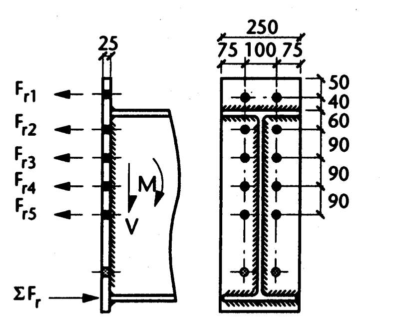
250 x 25 END PLATE - DESIGN GRADE 43
M24 8.8 BOLTS

FLUSH END PLATE



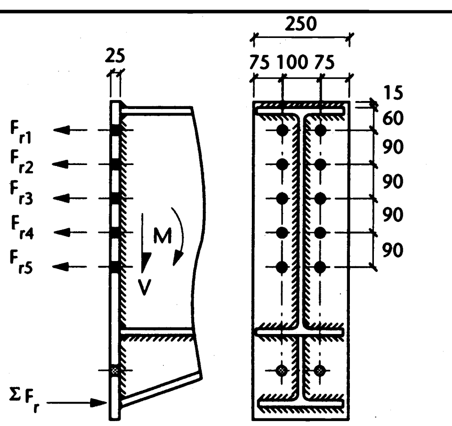
Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
610 x 229 x 140	396	331	265	200	1192	2456	519	1430	In tension zone = 106 Dedicated to shear = 264
125	396	330	264	199	1189	2168	514	1270	
113	396	330	264	197	1187	1907	509	1130	
101	396	329	263	196	1184	1674	503	1020	

EXTENDED END PLATE



Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	F_{r5} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
610 x 229 x 140	364	396	331	265	200	1556	2456	754	1430	In tension zone = 106 Dedicated to shear = 264
125	364	396	330	264	199	1553	2168	747	1270	
113	364	396	330	264	197	1551	1907	741	1130	
101	371	396	329	263	196	1555	1674	739	1020	

MINI HAUNCH - FOR ALL 610 x 229 UBs



Haunch Depth mm	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	F_{r5} kN	ΣF_r kN	Moment Capacity kNm	Min. thickness Haunch Flange mm	Bolt Shear Capacity kN per row
200 *	396	347	299	250	202	1495	872	16	In tension zone = 106 Dedicated to shear = 264
250	396	351	305	260	214	1526	963	16	
300	396	353	311	268	225	1553	1054	16	
350	396	356	315	275	235	1577	1147	17	
400	396	358	316	281	243	1595	1236	17	
450	396	360	314	287	251	1608	1325	17	
500	396	362	313	293	258	1621	1415	17	
560 $	396	363	310	298	266	1634	1522	17	

* *minimum recommended haunch depth.*

$ *maximum recommended haunch depth.*

WELDS

Serial Size	Tension Extended mm	Flange Flush mm	Web mm	Compression Flange in direct bearing mm
610 x 229 x 140	10FW+8pp	10FW	10FW	8FW
125	10FW+8pp	10FW	10FW	8FW
113	10FW+8pp	10FW	8FW	8FW
101	12FW	10FW	8FW	8FW

See: Notes - *pages 142 - 143*

Examples - *pages 145 - 149*

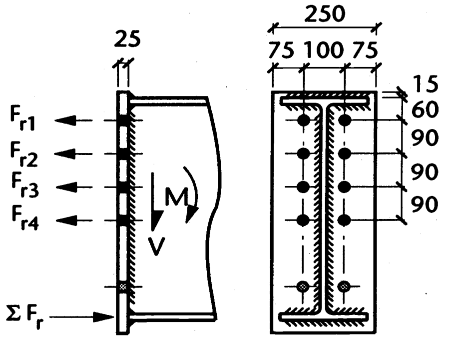
Column tables - *pages 182 - 184*

MOMENT CAPACITIES

533 x 210 UB
DESIGN GRADE 50
M24 8.8 BOLTS

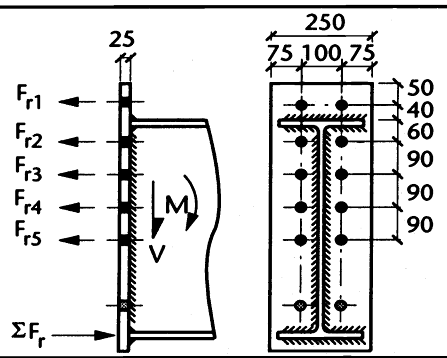
250 x 25 END PLATE - DESIGN GRADE 43

FLUSH END PLATE



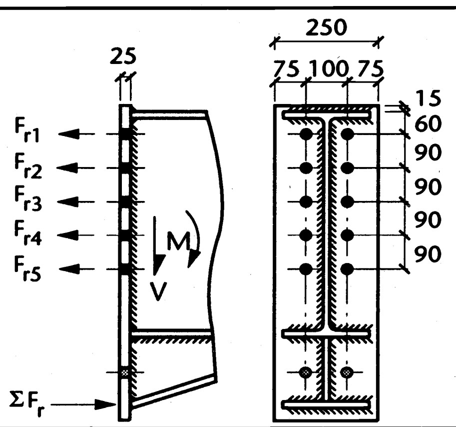
Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
533 x 210 x 122	396	321	246	170	1133	2180	**418**	1100	In tension zone = 106 Dedicated to shear = 264
109	396	320	244	169	1129	1913	**412**	975	
101	396	320	244	168	1128	1766	**410**	904	
92	396	319	243	166	1124	1623	**406**	840	
82	396	319	242	164	1121	1369	**401**	731	

EXTENDED END PLATE



Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	F_{r5} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
533 x 210 x 122	364	396	321	246	170	1496	2180	**626**	1100	In tension zone = 106 Dedicated to shear = 264
109	364	396	320	244	169	1493	1913	**620**	975	
101	364	396	320	244	168	1491	1766	**616**	904	
92	371	396	319	243	166	1496	1623	**616**	840	
82	364	396	319	242	49 (164)	1369	1369	**583**	731	

MINI HAUNCH - FOR ALL 533 x 210 UBs



Haunch Depth mm	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	F_{r5} kN	ΣF_r kN	Moment Capacity kNm	Min. thickness Haunch Flange mm	Bolt Shear Capacity kN per row
180 *	396	340	285	229	173	1423	**705**	16	In tension zone = 106 Dedicated to shear = 264
200	396	342	288	234	180	1440	**740**	16	
250	396	346	296	245	195	1478	**828**	17	
300	396	349	302	255	208	1511	**918**	17	
350	396	352	308	264	220	1540	**1009**	18	
400	396	355	313	272	230	1565	**1101**	18	

** minimum recommended haunch depth*

WELDS

Serial Size	Tension Flange: Extended mm	Tension Flange: Flush mm	Web mm	Compression Flange in direct bearing mm
533 x 210 x 122	10FW+9pp	10FW	10FW	8FW
109	10FW+9pp	10FW	10FW	8FW
101	10FW+7pp	10FW	8FW	8FW
92	12FW	10FW	8FW	8FW
82	10FW	10FW	8FW	8FW

See: Notes - pages 142 - 143

Examples - pages 145 - 149

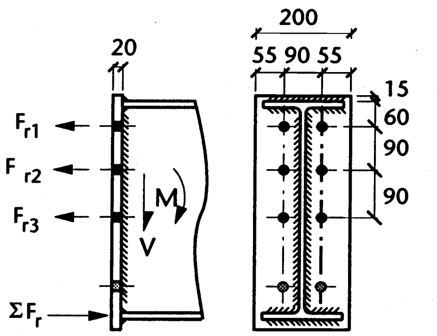
Column tables - pages 182 - 184

MOMENT CAPACITIES

457 x 191 UB
DESIGN GRADE 50
M20 8.8 BOLTS

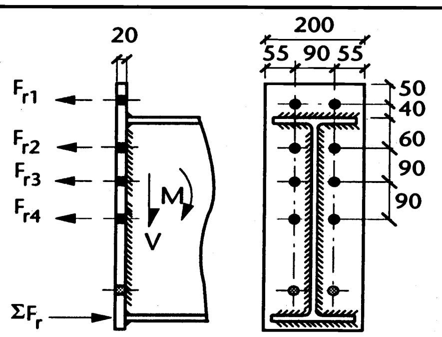
200 x 20 END PLATE - DESIGN GRADE 43

FLUSH END PLATE



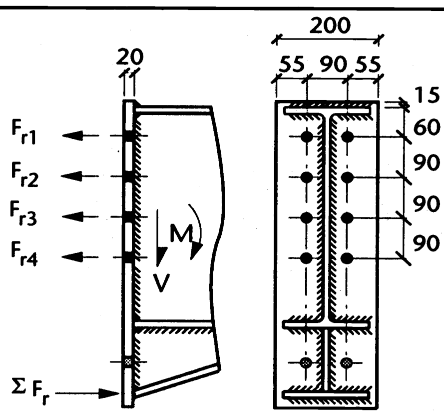
Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
457 x 191 x 98	274	212	150	636	1825	**207**	771	In tension zone = 74 Dedicated to shear = 184
89	274	212	149	635	1641	**205**	697	
82	274	211	148	633	1478	**203**	650	
74	274	211	148	632	1373	**201**	589	
67	274	210	147	631	1199	**199**	523	

EXTENDED END PLATE



Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
457 x 191 x 98	230	274	212	150	866	1825	**321**	771	In tension zone = 74 Dedicated to shear = 184
89	230	274	212	149	864	1641	**318**	697	
82	230	274	211	148	863	1478	**316**	650	
74	230	274	211	148	862	1373	**314**	589	
67	226	274	210	147	857	1199	**309**	523	

MINI HAUNCH - FOR ALL 457 x 191 UBs



Haunch Depth mm	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	ΣF_r kN	Moment Capacity kNm	Min. thickness Haunch Flange mm	Bolt Shear Capacity kN per row
200	274	232	190	148	844	**402**	11	In tension zone = 74 Dedicated to shear = 184
250	274	235	197	158	864	**452**	11	
300	274	237	202	167	880	**503**	11	
350	274	237	207	174	892	**553**	11	
400	274	237	211	180	902	**603**	11	
420 §	274	237	213	182	906	**623**	12	

§ *maximum recommended haunch depth.*

WELDS

Serial Size	Tension Extended mm	Flange Flush mm	Web mm	Compression Flange in direct bearing mm
457 x 191 x 98	12FW	8FW	8FW	8FW
89	12FW	8FW	8FW	8FW
82	12FW	8FW	8FW	8FW
74	12FW	8FW	8FW	8FW
67	10FW	8FW	8FW	8FW

See: Notes - pages 142 - 143

Examples - pages 145 - 149

Column tables - pages 185- 187

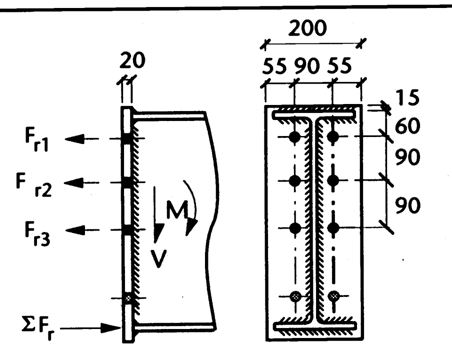
MOMENT CAPACITIES

457 x 152 UB
DESIGN GRADE 50

200 x 20 END PLATE - DESIGN GRADE 43

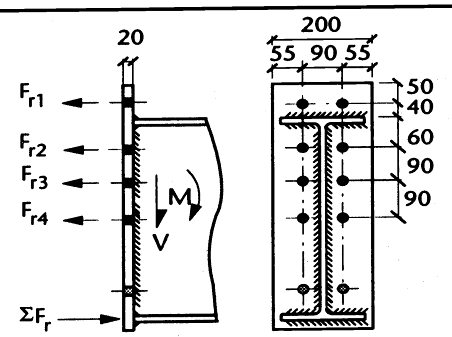
M20 8.8 BOLTS

FLUSH END PLATE



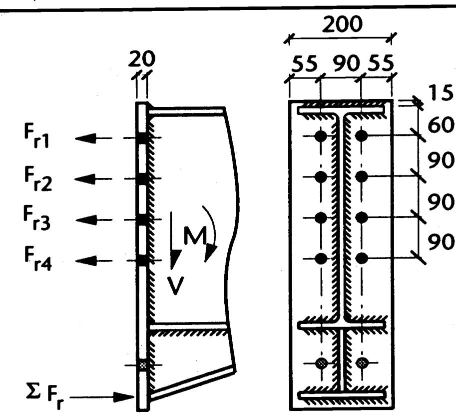
Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
457 x 152 x 82	274	212	149	635	1401	205	622	In tension zone = 74 Dedicated to shear = 184
74	274	211	148	634	1254	203	560	
67	274	211	147	632	1132	201	512	
60	274	210	147	631	1011	200	455	
52	274	210	146	630	826	197	389	

EXTENDED END PLATE



Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
457 x 152 x 82	226	274	212	149	861	1401	317	622	In tension zone = 74 Dedicated to shear = 184
74	226	274	211	148	859	1254	314	560	
67	226	274	211	147	862	1132	313	512	
60	226	274	210	147	857	1011	310	455	
52	222	274	210	120 (146)	825	826	299	389	

MINI HAUNCH - FOR ALL 457 x 152 UBs



Haunch Depth mm	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	ΣF_r kN	Moment Capacity kNm	Min. thickness Haunch Flange mm	Bolt Shear Capacity kN per row
200	274	232	190	147	843	398	13	In tension zone = 74 Dedicated to shear = 184
250	274	232	196	157	860	447	14	
300	274	232	202	166	874	496	14	
350	274	232	207	173	886	546	14	
400	274	232	211	180	897	596	14	
420 §	274	232	210	182	898	613	14	

§ *maximum recommended haunch depth.*

WELDS

Serial Size	Tension Extended mm	Flange Flush mm	Web mm	Compression Flange in direct bearing mm
457 x 152 x 82	10FW+7pp	10FW	8FW	8FW
74	10FW+7pp	10FW	8FW	8FW
67	12FW	10FW	8FW	8FW
60	10FW	10FW	6FW	8FW
52	8FW	8FW	6FW	6FW

See: Notes - pages 142 - 143

Examples - pages 145 - 149

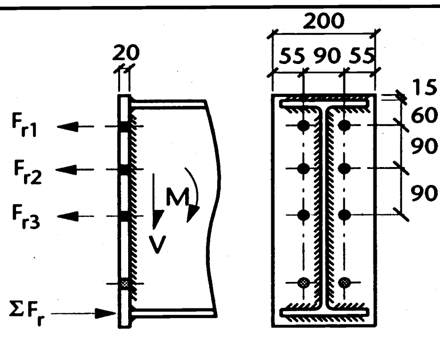
Column tables - pages 185- 187

MOMENT CAPACITIES

406 x 178 UB
DESIGN GRADE 50
M20 8.8 BOLTS

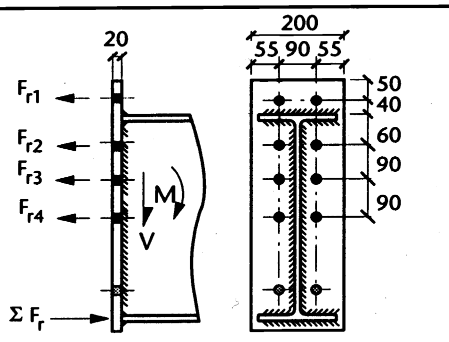
200 x 20 END PLATE - DESIGN GRADE 43

FLUSH END PLATE



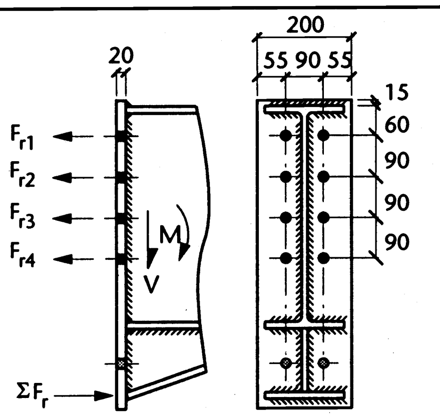
Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
406 x 178 x 74	274	203	131	608	1389	168	536	In tension zone = 74 Dedicated to shear = 184
67	274	202	130	606	1271	166	478	
60	274	201	129	604	1131	164	424	
54	274	201	128	603	962	162	373	

EXTENDED END PLATE



Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
406 x 178 x 74	230	274	203	131	837	1389	270	536	In tension zone = 74 Dedicated to shear = 184
67	230	274	202	130	836	1271	267	478	
60	226	274	201	129	830	1131	264	424	
54	222	274	201	128	825	962	259	373	

MINI HAUNCH - FOR ALL 406 x 178 UBs



Haunch Depth mm	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	ΣF_r kN	Moment Capacity kNm	Min. thickness Haunch Flange mm	Bolt Shear Capacity kN per row
200	274	228	181	135	818	346	12	In tension zone = 74 Dedicated to shear = 184
250	274	231	189	147	841	396	12	
300	274	231	196	157	858	444	12	
350	274	231	202	166	872	493	12	
370 $	274	231	204	169	877	514	12	

$ *maximum recommended haunch value.*

WELDS

Serial Size	Tension Extended mm	Flange Flush mm	Web mm	Compression Flange in direct bearing mm
406 x 178 x 74	12FW	8FW	8FW	8FW
67	12FW	8FW	8FW	8FW
60	10FW	8FW	6FW	8FW
54	8FW	8FW	6FW	6FW

See: Notes - pages 142 - 143

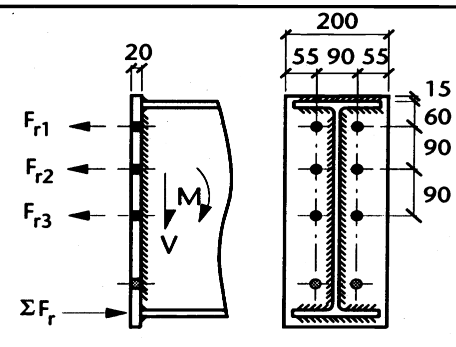
Examples - pages 145 - 149

Column tables - pages 185- 187

MOMENT CAPACITIES

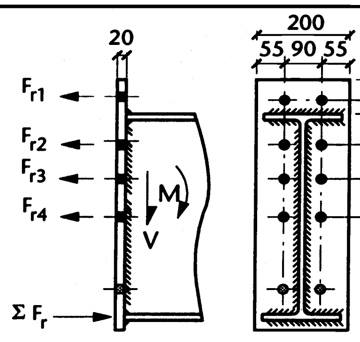
406 x 140 UB
DESIGN GRADE 50
200 x 20 END PLATE - DESIGN GRADE 43
M20 8.8 BOLTS

FLUSH END PLATE



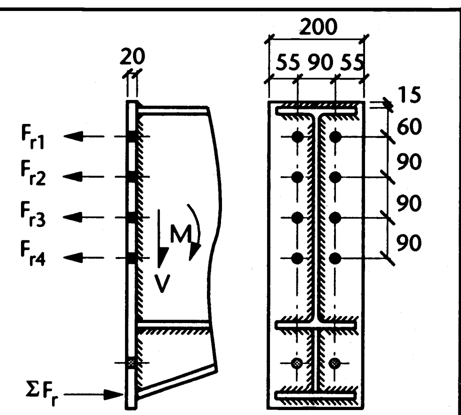
Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
406 x 140 x 46	274	201	128	603	793	**162**	316	In tension zone = 74 Dedicated to shear = 184
39	274	200	126	600	606	**159**	255	

EXTENDED END PLATE



Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
406 x 140 x 46	222	274	201	96 (128)	793	793	**254**	316	In tension zone = 74 Dedicated to shear = 184
39	222	274	110 (200)	0 (126)	606	606	**214**	255	

MINI HAUNCH - FOR ALL 406 x 140 UBs



Haunch Depth mm	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	ΣF_r kN	Moment Capacity kNm	Min. thickness Haunch Flange mm	Bolt Shear Capacity kN per row
200	274	228	181	135	818	**346**	14	In tension zone = 74 Dedicated to shear = 184

WELDS

Serial Size	Tension Flange: Extended mm	Tension Flange: Flush mm	Web mm	Compression Flange in direct bearing mm
406 x 140 x 46	8FW	8FW	6FW	6FW
39	8FW	8FW	6FW	6FW

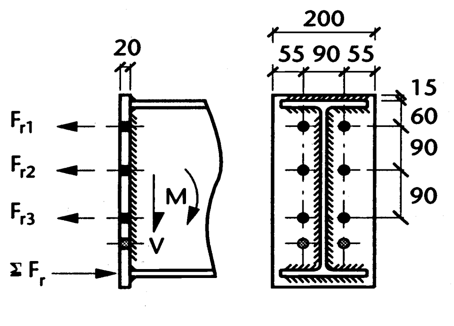
See: Notes - pages 142 - 143
Examples - pages 145 - 149
Column tables - pages 185 - 187

MOMENT CAPACITIES

356 x 171 UB
DESIGN GRADE 50
M20 8.8 BOLTS

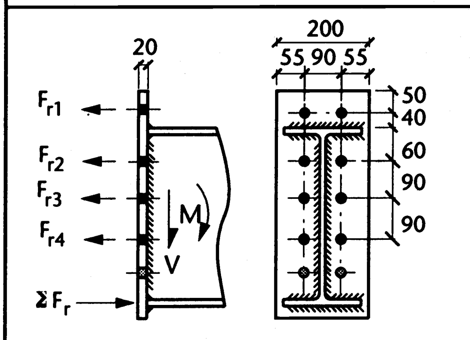
200 x 20 END PLATE - DESIGN GRADE 43

FLUSH END PLATE



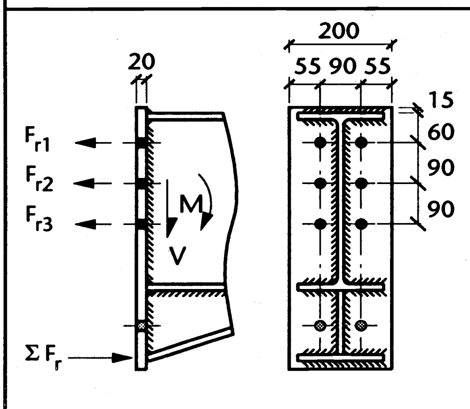
Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
356 x 171 x 67	274	191	107	572	1351	**133**	431	In tension zone = 74 Dedicated to shear = 184
57	274	190	105	569	1112	**130**	358	
51	274	189	104	567	980	**129**	318	
45	274	188	102	564	824	**127**	244	

EXTENDED END PLATE



Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
356 x 171 x 67	230	274	191	107	802	1351	**224**	431	In tension zone = 74 Dedicated to shear = 184
57	226	274	190	105	795	1112	**219**	358	
51	226	274	189	104	793	980	**217**	318	
45	222	274	188	102	787	824	**213**	244	

MINI HAUNCH - FOR ALL 356 x 171 UBs



Haunch Depth mm	F_{r1} kN	F_{r2} kN	F_{r3} kN	ΣF_r kN	Moment Capacity kNm	Min. thickness Haunch Flange mm	Bolt Shear Capacity kN per row
120 *	274	213	153	640	**214**	9	In tension zone = 74 Dedicated to shear = 184
150	274	218	161	653	**237**	9	
200	274	223	173	670	**275**	10	
250	274	228	182	684	**314**	10	
300	274	231	190	695	**353**	10	

** minimum recommended haunch depth.*

WELDS

Serial Size	Tension Extended mm	Flange Flush mm	Web mm	Compression Flange in direct bearing mm
356 x 171 x 67	12FW	10FW	8FW	8FW
57	10FW	10FW	6FW	8FW
51	10FW	10FW	6FW	6FW
45	8FW	8FW	6FW	6FW

See: Notes - pages 142 - 143

Examples - pages 145 - 149

Column tables - pages 185 - 187

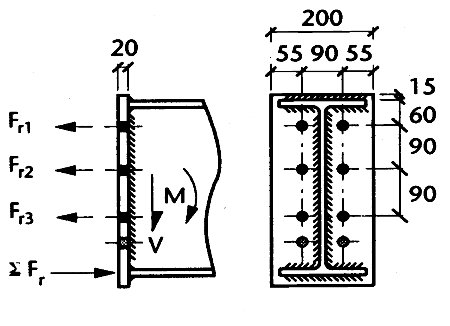
MOMENT CAPACITIES

356 x 127 UB
DESIGN GRADE 50

200 x 20 END PLATE - DESIGN GRADE 43

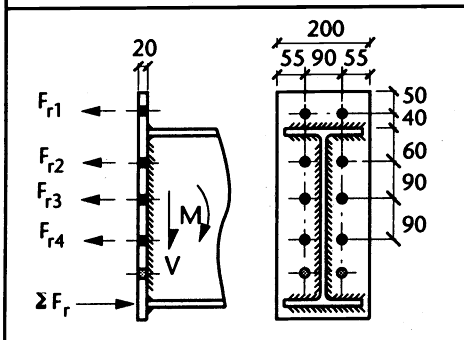
M20 8.8 BOLTS

FLUSH END PLATE



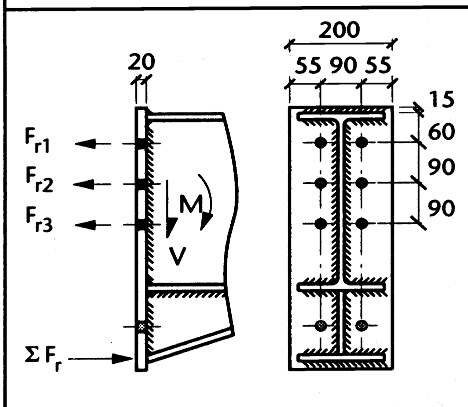
Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
356 x 127 x 39	274	188	102	564	670	**127**	232	In tension zone = 74 Dedicated to shear = 184
33	274	187	69 (100)	530	530	**121**	191	

EXTENDED END PLATE



Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
356 x 127 x 39	222	274	174 (188)	0 (102)	670	670	**199**	232	In tension zone = 74 Dedicated to shear = 184
33	222	274	34 (187)	0 (100)	530	530	**170**	191	

MINI HAUNCH - FOR ALL 356 x 127 UBs



Haunch Depth mm	F_{r1} kN	F_{r2} kN	F_{r3} kN	ΣF_r kN	Moment Capacity kNm	Min. thickness Haunch Flange mm	Bolt Shear Capacity kN per row
120 *	274	213	152	639	**212**	12	In tension zone = 74 Dedicated to shear = 184
150	274	217	160	652	**235**	13	

** minimum recommended haunch depth.*

WELDS

Serial Size	Tension Extended mm	Flange Flush mm	Web mm	Compression Flange in direct bearing mm
356 x 127 x 39	8FW	8FW	6FW	6FW
33	8FW	8FW	6FW	6FW

***See:* Notes - pages 142 - 143**

***Examples* - pages 145 - 149**

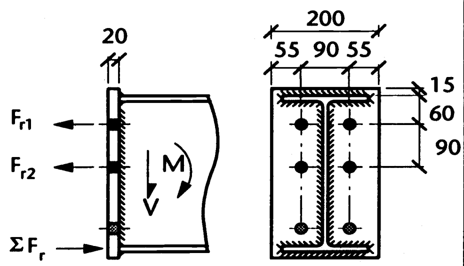
***Column tables* - pages 185 - 187**

MOMENT CAPACITIES

305 x 165 UB
DESIGN GRADE 50
M20 8.8 BOLTS

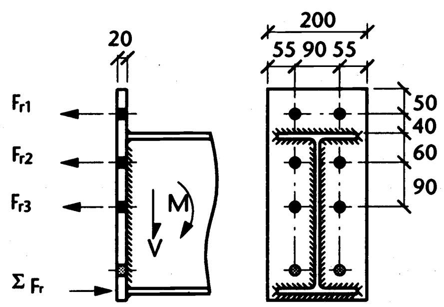
200 x 20 END PLATE - DESIGN GRADE 43

FLUSH END PLATE



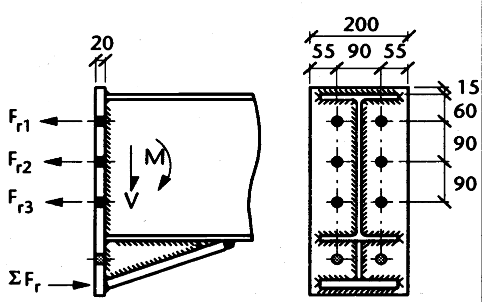
Serial Size	F_{r1} kN	F_{r2} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
305 x 165 x 54	274	173	447	1136	**93**	299	In tension zone = 74 Dedicated to shear = 184
46	274	172	446	972	**92**	256	
40	274	171	445	837	**91**	222	

EXTENDED END PLATE



Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
305 x 165 x 54	226	274	173	673	1136	**171**	299	In tension zone = 74 Dedicated to shear = 184
46	226	274	172	672	972	**169**	256	
40	222	274	171	667	837	**166**	222	

MINI HAUNCH - FOR ALL 305 x 165 UBs



Haunch Depth mm	F_{r1} kN	F_{r2} kN	F_{r3} kN	ΣF_r kN	Moment Capacity kNm	Min. thickness Haunch Flange mm	Bolt Shear Capacity kN per row
130	274	207	140	621	**185**	9	In tension zone = 74 Dedicated to shear = 184
160	274	212	150	636	**208**	9	
190	274	216	159	649	**230**	10	
220	274	220	166	661	**253**	10	
250	274	224	173	671	**276**	10	
280 §	274	226	179	679	**300**	10	

§ maximum recommended haunch depth.

WELDS

Serial Size	Tension Flange Extended mm	Tension Flange Flush mm	Web mm	Compression Flange in direct bearing mm
305 x 165 x 54	10FW	10FW	6FW	8FW
46	10FW	10FW	6FW	6FW
40	8FW	8FW	6FW	6FW

See: Notes - pages 142 - 143

Examples - pages 145 - 149

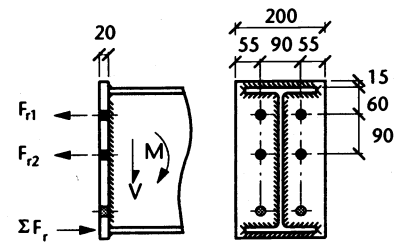
Column tables - pages 185- 187

MOMENT CAPACITIES

305 x 127 UB
DESIGN GRADE 50

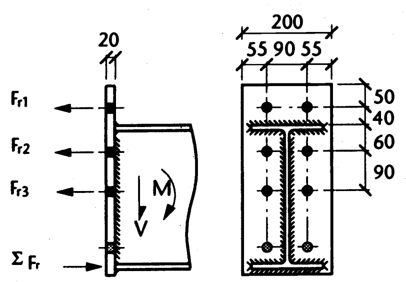
200 x 20 END PLATE - DESIGN GRADE 43
M20 8.8 BOLTS

FLUSH END PLATE



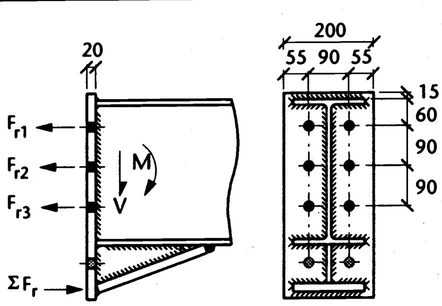
Serial Size	F_{r1} kN	F_{r2} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
305 x 127 x 48	274	173	447	871	93	251	In tension zone = 74 Dedicated to shear = 184
42	274	171	445	748	92	217	
37	274	171	445	657	91	192	

EXTENDED END PLATE



Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
305 x 127 x 48	226	274	173	673	871	171	251	In tension zone = 74 Dedicated to shear = 184
42	226	274	171	671	748	169	217	
37	222	274	161 (171)	657	657	164	192	

MINI HAUNCH - FOR ALL 305 x 127 UBs



Haunch Depth mm	F_{r1} kN	F_{r2} kN	F_{r3} kN	ΣF_r kN	Moment Capacity kNm	Min. thickness Haunch Flange mm	Bolt Shear Capacity kN per row
130	274	207	140	621	185	12	In tension zone = 74 Dedicated to shear = 184
160	274	212	150	636	207	12	
190	274	216	159	649	230	13	
220	274	220	166	661	253	13	
250	274	224	173	671	276	13	
280 §	274	226	179	679	300	13	

§ maximum recommended haunch depth.

WELDS

Serial Size	Tension Flange Extended mm	Tension Flange Flush mm	Web mm	Compression Flange in direct bearing mm
305 x 127 x 48	10FW	10FW	8FW	8FW
42	10FW	10FW	6FW	8FW
37	8FW	8FW	6FW	6FW

See: Notes - pages 142 - 143
Examples - pages 145 - 149
Column tables - pages 185 - 187

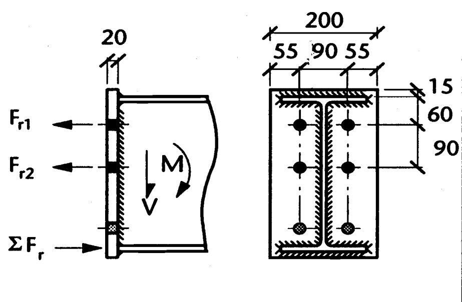
MOMENT CAPACITIES

305 x 102 UB
DESIGN GRADE 50

200 x 20 END PLATE - DESIGN GRADE 43

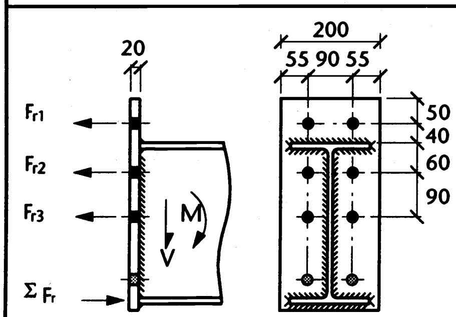
M20 8.8 BOLTS

FLUSH END PLATE



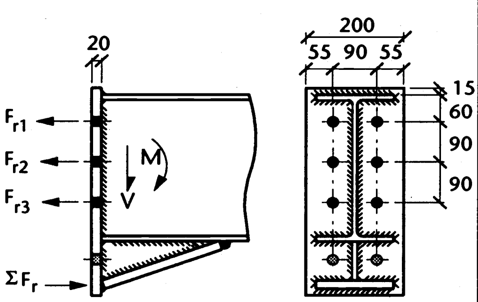
Serial Size	F_{r1} kN	F_{r2} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
305 x 102 x 33	274	174	448	550	95	171	In tension zone = 74 Dedicated to shear = 184
28	274	173	447	451	94	145	
25	274	69 (172)	343	343	77	119	

EXTENDED END PLATE



Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
305 x 102 x 33	222	274	54 (174)	550	550	153	171	In tension zone = 74 Dedicated to shear = 184
28	222	228 (274)	0 (173)	451	451	132	145	
25	219	124 (274)	0 (172)	343	343	105	119	

MINI HAUNCH - FOR ALL 305 x 102 UBs



Haunch Depth mm	F_{r1} kN	F_{r2} kN	F_{r3} kN	ΣF_r kN	Moment Capacity kNm	Min. thickness Haunch Flange mm	Bolt Shear Capacity kN per row
130	274	208	141	623	187	15	In tension zone = 74 Dedicated to shear = 184

WELDS

Serial Size	Tension Flange Extended mm	Tension Flange Flush mm	Web mm	Compression Flange in direct bearing mm
305 x 102 x 33	8FW	8FW	6FW	6FW
28	8FW	8FW	6FW	6FW
25	6FW	6FW	6FW	6FW

See: Notes - pages 142 - 143

Examples - pages 145 - 149

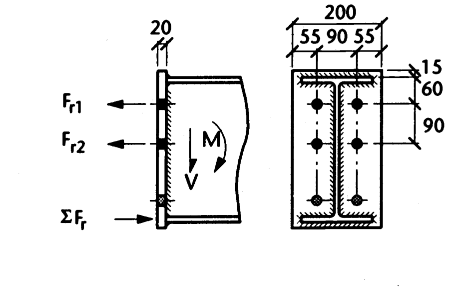
Column tables - pages 185 - 187

MOMENT CAPACITIES

254 x 146 UB
DESIGN GRADE 50
M20 8.8 BOLTS

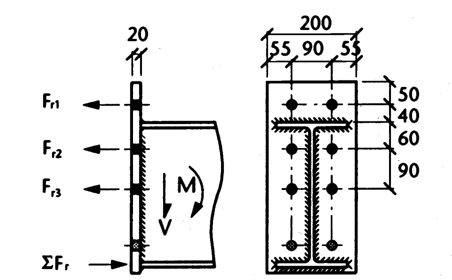
200 x 20 END PLATE - DESIGN GRADE 43

FLUSH END PLATE



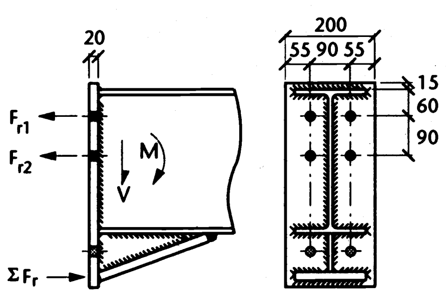
Serial Size	F_{r1} kN	F_{r2} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN
254 x 146 x 43	274	146	420	930	**68**	202	In tension zone = 74 Dedicated to shear = 184
37	274	145	419	793	**67**	172	
31	274	142	416	624	**65**	125	

EXTENDED END PLATE



Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
254 x 146 x 43	226	274	146	646	930	**134**	202	In tension zone = 74 Dedicated to shear = 184
37	222	274	145	641	793	**131**	172	
31	222	274	128 (142)	624	624	**127**	125	

MINI HAUNCH - FOR ALL 254 x 146 UBs



Haunch Depth mm	F_{r1} kN	F_{r2} kN	F_{r3} kN	ΣF_r kN	Moment Capacity kNm	Min. thickness Haunch Flange mm	Bolt Shear Capacity kN per row
85 *	274	183	93	550	**116**	10	In tension zone = 74 Dedicated to shear = 184
100	274	188	102	564	**127**	10	
130	274	196	118	589	**148**	10	
160	274	203	132	609	**169**	10	
190	274	209	143	626	**191**	10	
230 §	274	215	156	645	**221**	11	

** minimum recommended haunch depth.*

§ maximum recommended haunch depth.

WELDS

Serial Size	Tension Extd mm	Flange Flush mm	Web mm	Compression Flange in direct bearing mm
254 x 146 x 43	10FW	10FW	6FW	8FW
37	8FW	8FW	6FW	6FW
31	8FW	8FW	6FW	6FW

See: Notes - pages 142 - 143

Examples - pages 145 - 149

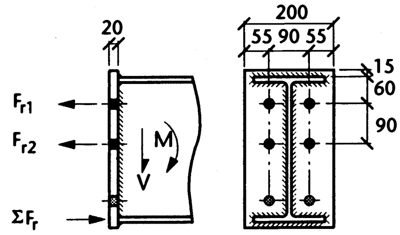
Column tables - pages 185 - 187

MOMENT CAPACITIES

254 x 102 UB
DESIGN GRADE 50

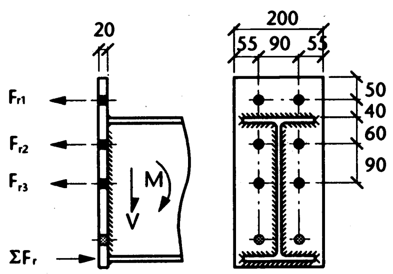
200 x 20 END PLATE - DESIGN GRADE 43
M20 8.8 BOLTS

FLUSH END PLATE



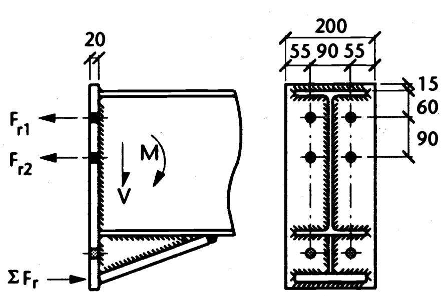
Serial Size	F_{r1} kN	F_{r2} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN
254 x 102 x 28	274	148	422	507	**69**	126	In tension zone = 74 Dedicated to shear = 184
25	274	146	420	425	**68**	109	
22	274	69 (144)	343	343	**59**	92.5	

EXTENDED END PLATE



Serial Size	F_{r1} kN	F_{r2} kN	F_{r3} kN	ΣF_r kN	Beam P_c kN	Moment Capacity kNm	Beam M_{cx} kNm	Bolt Shear Capacity kN per row
254 x 102 x 28	222	274	11 (148)	507	507	**120**	126	In tension zone = 74 Dedicated to shear = 184
25	219	206 (274)	0 (146)	425	425	**104**	109	
22	219	124 (274)	0 (144)	343	343	**87**	92.5	

MINI HAUNCH - FOR ALL 254 x 102 UBs



Haunch Depth mm	F_{r1} kN	F_{r2} kN	F_{r3} kN	ΣF_r kN	Moment Capacity kNm	Min. thickness Haunch Flange mm	Bolt Shear Capacity kN per row
85 *	274	184	95	553	**119**	13	In tension zone = 74 Dedicated to shear = 184
100	274	189	104	567	**129**	13	

WELDS

Serial Size	Tension Extd mm	Flange Flush mm	Web mm	Compression Flange in direct bearing mm
254 x 102 x 28	8FW	8FW	6FW	6FW
25	6FW	6FW	6FW	6FW
22	6FW	6FW	6FW	6FW

See: Notes - pages 142 - 143

Examples - pages 145 - 149

Column tables - pages 185 - 187

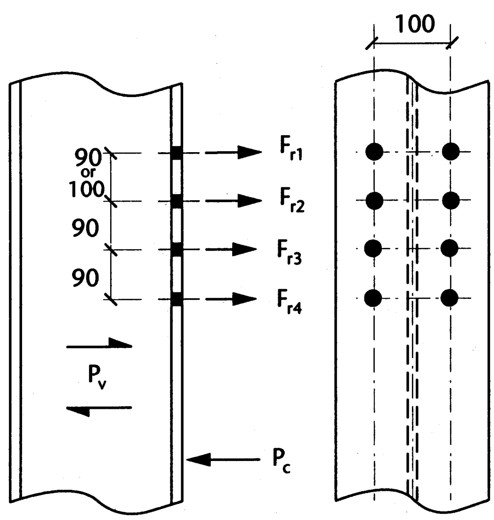
MOMENT CAPACITIES

UNSTIFFENED COLUMNS

DESIGN GRADES 43 & 50

for use with STANDARD END PLATES

M24 8.8 BOLTS



See:

Notes - *page 144*

Examples - *pages 145 - 149*

Beam Connection tables - *pages 150 - 154*

and pages 166 - 170

Serial Size	DESIGN GRADE 43						DESIGN GRADE 50					
	Bolt row forces				Compression capacity	Web panel shear capacity	Bolt row forces				Compression capacity	Web panel shear capacity
	F_{r1} kN	F_{r2} kN	F_{r3} kN	$F_{r4,5,6}$ kN	P_c kN	P_v kN	F_{r1} kN	F_{r2} kN	F_{r3} kN	$F_{r4,5,6}$ kN	P_c kN	P_v kN
356 x 368 x 202	396	396	396	396	1292	1000	396	396	396	396	1682	1302
177	396	396	383	318	1054	849	396	396	396	394	1372	1105
153	379	369	292	292	864	725	396	396	363	314	1125	944
129	333	322	270	270	688	605	368	353	285	285	896	787
305 x 305 x 283	396	396	396	396	2577	1503	396	396	396	396	3386	1974
240	396	396	396	396	2095	1288	396	396	396	396	2727	1677
198	396	396	396	396	1588	1037	396	396	396	396	2068	1350
158	396	396	396	330	1166	816	396	396	396	396	1518	1062
137	394	354	301	301	964	703	396	396	375	325	1255	915
118	349	315	278	278	784	595	388	343	296	296	1021	774
97	311	283	260	178	632	503	338	301	271	271	816	649
254 x 254 x 167	396	396	396	396	1533	882	396	396	396	396	1995	1149
132	396	396	349	326	1113	685	396	396	396	380	1449	892
107	375	304	287	287	845	551	396	355	307	307	1100	717
89	331	275	266	206	638	434	364	292	280	280	830	566
73	297	215	140	140	505	360	320	267	256	181	652	465
203 x 203 x 86	367	283	283	283	802	459	396	317	302	302	1044	598
71	322	263	263	195	591	353	353	276	276	276	770	460
60	291	163	134	134	515	322	312	258	189	173	664	415
52	246	102	102	102	424	272	290	159	132	132	547	351
46	189	78	78	78	372	245	244	101	101	101	480	316

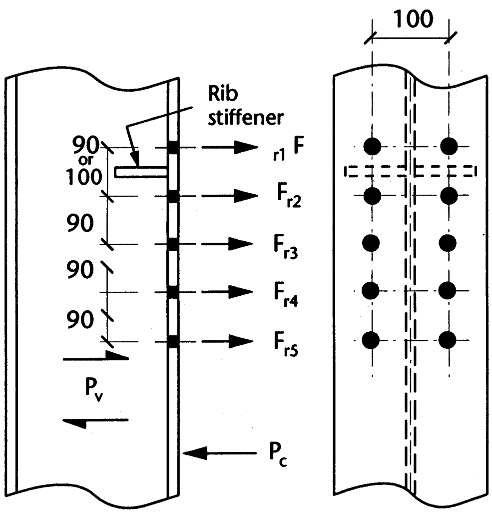
Mar. 97 Revision: *column for $F_{r4,5,6}$ added*

MOMENT CAPACITIES

RIB STIFFENED COLUMNS

DESIGN GRADES 43 & 50

for use with STANDARD END PLATES M24 8.8 BOLTS



See:

Notes - *page 144*

Examples - *pages 145 - 149*

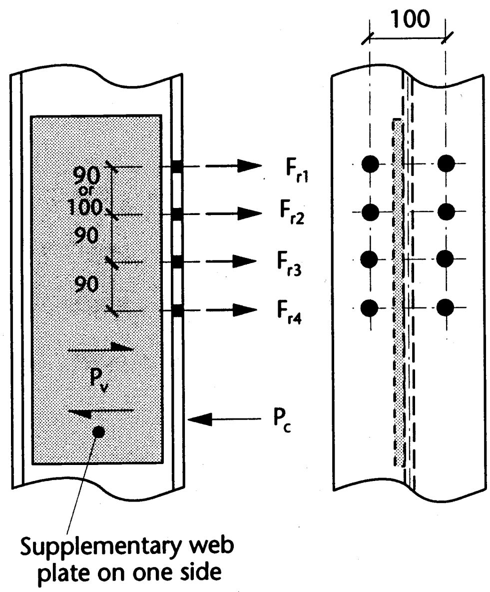
Beam Connection tables - *pages 150 - 154 and pages 166 - 170*

Serial Size	DESIGN GRADE 43							DESIGN GRADE 50						
	Bolt row forces					Compression capacity	Web panel shear capacity	Bolt row forces					Compression capacity	Web panel shear capacity
	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	$F_{r5,6}$ kN	P_c kN	P_v kN	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	$F_{r5,6}$ kN	P_c kN	P_v kN
356 x 368 x 202	396	396	396	396	396	1292	1000	396	396	396	396	396	1682	1302
177	396	396	396	383	318	1054	849	396	396	396	396	396	1372	1105
153	379	379	369	292	292	864	725	396	396	396	363	314	1125	944
129	333	333	322	270	270	688	605	368	368	353	285	285	896	787
305 x 305 x 283	396	396	396	396	396	2577	1503	396	396	396	396	396	3386	1974
240	396	396	396	396	396	2095	1288	396	396	396	396	396	2727	1677
198	396	396	396	396	396	1588	1037	396	396	396	396	396	2068	1350
158	396	396	396	396	330	1166	816	396	396	396	396	396	1518	1062
137	394	394	354	301	301	964	703	396	396	396	375	325	1255	915
118	349	349	315	278	278	784	595	388	388	343	296	296	1021	774
97	311	311	283	260	178	632	503	338	338	301	271	271	816	649
254 x 254 x 167	396	396	396	396	396	1533	882	396	396	396	396	396	1995	1149
132	396	396	396	349	326	1113	685	396	396	396	396	396	1449	892
107	375	375	304	287	287	845	551	396	396	355	307	307	1100	717
89	331	331	275	266	228	638	434	364	364	292	280	280	830	566
73	297	297	215	140	140	505	360	320	320	267	256	181	652	465
203 x 203 x 86	375	375	283	283	283	802	459	396	396	328	302	302	1044	598
71	331	331	263	263	195	591	353	364	364	276	276	276	770	460
60	297	297	185	134	134	515	322	320	320	258	218	173	664	415
52	270	270	102	102	102	424	272	297	297	183	132	132	547	351
46	209	209	78	78	78	372	245	270	270	101	101	101	480	316

Mar. 97 Revision: *column for* $F_{r5,6}$ *added*

MOMENT CAPACITIES
WEB STIFFENED COLUMN
SUPPLEMENTARY WEB PLATE

DESIGN GRADES 43 & 50
for use with STANDARD END PLATES
M24 8.8 BOLTS



See:

Notes - *page 144*

Examples - *pages 145 - 149*

Beam Connection tables - *pages 150 - 154*

and pages 166 - 170

Serial Size	DESIGN GRADE 43						DESIGN GRADE 50					
	Bolt row forces				Compression capacity	Web panel shear capacity	Bolt row forces				Compression capacity	Web panel shear capacity
	F_{r1} kN	F_{r2} kN	F_{r3} kN	$F_{r4,5,6}$ kN	P_c kN	P_v kN	F_{r1} kN	F_{r2} kN	F_{r3} kN	$F_{r4,5,6}$ kN	P_c kN	P_v kN
356 x 368 x 202	396	396	396	396	1938	1775	396	396	396	396	2523	2311
177	396	396	383	318	1580	1517	396	396	396	394	2058	1976
153	379	369	292	292	1296	1306	396	396	363	314	1687	1701
129	333	322	270	270	1032	1098	368	353	285	285	1344	1430
305 x 305 x 283	396	396	396	396	3866	2517	396	396	396	396	5078	3407
240	396	396	396	396	3142	2190	396	396	396	396	4091	2851
198	396	396	396	396	2383	1790	396	396	396	396	3102	2330
158	396	396	396	330	1749	1431	396	396	396	396	2277	1863
137	394	354	301	301	1447	1244	396	396	375	325	1883	1619
118	349	315	278	278	1176	1061	388	343	296	296	1532	1381
97	311	283	260	178	948	905	338	301	271	271	1224	1169
254 x 254 x 167	396	396	396	396	2299	1493	396	396	396	396	2993	1944
132	396	396	349	326	1669	1182	396	396	396	380	2173	1538
107	375	304	287	287	1267	965	396	355	307	307	1650	1256
89	331	275	266	206	957	769	364	292	280	280	1245	1001
73	297	215	140	140	758	645	320	267	256	181	979	832
203 x 203 x 86	367	283	283	283	1202	792	396	317	302	302	1565	1031
71	322	263	263	195	887	617	353	276	276	276	1155	803
60	291	163	134	134	772	568	312	258	189	173	996	734
52	246	102	102	102	636	484	290	159	132	132	821	625
46	189	78	78	78	558	438	244	101	101	101	720	566

Mar. 97 Revision: *column for $F_{r4,5,6}$ added*

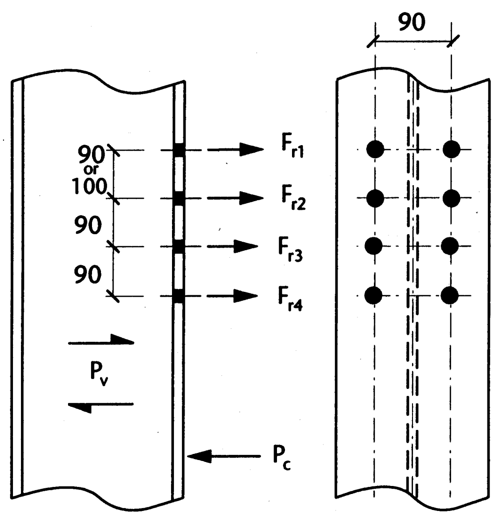
MOMENT CAPACITIES

UNSTIFFENED COLUMNS

DESIGN GRADES 43 & 50

for use with STANDARD END PLATES

M20 8.8 BOLTS



See:

Notes - *page 144*

Examples - *pages 145 - 149*

Beam Connection tables - *pages 155 - 165*

and pages 171 - 181

Serial Size	DESIGN GRADE 43						DESIGN GRADE 50					
	Bolt row forces				Compression capacity	Web panel shear capacity	Bolt row forces				Compression capacity	Web panel shear capacity
	F_{r1} kN	F_{r2} kN	F_{r3} kN	$F_{r4,5,6}$ kN	P_c kN	P_v kN	F_{r1} kN	F_{r2} kN	F_{r3} kN	$F_{r4,5,6}$ kN	P_c kN	P_v kN
356 x 368 x 202	274	274	274	274	1201	1000	274	274	274	274	1565	1302
177	274	274	274	274	975	849	274	274	274	274	1270	1105
153	274	274	274	239	796	725	274	274	274	274	1036	944
129	266	266	230	211	630	605	274	274	274	229	821	787
305 x 305 x 283	274	274	274	274	2437	1503	274	274	274	274	3202	1974
240	274	274	274	274	1971	1288	274	274	274	274	2565	1677
198	274	274	274	274	1485	1037	274	274	274	274	1933	1350
158	274	274	274	274	1081	816	274	274	274	274	1407	1062
137	274	274	274	274	890	703	274	274	274	274	1158	915
118	274	274	233	221	720	595	274	274	274	242	937	774
97	243	236	199	199	577	503	270	260	213	213	744	649
254 x 254 x 167	274	274	274	274	1429	882	274	274	274	274	1860	1149
132	274	274	274	274	1029	685	274	274	274	274	1339	892
107	274	274	257	231	774	551	274	274	274	274	1008	717
89	263	227	206	206	581	434	274	272	222	222	756	566
73	230	202	189	189	457	360	252	216	199	199	590	465
203 x 203 x 86	274	263	226	226	731	459	274	274	274	248	952	598
71	263	202	202	202	536	353	274	240	217	217	698	460
60	229	187	187	155	462	322	251	197	197	197	597	415
52	211	172	118	118	379	272	228	186	186	152	489	351
46	198	97	90	90	331	245	211	169	116	116	427	316

Mar. 97 Revision: *column for* $F_{r4,5,6}$ *added*

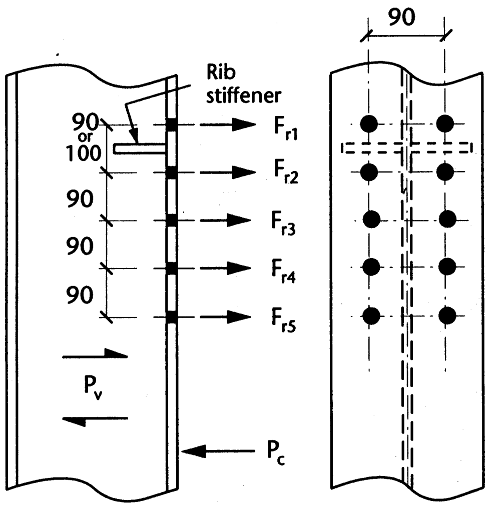
MOMENT CAPACITIES

RIB STIFFENED COLUMN

DESIGN GRADES 43 & 50

for use with STANDARD END PLATES

M20 8.8 BOLTS



See:

Notes - page 144

Examples - pages 145 - 149

Beam Connection tables - pages 155 - 165

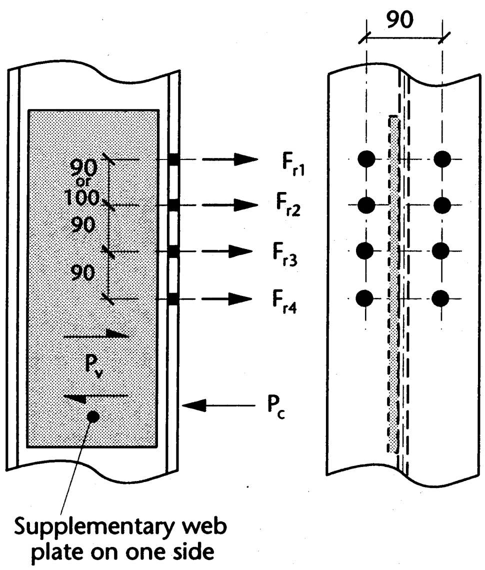
and pages 171 - 181

Serial Size	DESIGN GRADE 43							DESIGN GRADE 50						
	Bolt row forces					Compression capacity	Web panel shear capacity	Bolt row forces					Compression capacity	Web panel shear capacity
	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	$F_{r5,6}$ kN	P_c kN	P_v kN	F_{r1} kN	F_{r2} kN	F_{r3} kN	F_{r4} kN	$F_{r5,6}$ kN	P_c kN	P_v kN
356 x 368 x 202	274	274	274	274	274	1201	1000	274	274	274	274	274	1565	1302
177	274	274	274	274	274	975	849	274	274	274	274	274	1270	1105
153	274	274	274	274	274	796	725	274	274	274	274	274	1036	944
129	265	265	265	230	211	630	605	274	274	274	274	229	821	787
305 x 305 x 283	274	274	274	274	274	2437	1503	274	274	274	274	274	3202	1974
240	274	274	274	274	274	1971	1288	274	274	274	274	274	2565	1677
198	274	274	274	274	274	1485	1037	274	274	274	274	274	1933	1350
158	274	274	274	274	274	1081	816	274	274	274	274	274	1407	1062
137	274	274	274	274	274	890	703	274	274	274	274	274	1158	915
118	274	274	274	233	221	720	595	274	274	274	274	266	937	774
97	243	243	236	199	199	577	503	270	270	260	213	213	744	649
254 x 254 x 167	274	274	274	274	274	1429	882	274	274	274	274	274	1860	1149
132	274	274	274	274	274	1029	685	274	274	274	274	274	1339	892
107	274	274	274	257	231	774	551	274	274	274	274	274	1008	717
89	263	263	227	206	206	581	434	274	274	272	222	222	756	566
73	230	230	202	189	189	457	360	252	252	216	199	199	590	465
203 x 203 x 86	274	274	263	226	226	731	459	274	274	274	274	255	952	598
71	263	263	202	202	202	536	353	274	274	240	217	217	698	460
60	230	230	187	187	179	462	322	252	252	197	197	197	597	415
52	212	212	175	118	118	379	272	230	230	186	186	169	489	351
46	199	199	101	90	90	331	245	212	212	174	116	116	427	316

Mar. 97 Revision: column for $F_{r5,6}$ added

MOMENT CAPACITIES
WEB STIFFENED COLUMN
SUPPLEMENTARY WEB PLATE

DESIGN GRADES 43 & 50
for use with STANDARD END PLATES
M20 8.8 BOLTS



See:

Notes - *page 144*

Examples - *pages 145 - 149*

Beam Connection tables - *pages 155 - 165*

and pages 171 - 181

Serial Size	DESIGN GRADE 43						DESIGN GRADE 50					
	Bolt row forces				Compression capacity	Web panel shear capacity	Bolt row forces				Compression capacity	Web panel shear capacity
	F_{r1} kN	F_{r2} kN	F_{r3} kN	$F_{r4,5,6}$ kN	P_c kN	P_v kN	F_{r1} kN	F_{r2} kN	F_{r3} kN	$F_{r4,5,6}$ kN	P_c kN	P_v kN
356 x 368 x 202	274	274	274	274	1802	1775	274	274	274	274	2346	2311
177	274	274	274	274	1463	1517	274	274	274	274	1904	1976
153	274	274	274	239	1194	1306	274	274	274	274	1554	1701
129	266	266	230	211	945	1098	274	274	274	229	1231	1430
305 x 305 x 283	274	274	274	274	3656	2517	274	274	274	274	4803	3307
240	274	274	274	274	2956	2190	274	274	274	274	3848	2851
198	274	274	274	274	2227	1790	274	274	274	274	2899	2330
158	274	274	274	274	1621	1431	274	274	274	274	2111	1863
137	274	274	274	274	1335	1244	274	274	274	274	1738	1619
118	274	274	233	221	1080	1061	274	274	274	242	1406	1381
97	243	236	199	199	865	906	270	260	213	213	1117	1169
254 x 254 x 167	274	274	274	274	2143	1493	274	274	274	274	2790	1944
132	274	274	274	274	1543	1182	274	274	274	274	2009	1538
107	274	274	257	231	1162	965	274	274	274	274	1512	1256
89	263	227	206	206	871	769	274	272	222	222	1135	1001
73	230	202	189	189	686	645	252	216	199	199	885	832
203 x 203 x 86	274	263	226	226	1097	792	274	274	274	248	1428	1031
71	263	202	202	202	804	617	274	240	217	217	1046	803
60	229	187	187	155	694	568	251	197	197	197	895	734
52	211	172	118	118	569	484	228	186	186	152	734	625
46	198	97	90	90	496	438	211	169	116	116	641	566

Mar. 97 Revision: *column for $F_{r4,5,6}$ added*

Portal Frame Eaves Haunch and Apex Connections

NOTES ON USE OF THE CAPACITY TABLES

EAVES CONNECTIONS

Connection Moment Capacity

Moment capacities of haunched connections shown are calculated using the method of Section 2.8. The web of the haunch has been utilised to carry compression (STEP 2B), where necessary. The moment capacities stated should be compared to the moment which results from the frame analysis.

The sign convention is as illustrated on each diagram.

The moment capacity shown in the reversed condition includes the effect of the truncated column top in the column web crushing and buckling checks, which are generally critical. A compression stiffener at the column top will produce a significant increase in reverse moment capacity.

Connection Shear Capacity

Connection shear capacity is the vertical shear available from the bolt rows shown. Increased shear capacity will generally be available in the reversal load case, since a greater number of bolts will be dedicated to the transfer of shear alone.

Maximum Axial Force in Rafter

If the axial force in the rafter exceeds the limit indicated (compression for positive moments and tension for negative moments), the moment capacities quoted are no longer valid and the connection capacity must be re-calculated.

Haunch Cutting

The haunch may be cut from the section size shown. If fabricated from plate, the flange should be at least equal in area to the haunch flange and the web plate should be at least as thick as the web of the haunch section size shown.

Material Grade

The haunch, end plate and stiffener material has been taken as design grade 43.

Weld Sizes

Weld sizes shown have been calculated in accordance with STEP 7. They should not be changed without re-calculating the connection moment capacity. The weld for the haunch flange to end plate has been sized as a tension weld, based on the reversal moment, not the positive (gravity) load case. For the positive (gravity) load case, it has been assumed that the haunch cutting is fitted to the end plate. If a bearing fit is not provided with either a haunch fabricated from plate or from a section cutting, full strength welds should be specified.

Overall Haunch Depth

The overall depth shown on each diagram is that measured from where the top of the rafter meets the end plate to where the underside of the haunch meets the end plate. Moment capacities have been calculated using this minimum dimension, and are conservative where greater overall depths are used.

Stiffeners

Rib, Morris and compression stiffeners have been designed in accordance with STEP 6.

APEX CONNECTIONS

Connection Moment Capacity

Connections are sized to ensure that the moment capacity in the +ve direction, calculated using the method of Section 2.8, is greater or equal to M_{cx}. The web of the rafter has been utilised to carry compression (STEP 2B), where necessary.

Maximum Axial Force in Rafter

If the axial force in the rafter exceeds the limit indicated (compression for positive moments and tension for negative moments), the moment capacities quoted are no longer valid and the connection capacity must be re-calculated.

Material Grade

End plate and haunch material has been taken as design grade 43.

Weld Sizes

Weld sizes shown have been calculated in accordance with STEP 7. They should not be changed without re-calculating the connection moment capacity.

The weld for the haunch flange to end plate has been sized to act as a tension weld for positive moments, and as a compression weld for negative moments.

Worked Example Using the Capacity Tables

A portal frame analysis results in the following:

	Rafter	*533 x 210 x 82*	
	Column	*686 x 254 x 125*	
	Connection forces:	*Moment*	*1085kNm,*
		Axial (Rafter)	*132kN (compression),*
		Vertical Shear	*210kN*

Provide a suitable eaves haunch connection

Page 191 shows that a flush end plate haunch connection when provided with compression stiffeners and a pair of rib stiffeners between the top bolt rows will be satisfactory.

Axial (Rafter)	*132kN*	<	*799kN*	∴ *moment capacity is valid*
Moment	*1085kNm*	<	*1094kNm*	∴ *satisfactory*
Vertical Shear	*210kN*	<	*1320kN*	∴ *satisfactory*

(Alternatively, an extended end plate haunch connection provided with compression stiffeners only is also satisfactory.)

****

Axial Forces

The following demonstrates the effect of the axial force in the above example; axial compression reduces an applied positive moment (STEP 4):

$$M_m = 1085 - 132 \times \left(1.075 - \frac{0.528}{2} - \frac{0.015}{2}\right) = 979\text{kNm}$$

Connection design software shows the bolt forces are reduced as follows:

Row No.	*Original bolt row forces (kN)*	*Reduced bolt row forces (kN)*	*Lever arm (m)*	*Moment (kNm)*
1	317	317	1.008	320
2	317	317	0.918	291
3	295	295	0.828	244
4	275	199	0.738	147
5	56	0	0.648	0
Totals	*1260kN*	*1128kN*		*1002kNm*

1002kNm > 979kNm ∴ *satisfactory*

$$P_c = 1128\text{kN} + 132\text{kN} = 1260\text{kN}$$

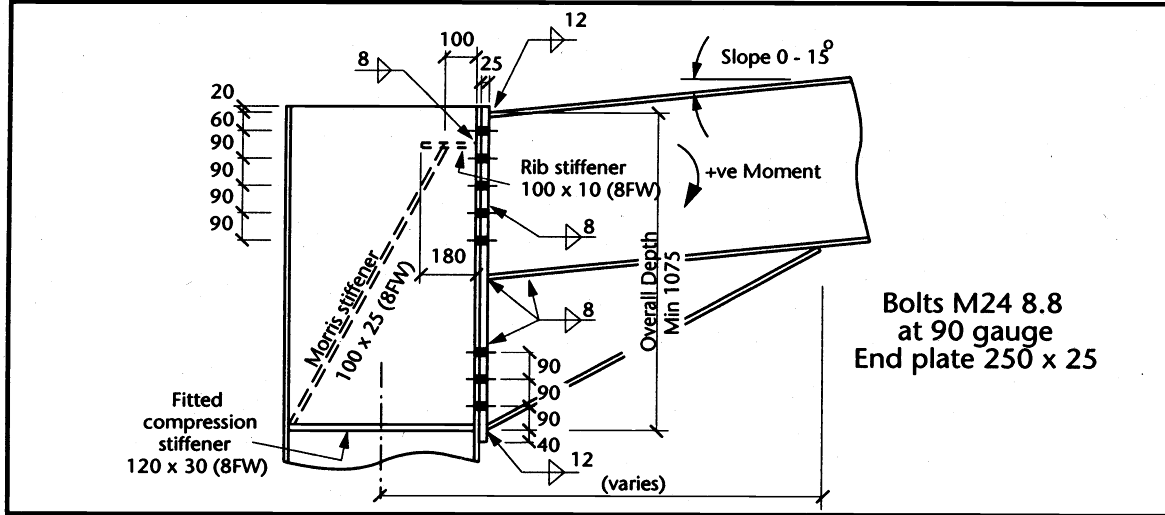
= *value of* P_c *for this configuration*

Note: *In this example the column web panel shear resistance (1260kN) limits the development of the bolt row forces. Axial compression further reduces the sum of the bolt row forces (1260 – 132 = 1128kN). Bolt row forces are therefore reduced in accordance with STEP 4, hence the connection moment capacity is reduced. However, the axial compression reduces the applied moment (M_m =979 kNm) and at this level of axial load,the reduced connection capacity exceeds the modified moment.*

Inspection of the above table indicates that at an axial load of 800kN, the modified moment exceeds the reduced connection capacity.

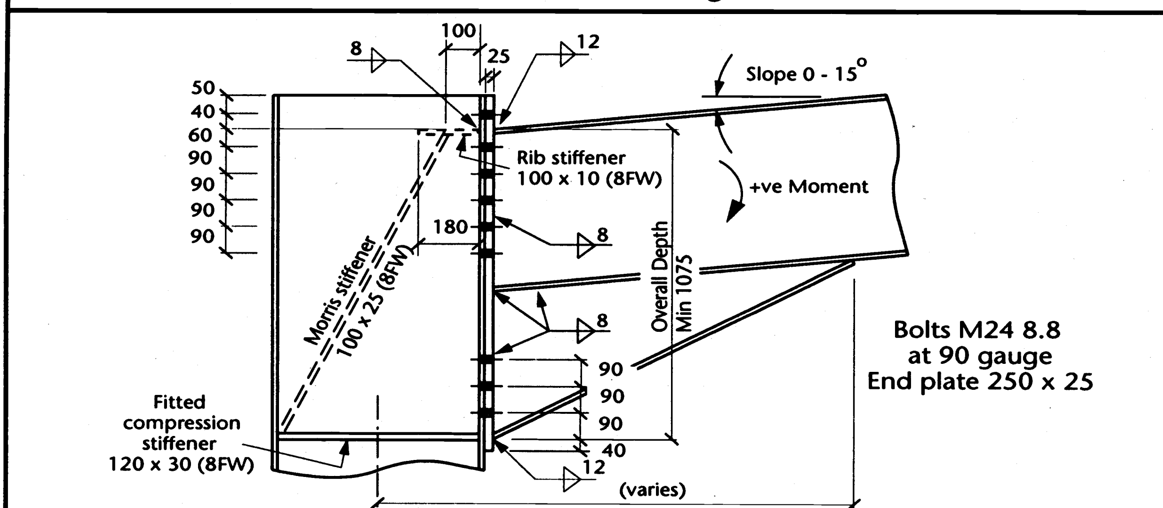
PORTAL FRAME EAVES HAUNCH CONNECTIONS

533 x 210 x 92 UB Rafters to 762 x 267 UB and 686 x 254 UB Columns
Flush End Plate - All Design Grade 43 Steel



Column		Connection Moment Capacity (kNm)			Connection Shear Capacity (kN)	Maximum Axial Force in Rafter (kN) (compⁿ + ve)	Haunch Cutting
		Compression Stiffener only	Compression Stiffener & Rib Stiffener	Compression Stiffener & Morris Stiffener			
762 x 267 x 173 UB	+ ve	1319	1320	1320	1320	+ 1199	610 x 229 x 101 UB
(Mcx = 1640kNm)	- ve		- 587			- 357	
762 x 267 x 147 UB	+ ve	1241	1259	1259	1320	+ 1241	
(Mcx = 1370kNm)	- ve		- 452			- 463	
686 x 254 x 140 UB	+ ve	1159	1172	1281	1320 .	+ 782	
(Mcx = 1210kNm)	- ve		- 447			- 502	
686 x 254 x 125 UB	+ ve	1080	1094	1217	1320	+ 750	
(Mcx = 1060kNm)	- ve		- 386			- 481	

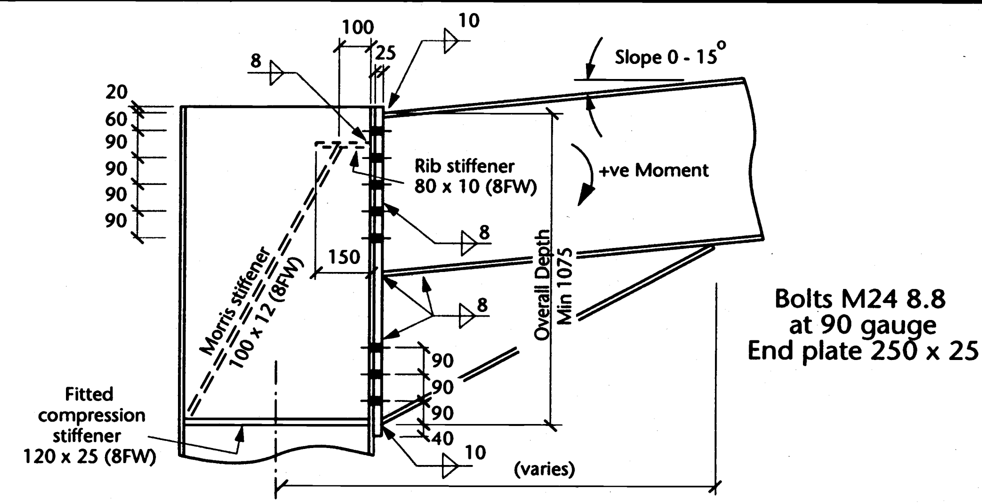
533 x 210 x 92 UB Rafters to 762 x 267 UB and 686 x 254 UB Columns
Extended End Plate - All Design Grade 43 Steel



Column		Connection Moment Capacity (kNm)			Connection Shear Capacity (kN)	Maximum Axial Force in Rafter (kN) (compⁿ + ve)	Haunch Cutting
		Compression Stiffener only	Compression Stiffener & Rib Stiffener	Compression Stiffener & Morris Stiffener			
762 x 267 x 173 UB	+ ve	1548	1560	1661	1425	+ 684	610 x 229 x 101 UB
(Mcx = 1640kNm)	- ve		- 671			- 261	
762 x 267 x 147 UB	+ ve	1401	1414	1554	1425	+ 651	
(Mcx = 1370kNm)	- ve		- 518			- 389	
686 x 254 x 140 UB	+ ve	1287	1299	1600	1425	+ 85	
(Mcx = 1210kNm)	- ve		- 518			- 423	
686 x 254 x 125 UB	+ ve	1198	1207	1486	1425	+ 201	
(Mcx = 1060kNm)	- ve		- 446			- 413	

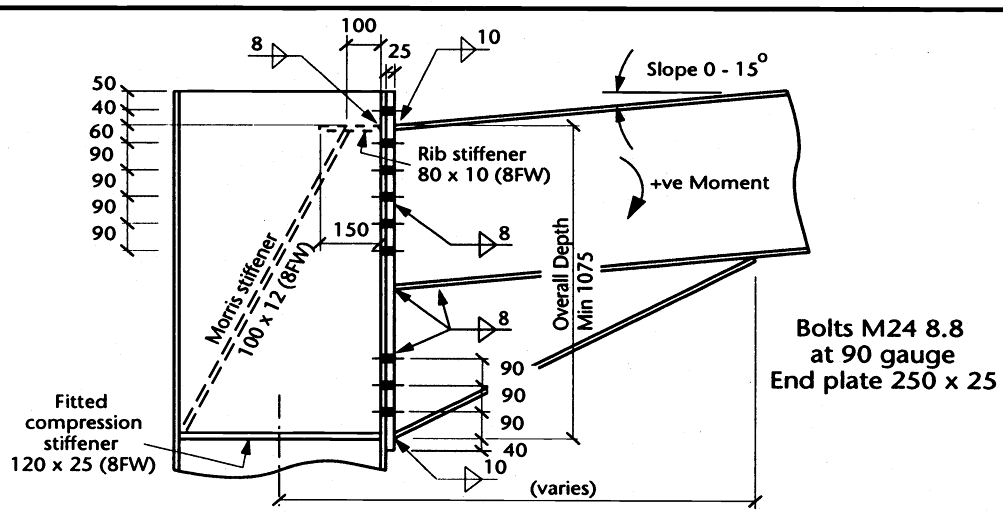
PORTAL FRAME EAVES HAUNCH CONNECTIONS

533 x 210 x 82 UB Rafters to 762 x 267 UB and 686 x 254 UB Columns
Flush End Plate - All Design Grade 43 Steel



Column		Connection Moment Capacity (kNm) Compression Stiffener only	Compression Stiffener & Rib Stiffener	Compression Stiffener & Morris Stiffener	Connection Shear Capacity (kN)	Maximum Axial Force in Rafter (kN) (compⁿ + ve)	Haunch Cutting
762 x 267 x 147 UB	+ ve	1211	1216	1216	1320	+ 1160	610 x 229 x 101 UB
(M_{cx} = 1370kNm)	- ve		- 449			- 221	
686 x 254 x 152 UB	+ ve	1226	1229	1265	1320	+ 988	
(M_{cx} = 1330kNm)	- ve		- 518			- 148	
686 x 254 x 140 UB	+ ve	1157	1164	1236	1320	+ 865	
(M_{cx} = 1210kNm)	- ve		- 444			- 228	
686 x 254 x 125 UB	+ ve	1080	1094	1196	1320	+ 799	
(M_{cx} = 1060kNm)	- ve		- 382			- 294	

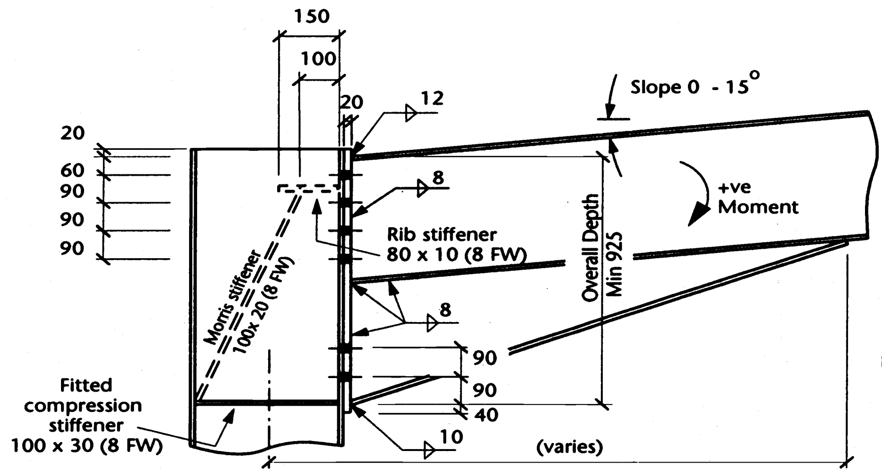
533 x 210 x 82 UB Rafters to 762 x 267 UB and 686 x 254 UB Columns
Extended End Plate - All Design Grade 43 Steel



Column		Connection Moment Capacity (kNm) Compression Stiffener only	Compression Stiffener & Rib Stiffener	Compression Stiffener & Morris Stiffener	Connection Shear Capacity (kN)	Maximum Axial Force in Rafter (kN) (compⁿ + ve)	Haunch Cutting
762 x 267 x 147 UB	+ ve	1383	1414	1415	1425	+ 664	610 x 229 x 101 UB
(M_{cx} = 1370kNm)	- ve		- 514			- 147	
686 x 254 x 152 UB	+ ve	1368	1375	1495	1425	+ 161	
(M_{cx} = 1330kNm)	- ve		- 599			- 56	
686 x 254 x 140 UB	+ ve	1288	1299	1454	1425	+ 94	
(M_{cx} = 1210kNm)	- ve		- 514			- 149	
686 x 254 x 125 UB	+ ve	1198	1207	1348	1425	+ 220	
(M_{cx} = 1060kNm)	- ve		- 443			- 225	

PORTAL FRAME EAVES HAUNCH CONNECTIONS

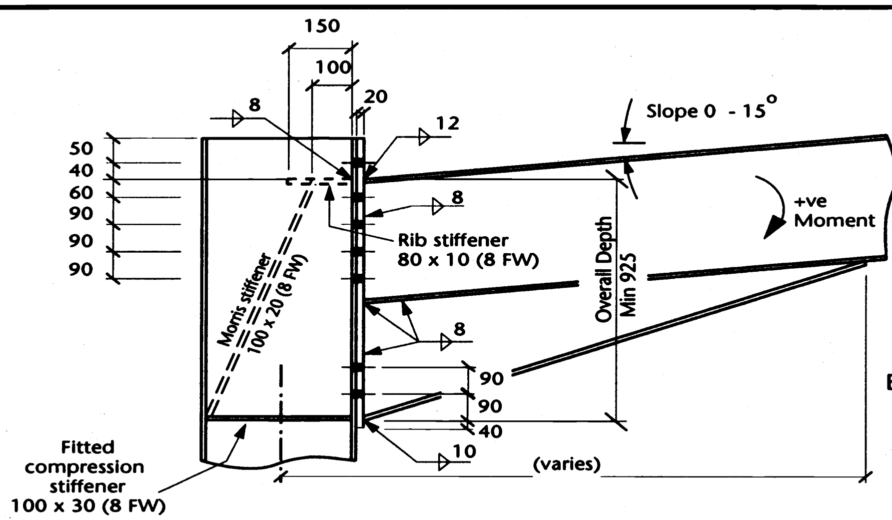
457 x 191 x 74 UB Rafters to 686 x 254 UB and 610 x 229 UB Columns
Flush End Plate - All Design Grade 43 Steel



Bolts M24 8.8
at 90 gauge
End plate 200 x 20

Column		Connection Moment Capacity (kNm): Compression Stiffener only	Compression Stiffener & Rib Stiffener	Compression Stiffener & Morris Stiffener	Connection Shear Capacity (kN)	Maximum Axial Force in Rafter (kN) (compⁿ + ve)	Haunch Cutting
686 x 254 x 140 UB	+ ve	864	864	864	950	+ 880	533 x 210 x 82 UB
(Mcx = 1210kNm)	- ve		– 373			– 184	
686 x 254 x 125 UB	+ ve	834	841	841	950	+ 851	
(Mcx = 1060kNm)	- ve		– 320			– 216	
610 x 229 x 113 UB	+ ve	797	803	851	950	+ 665	
(Mcx = 871kNm)	- ve		– 315			– 230	
610 x 229 x 101 UB	+ ve	718	776	796	950	+ 662	
(Mcx = 794kNm)	- ve		– 278			– 234	

457 x 191 x 74 UB Rafters to 686 x 254 UB and 610 x 229 UB Columns
Extended End Plate - All Design Grade 43 Steel

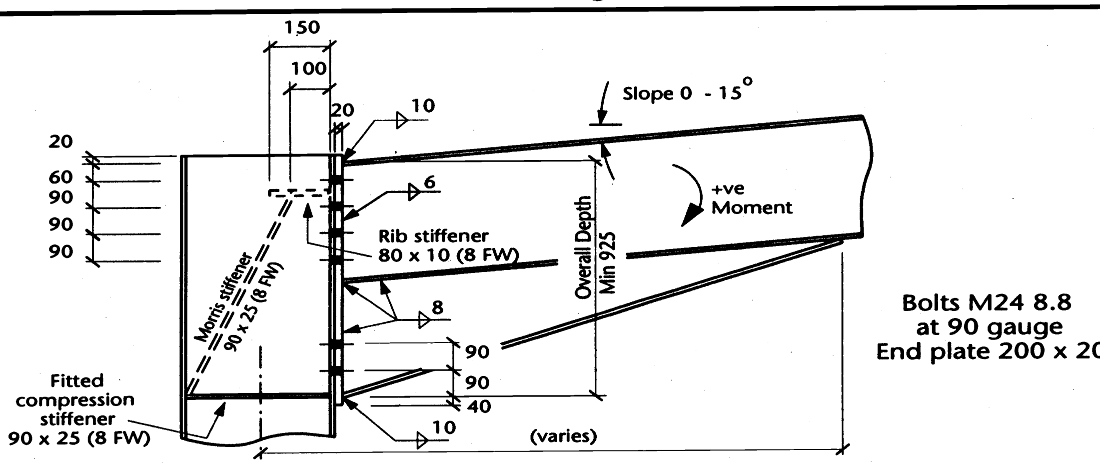


Bolts M24 8.8
at 90 gauge
End plate 200 x 20

Column		Connection Moment Capacity (kNm): Compression Stiffener only	Compression Stiffener & Rib Stiffener	Compression Stiffener & Morris Stiffener	Connection Shear Capacity (kN)	Maximum Axial Force in Rafter (kN) (compⁿ + ve)	Haunch Cutting
686 x 254 x 140 UB	+ ve	1049	1049	1110	1056	+ 591	533 x 210 x 82 UB
(Mcx = 1210kNm)	- ve		– 431			–105	
686 x 254 x 125 UB	+ ve	992	996	1082	1056	+ 493	
(Mcx = 1060kNm)	- ve		– 371			– 147	
610 x 229 x 113 UB	+ ve	894	897	1089	1056	+ 191	
(Mcx = 871kNm)	- ve		– 369			–157	
610 x 229 x 101 UB	+ ve	858	872	984	1056	+ 207	
(Mcx = 794kNm)	- ve		– 325			–170	

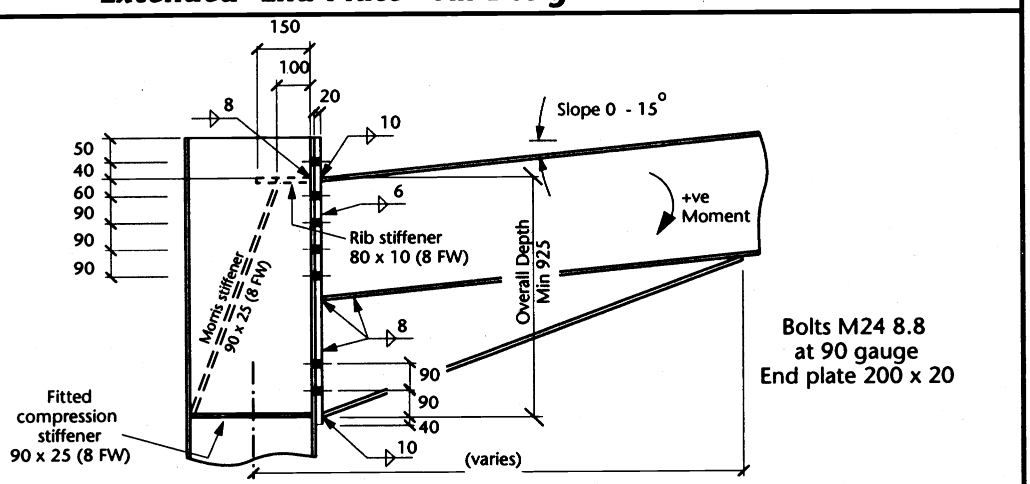
PORTAL FRAME EAVES HAUNCH CONNECTIONS

457 x 191 x 67 UB Rafters to 686 x 254 UB, 610 x 229 UB and 533 x 210 UB Columns Flush End Plate - All Design Grade 43 Steel



Column		Connection Moment Capacity (kNm)			Connection Shear Capacity (kN)	Maximum Axial Force in Rafter (kN) (compⁿ + ve)	Haunch Cutting
		Compression Stiffener only	Compression Stiffener & Rib Stiffener	Compression Stiffener & Morris Stiffener			
686 x 254 x 125 UB	+ ve	806	809	809	950	+ 790	533 x 210 x 82 UB
(Mcx = 1060kNm)	- ve		- 319			- 219	
610 x 229 x 113 UB	+ ve	797	801	820	950	+ 711	
(Mcx = 871kNm)	- ve		- 313			- 234	
610 x 229 x 101 UB	+ ve	719	776	796	950	+ 670	
(Mcx = 794kNm)	- ve		- 275			- 238	
533 x 210 x 92 UB	+ ve	684	692	806	950	+ 330	
(Mcx = 651kNm)	- ve		- 279			- 242	

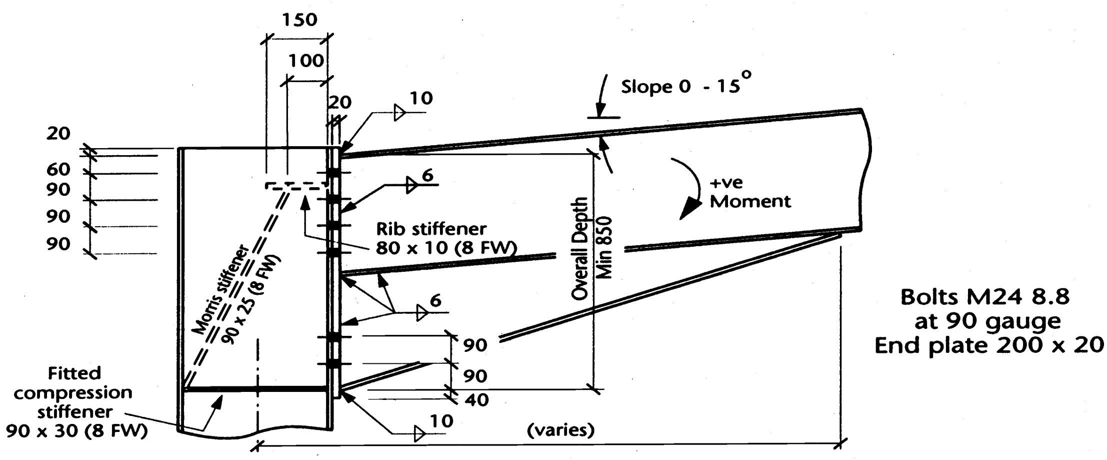
457 x 191 x 67 UB Rafters to 686 x 254 UB, 610 x 229 UB and 533 x 210 UB Columns Extended End Plate - All Design Grade 43 Steel



Column		Connection Moment Capacity (kNm)			Connection Shear Capacity (kN)	Maximum Axial Force in Rafter (kN) (compⁿ + ve)	Haunch Cutting
		Compression Stiffener only	Compression Stiffener & Rib Stiffener	Compression Stiffener & Morris Stiffener			
686 x 254 x 125 UB	+ ve	991	994	1057	1056	+ 511	533 x 210 x 82 UB
(Mcx = 1060kNm)	- ve		- 369			- 151	
610 x 229 x 113 UB	+ ve	894	896	1065	1056	+ 199	
(Mcx = 871kNm)	- ve		- 366			- 161	
610 x 229 x 101 UB	+ ve	857	871	981	1056	+ 215	
(Mcx = 794kNm)	- ve		- 323			- 173	
533 x 210 x 92 UB	+ ve	766	768	1023	1056	+ 70	
(Mcx = 651kNm)	- ve		- 332			- 170	

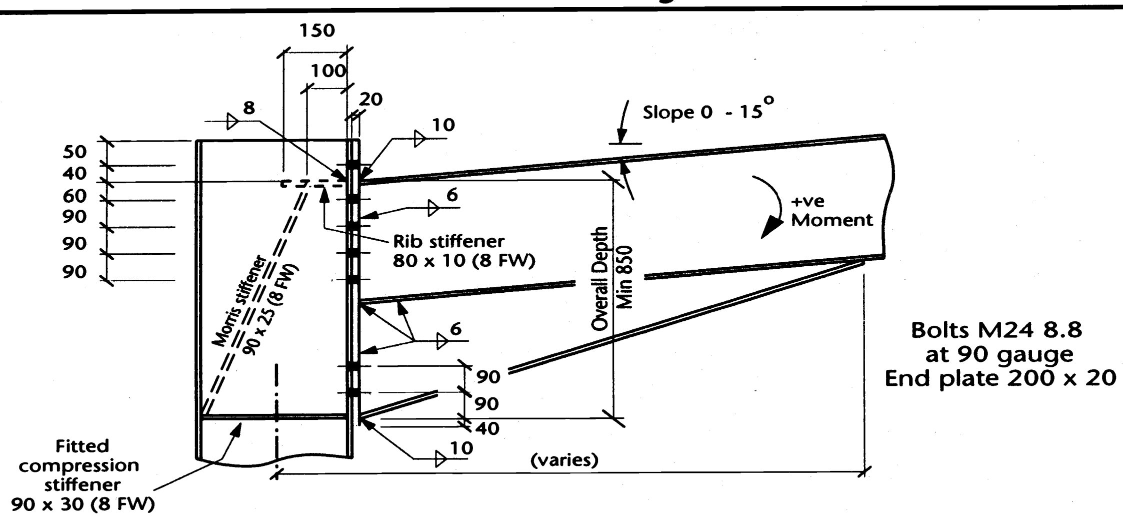
PORTAL FRAME EAVES HAUNCH CONNECTIONS

457 x 152 x 60 UB Rafters to 610 x 229 UB UB and 533 x 210 UB Columns
Flush End Plate - All Design Grade 43 Steel



Column		Connection Moment Capacity (kNm)			Connection	Maximum Axial	Haunch Cutting
		Compression Stiffener only	Compression Stiffener & Rib Stiffener	Compression Stiffener & Morris Stiffener	Shear Capacity (kN)	Force in Rafter (kN) (compⁿ + ve)	
610 x 229 x 113 UB	+ ve	699	702	702	950	+ 723	457 x 191 x 67 UB
(Mcx = 871kNm)	- ve	− 284				− 191	
610 x 229 x 101 UB	+ ve	641	683	683	950	+ 668	
(Mcx = 794kNm)	- ve	− 250				− 237	
533 x 210 x 92 UB	+ ve	615	620	690	950	+ 367	
(Mcx = 651kNm)	- ve	− 254				− 234	
533 x 210 x 82 UB	+ ve	501	574	597	950	+ 418	
(Mcx = 566kNm)	- ve	− 219				− 163	

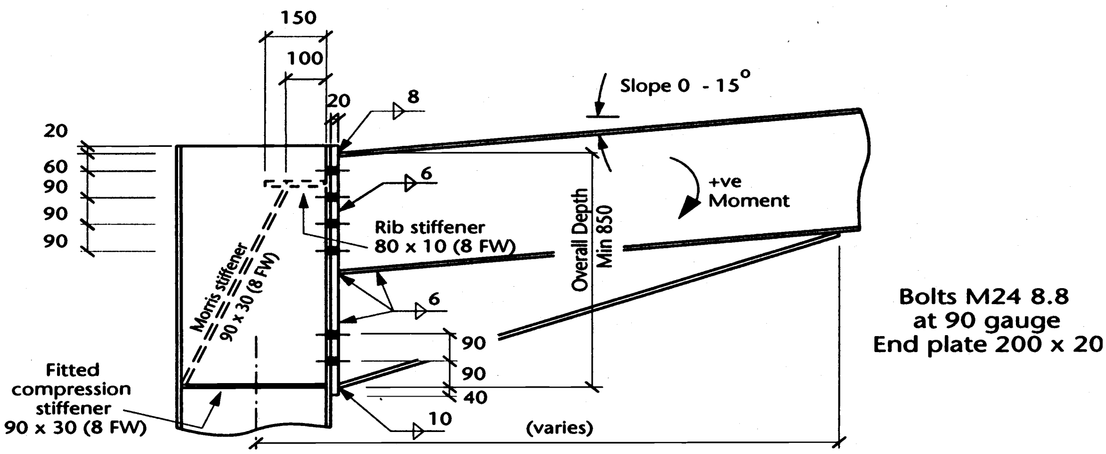
457 x 152 x 60 UB Rafters to 610 x 229 UB UB and 533 x 210 UB Columns
Extended End Plate - All Design Grade 43 Steel



Column		Connection Moment Capacity (kNm)			Connection	Maximum Axial	Haunch Cutting
		Compression Stiffener only	Compression Stiffener & Rib Stiffener	Compression Stiffener & Morris Stiffener	Shear Capacity (kN)	Force in Rafter (kN) (compⁿ + ve)	
610 x 229 x 113 UB	+ ve	803	805	914	1056	+ 197	457 x 191 x 67 UB
(Mcx = 871kNm)	- ve	− 332				− 119	
610 x 229 x 101 UB	+ ve	770	783	866	1056	+ 213	
(Mcx = 794kNm)	- ve	− 293				− 173	
533 x 210 x 92 UB	+ ve	699	701	899	1056	+ 100	
(Mcx = 651kNm)	- ve	− 301				− 163	
533 x 210 x 82 UB	+ ve	624	652	744	1056	+ 112	
(Mcx = 566kNm)	- ve	− 260				− 101	

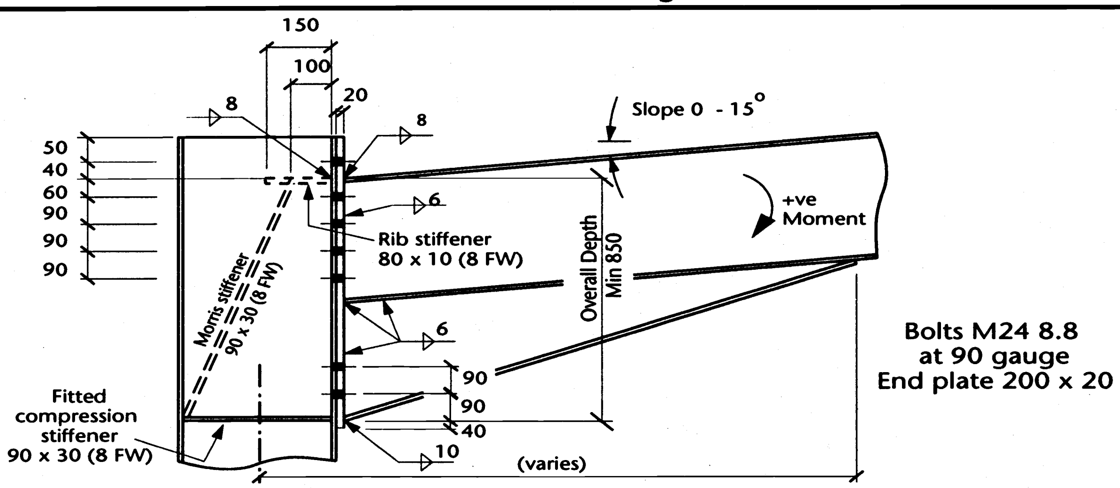
PORTAL FRAME EAVES HAUNCH CONNECTIONS

457 x 152 x 52 UB Rafters to 610 x 229 UB, 533 x 210 UB and 457 x 191 UB Columns
Flush End Plate - All Design Grade 43 Steel



Column		Connection Moment Capacity (kNm)			Connection	Maximum Axial	Haunch Cutting
		Compression Stiffener only	Compression Stiffener & Rib Stiffener	Compression Stiffener & Morris Stiffener	Shear Capacity (kN)	Force in Rafter (kN) (compⁿ + ve)	
610 x 229 x 101 UB	+ ve	641	664	664	950	+ 678	457 x 191 x 67 UB
(M_{cx} = 794kNm)	- ve		– 249			– 240	
533 x 210 x 92 UB	+ ve	614	618	672	950	+ 411	
(M_{cx} = 651kNm)	- ve		– 253			– 237	
533 x 210 x 82 UB	+ ve	501	574	597	950	+ 432	
(M_{cx} = 566kNm)	- ve		– 218			– 165	
457 x 191 x 74 UB	+ ve	485	495	654	950	+ 101	
(M_{cx} = 456kNm)	- ve		– 214			– 226	

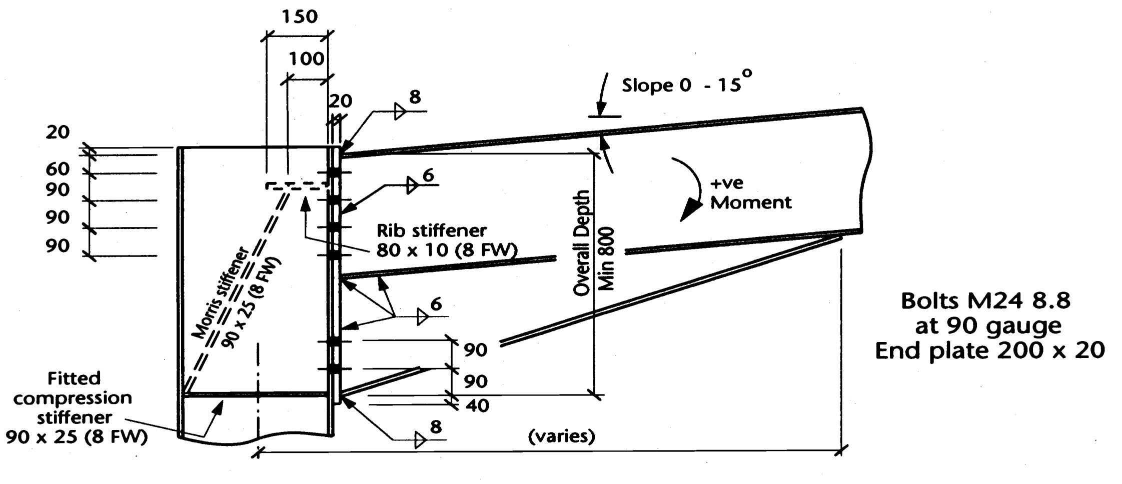
457 x 152 x 52 UB Rafters to 610 x 229 UB, 533 x 210 UB and 457 x 191 UB Columns
Extended End Plate - All Design Grade 43 Steel



Column		Connection Moment Capacity (kNm)			Connection	Maximum Axial	Haunch Cutting
		Compression Stiffener only	Compression Stiffener & Rib Stiffener	Compression Stiffener & Morris Stiffener	Shear Capacity (kN)	Force in Rafter (kN) (compⁿ + ve)	
610 x 229 x 101 UB	+ ve	770	783	866	1056	+ 220	457 x 191 x 67 UB
(M_{cx} = 794kNm)	- ve		– 292			– 175	
533 x 210 x 92 UB	+ ve	700	702	888	1056	+ 70	
(M_{cx} = 651kNm)	- ve		– 301			– 165	
533 x 210 x 82 UB	+ ve	625	653	744	1056	+ 116	
(M_{cx} = 566kNm)	- ve		– 259			– 103	
457 x 191 x 74 UB	+ ve	548	554	812	1056	+ 70	
(M_{cx} = 456kNm)	- ve		– 260			– 158	

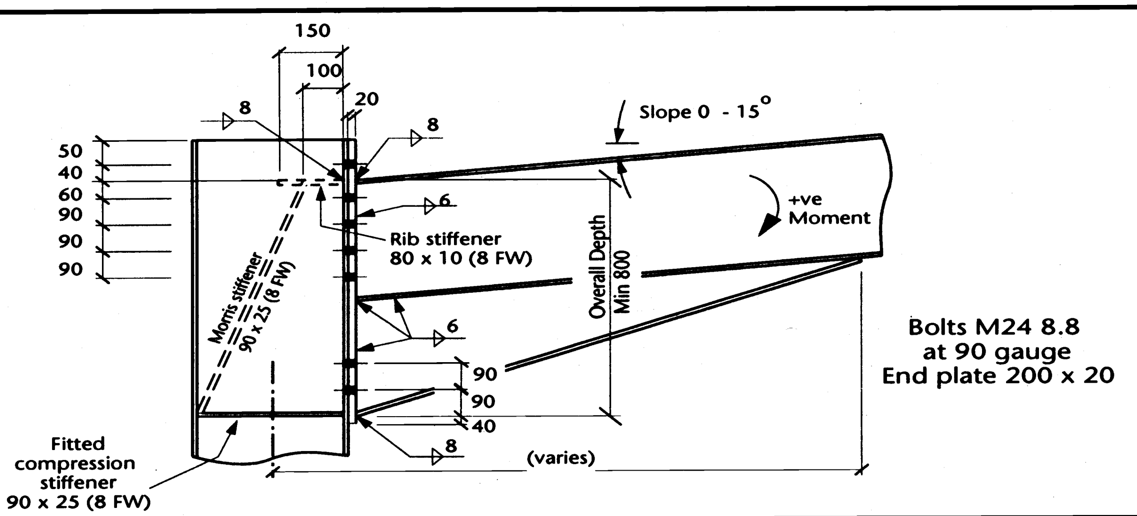
PORTAL FRAME EAVES HAUNCH CONNECTIONS

406 x 140 x 46 UB Rafters to 533 x 210 UB and 457 x 191 UB Columns
Flush End Plate - All Design Grade 43 Steel



Column		Connection Moment Capacity (kNm): Compression Stiffener only	Compression Stiffener & Rib Stiffener	Compression Stiffener & Morris Stiffener	Connection Shear Capacity (kN)	Maximum Axial Force in Rafter (kN) (compⁿ + ve)	Haunch Cutting
533 x 210 x 92 UB	+ ve	543	546	563	950	+ 631	457 x 152 x 52 UB
(Mcx = 651kNm)	- ve		– 235			– 138	
533 x 210 x 82 UB	+ ve	457	516	532	950	+ 566	
(Mcx = 566kNm)	- ve		– 202			– 167	
457 x 191 x 74 UB	+ ve	447	458	555	950	+ 157	
(Mcx = 456kNm)	- ve		– 198			– 193	
457 x 191 x 67 UB	+ ve	395	422	496	950	+ 138	
(Mcx = 405kNm)	- ve		– 167			– 152	

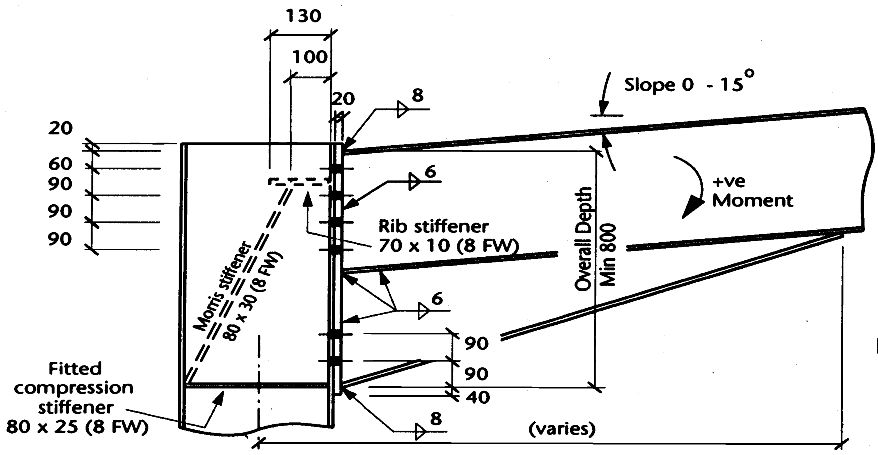
406 x 140 x 46 UB Rafters to 533 x 210 UB and 457 x 191 UB Columns
Extended End Plate - All Design Grade 43 Steel



Column		Connection Moment Capacity (kNm): Compression Stiffener only	Compression Stiffener & Rib Stiffener	Compression Stiffener & Morris Stiffener	Connection Shear Capacity (kN)	Maximum Axial Force in Rafter (kN) (compⁿ + ve)	Haunch Cutting
533 x 210 x 92 UB	+ ve	631	633	735	1056	+ 70	457 x 152 x 52 UB
(Mcx = 651kNm)	- ve		– 279			– 66	
533 x 210 x 82 UB	+ ve	566	594	656	1056	+ 176	
(Mcx = 566kNm)	- ve		– 241			– 104	
457 x 191 x 74 UB	+ ve	509	515	701	1056	+ 70	
(Mcx = 456kNm)	- ve		– 241			– 125	
457 x 191 x 67 UB	+ ve	460	483	615	1056	+ 70	
(Mcx = 405kNm)	- ve		– 204			– 95	

PORTAL FRAME EAVES HAUNCH CONNECTIONS

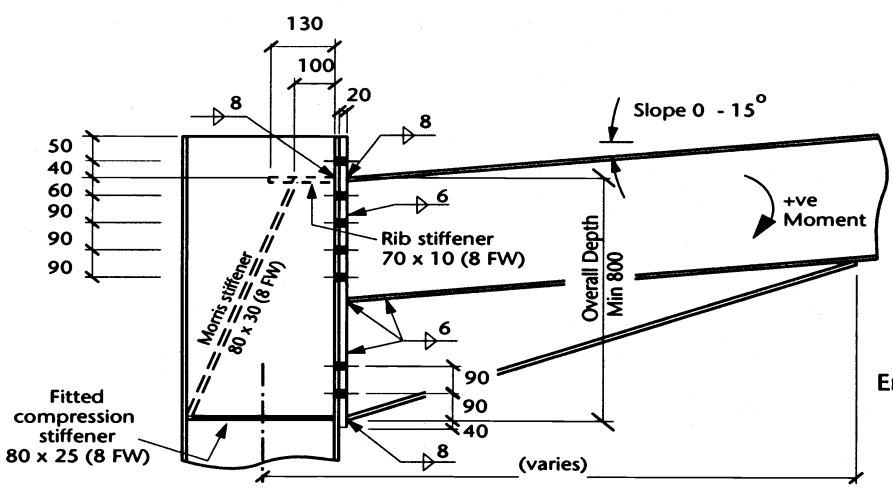
406 x 140 x 39 UB Rafters to 457 x 191 UB and 406 x 178 UB Columns
Flush End Plate - All Design Grade 43 Steel



Bolts M24 8.8
at 90 gauge
End plate 200 x 20

Column		Connection Moment Capacity (kNm) Compression Stiffener only	Compression Stiffener & Rib Stiffener	Compression Stiffener & Morris Stiffener	Connection Shear Capacity (kN)	Maximum Axial Force in Rafter (kN) ($comp^n$ + ve)	Haunch Cutting
457 x 191 x 74 UB	+ ve	447	455	532	950	+ 213	457 x 152 x 52UB
(M_{cx} = 456kNm)	- ve	– 196				– 196	
457 x 191 x 67 UB	+ ve	395	422	495	950	+ 161	
(M_{cx} = 405kNm)	- ve	– 165				– 155	
406 x 178 x 60 UB	+ ve	346	359	497	950	+ 70	
(M_{cx} = 329kNm)	- ve	– 147				– 176	
406 x 178 x 54 UB	+ ve	284	334	370	902	+ 185	
(M_{cx} = 289kNm)	- ve	– 136				– 84	

406 x 140 x 39 UB Rafters to 457 x 191 UB and 406 x 178 UB Columns
Extended End Plate - All Design Grade 43 Steel

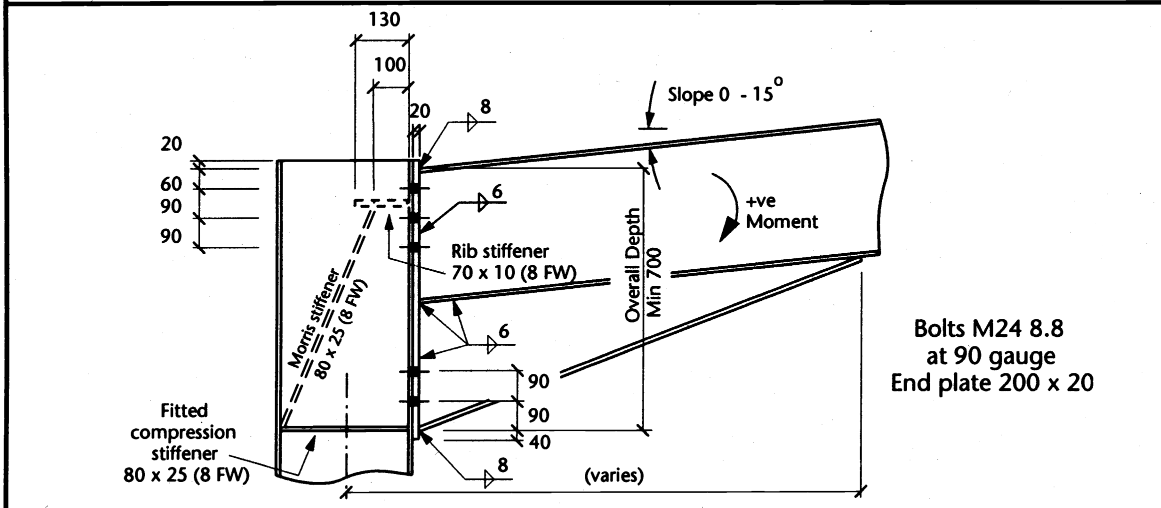


Bolts M24 8.8
at 90 gauge
End plate 200 x 20

Column		Connection Moment Capacity (kNm) Compression Stiffener only	Compression Stiffener & Rib Stiffener	Compression Stiffener & Morris Stiffener	Connection Shear Capacity (kN)	Maximum Axial Force in Rafter (kN) ($comp^n$ + ve)	Haunch Cutting
457 x 191 x 74 UB	+ ve	509	516	701	1056	+ 70	457 x 152 x 52UB
(M_{cx} = 456kNm)	- ve	– 241				– 127	
457 x 191 x 67 UB	+ ve	460	483	615	1056	+ 70	
(M_{cx} = 405kNm)	- ve	– 204				– 97	
406 x 178 x 60 UB	+ ve	395	403	611	1056	+ 70	
(M_{cx} = 329kNm)	- ve	– 186				– 120	
406 x 178 x 54 UB	+ ve	354	379	460	1008	+ 70	
(M_{cx} = 289kNm)	- ve	– 169				-31	

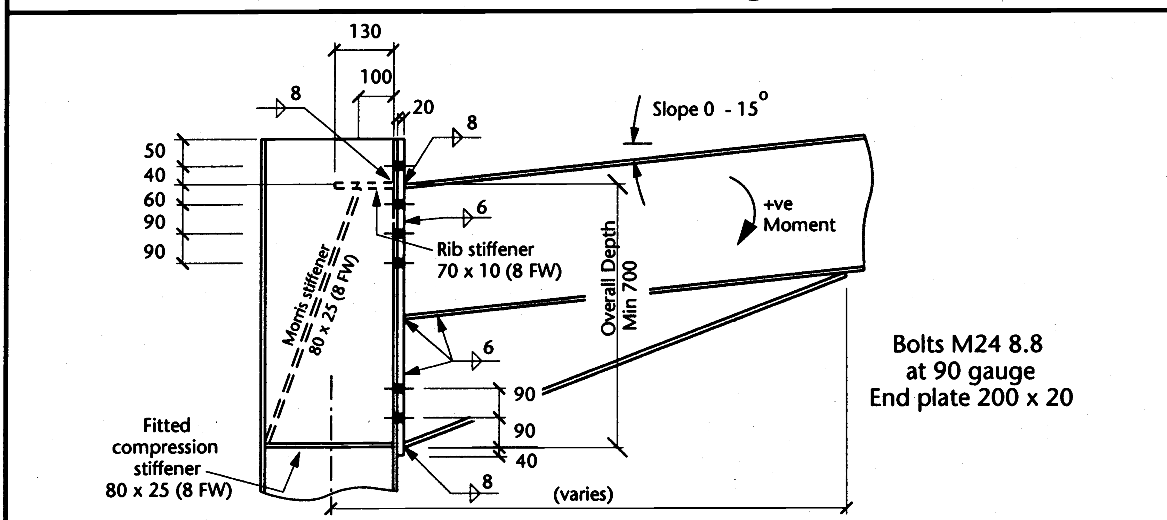
PORTAL FRAME EAVES HAUNCH CONNECTIONS

356 x 127 x 33 UB Rafters to 457 x 191 UB and 406 x 140 UB Columns Flush End Plate - All Design Grade 43 Steel



Column		Connection Moment Capacity (kNm)			Connection Shear Capacity (kN)	Maximum Axial Force in Rafter (kN) (compⁿ + ve)	Haunch Cutting
		Compression Stiffener only	Compression Stiffener & Rib Stiffener	Compression Stiffener & Morris Stiffener			
457 x 191 x 67 UB	+ ve	332	339	374	796	+ 156	406 x 140 x 39 UB
(M_{cx} = 405kNm)	- ve		– 168			–147	
406 x 178 x 60 UB	+ ve	283	309	359	796	+ 70	
(M_{cx} = 329kNm)	- ve		– 126			–176	
406 x 178 x 54 UB	+ ve	207	281	281	796	+ 211	
(M_{cx} = 289kNm)	- ve		– 117			– 84	

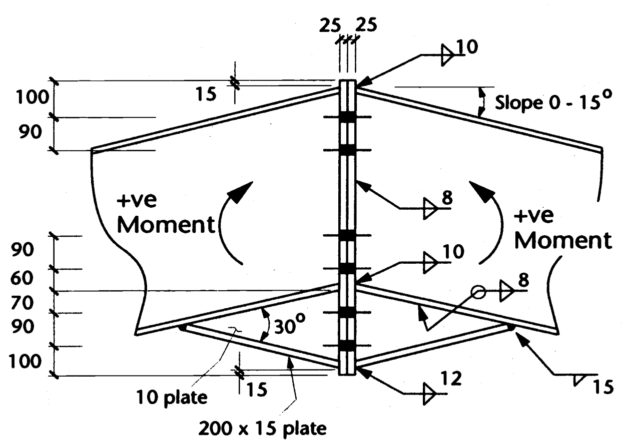
356 x 127 x 33 UB Rafters to 457 x 191 UB and 406 x 140 UB Columns Extended End Plate - All Design Grade 43 Steel



Column		Connection Moment Capacity (kNm)			Connection Shear Capacity (kN)	Maximum Axial Force in Rafter (kN) (compⁿ + ve)	Haunch Cutting
		Compression Stiffener only	Compression Stiffener & Rib Stiffener	Compression Stiffener & Morris Stiffener			
457 x 191 x 67 UB	+ ve	387	390	512	902	+ 70	406 x 140 x 39 UB
(M_{cx} = 405kNm)	- ve		– 207			– 72	
406 x 178 x 60 UB	+ ve	341	349	466	902	+ 70	
(M_{cx} = 329kNm)	- ve		– 160			– 120	
406 x 178 x 54 UB	+ ve	287	328	357	902	+ 70	
(M_{cx} = 289kNm)	- ve		– 144			– 32	

PORTAL FRAME APEX HAUNCH CONNECTIONS

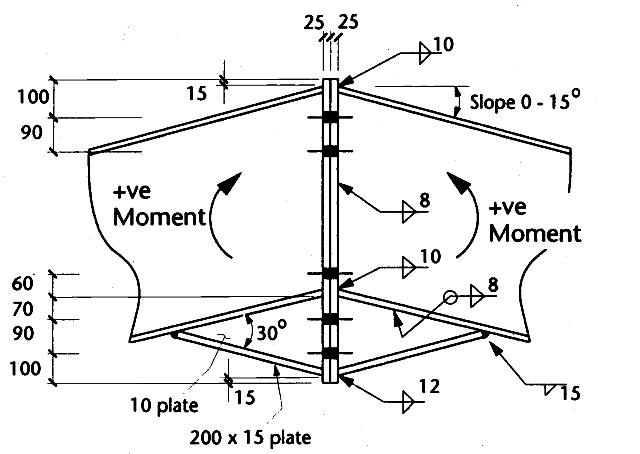
533 x 210 x 92 UB Rafter *(M_{cx} = 651kNm)*



	Moment Capacity (kNm)	Maximum Axial Force in Rafter (kN) (comp^n +ve)	Shear Capactiy
+ve Moment	652	333	950
– ve Moment	– 435	– 70	

- Bolts M24 grade 8.8
- 90 gauge
- End plate 250 x 25

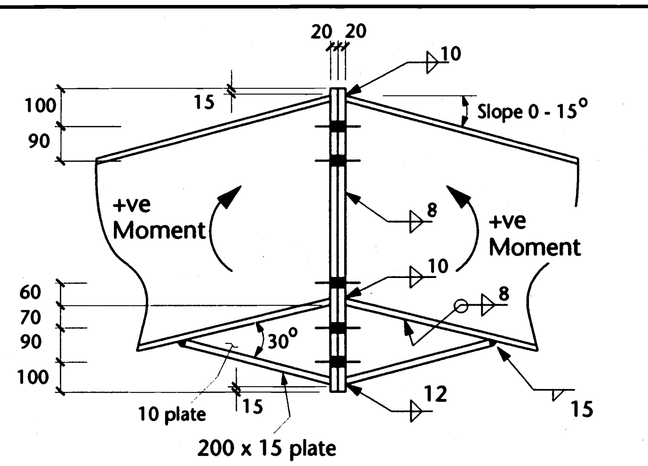
533 x 210 x 82 UB Rafter *(M_{cx} = 566kNm)*



	Moment Capacity (kNm)	Maximum Axial Force in Rafter (kN) (comp^n +ve)	Shear Capactiy
+ve Moment	567	398	845
– ve Moment	– 431	– 70	

- Bolts M24 grade 8.8
- 90 gauge
- End plate 250 x 25

457 x 191 x 74UB Rafter *(M_{cx} = 456kNm)*

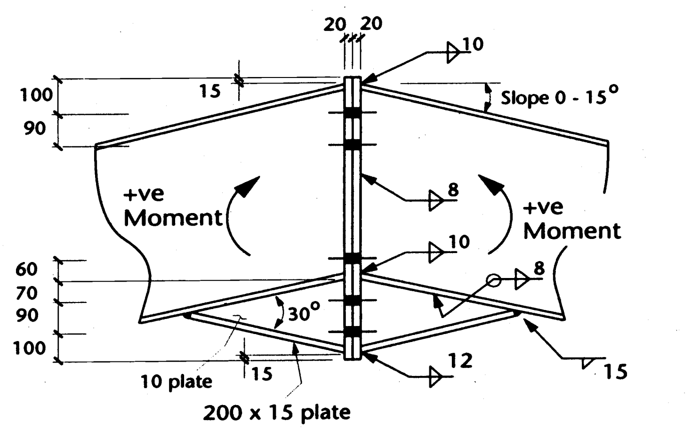


	Moment Capacity (kNm)	Maximum Axial Force in Rafter (kN) (comp^n +ve)	Shear Capactiy
+ve Moment	466	550	845
– ve Moment	– 334	– 70	

- Bolts M24 grade 8.8
- 90 gauge
- End plate 200 x 20

PORTAL FRAME APEX HAUNCHCONNECTIONS

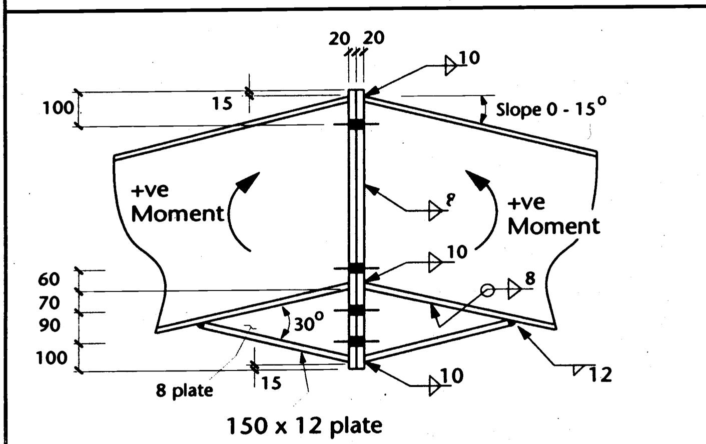
457 x 191 x 67UB Rafter (M_{cx} = 405kNm)



	Moment Capacity (kNm)	Maximum Axial Force in Rafter (kN) (compⁿ +ve)	Shear Capactiy
+ve Moment	462	395	845
– ve Moment	– 332	– 70	

- Bolts M24 grade 8.8
- 90 gauge
- End plate 200 x 20

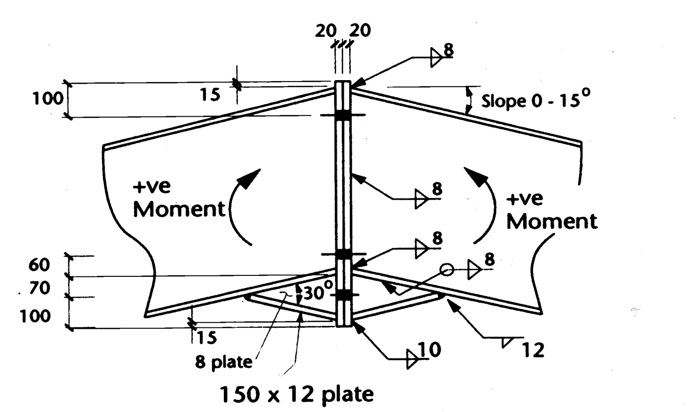
457 x 152 x 60 UB Rafter (M_{cx} = 353kNm)



	Moment Capacity (kNm)	Maximum Axial Force in Rafter (kN) (compⁿ +ve)	Shear Capactiy
+ve Moment	458	237	580
– ve Moment	– 179	– 70	

- Bolts M24 grade 8.8
- 90 gauge
- End plate 200 x 20

457 x 152 x 52 UB Rafter (M_{cx} = 301kNm)

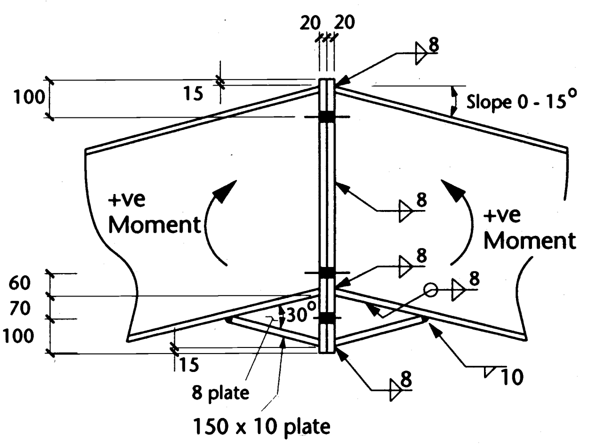


	Moment Capacity (kNm)	Maximum Axial Force in Rafter (kN) (compⁿ +ve)	Shear Capactiy
+ve Moment	287	372	475
– ve Moment	– 151	– 70	

- Bolts M24 grade 8.8
- 90 gauge
- End plate 200 x 20

PORTAL FRAME APEX HAUNCH CONNECTIONS

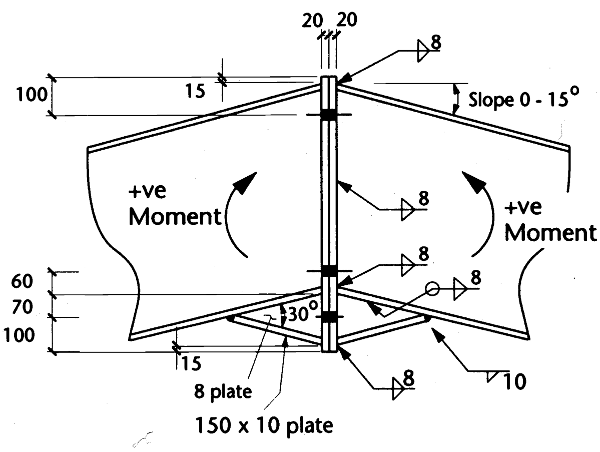
406 x 140 x 46 UB Rafter (M_{cx} = 245kNm)



	Moment Capacity (kNm)	Maximum Axial Force in Rafter (kN) (comp^n +ve)	Shear Capactiy
+ve Moment	260	245	475
– ve Moment	– 138	– 70	

- Bolts M24 grade 8.8
- 90 gauge
- End plate 200 x 20

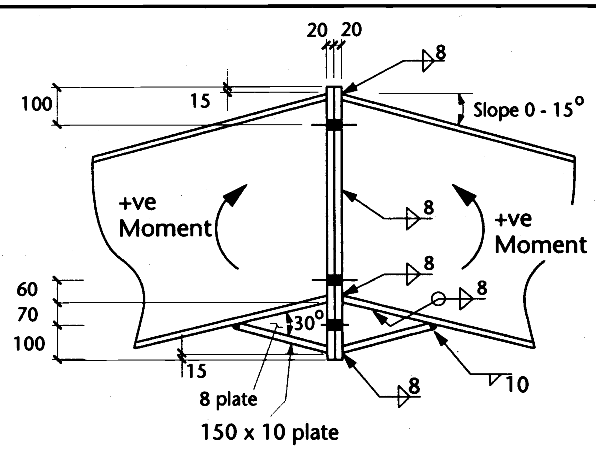
406 x 140 x 39 UB Rafter (M_{cx} = 198kNm)



	Moment Capacity (kNm)	Maximum Axial Force in Rafter (kN) (comp^n +ve)	Shear Capactiy
+ve Moment	254	84	475
– ve Moment	– 136	– 70	

- Bolts M24 grade 8.8
- 90 gauge
- End plate 200 x 20

356 x 127 x 33 UB Rafter (M_{cx} = 148kNm)



	Moment Capacity (kNm)	Maximum Axial Force in Rafter (kN) (comp^n +ve)	Shear Capactiy
+ve Moment	198	70	475
– ve Moment	– 122	– 70	

- Bolts M24 grade 8.8
- 90 gauge
- End plate 200 x 20

WIND-MOMENT CONNECTIONS

NOTES ON USE OF THE CAPACITY TABLES

Tables are presented for connections suitable for use in wind-moment frames as described in Section 3. Connections using M20 8.8 bolts, with flush and extended end plate details are shown, followed by similar connections with M24 8.8 bolts. The details are made symmetrical to suit the reversible moments expected in wind-moment frames.

The moment capacities of the connection shown may be used for all weights of beams, in design grades 43 or 50, within the serial sizes indicated. All end plates are design grade 43. Column side capacities for design grades 43 and 50 must be checked as described below.

For the connection to work in the intended manner it is important that plate size and steel grade, bolt sizes, weld sizes and dimensions are rigidly adhered to. Deviating from them may either reduce the resistance of the connection, compromise its ductility or invalidate the column check. A table of dimensions for detailing to suit individual beams is provided on page 219.

Axial forces in the beams within wind-moment frames are generally ignored in design (reference 11), and therefore the standard connections are calculated without considering them.

BEAM SIDE

Moment Capacity The moment capacity for the beam side of the connections shown is calculated using the method of Section 2.8. Bolt row forces are shown in the diagram.

An asterisk * indicates that, with the detail illustrated, the beam sections noted can only be used in design grade 50 steel because, in design grade 43 steel, they have a beam flange compression flange capacity which is less than ΣF_r and the connection is not sufficiently ductile.

If reduced bolt row forces on the column side limit development of the beam side forces shown, a reduced moment capacity must be calculated.

Dimension A Is the lever arm from the centre of compression to the lowest row of tension bolts.

Weld Sizes All flange welds to be full strength with a minimum visible fillet of 10mm (Section 2, STEP 7). All web welds to be continuous 8FW.

COLUMN SIDE

Tension Zone A tick ✔ in the table indicates that the column flange and web in tension have a greater capacity than the beam force(s) indicated in the beam table. Where the column has a smaller capacity, reduced bolt row forces are shown. A reduced moment resistance may be determined from these lower forces, or the column flange may be stiffened in the tension zone (Section 2, STEP 6D).

The capacities have been calculated assuming that the column top is at least 100mm above the beam flange or top row of bolts.

Where stiffening is employed the bolt row forces must be re-calculated (Section 2, STEP 1A) and the compression zone checked (Section 2, STEP 2A).

Compression Zone A tick ✔ in the table indicates that the column web has a greater compression capacity than the sum of the bolt row forces (ΣF_r). The check was made using a stiff bearing length from the beam side of the connection of 50mm.

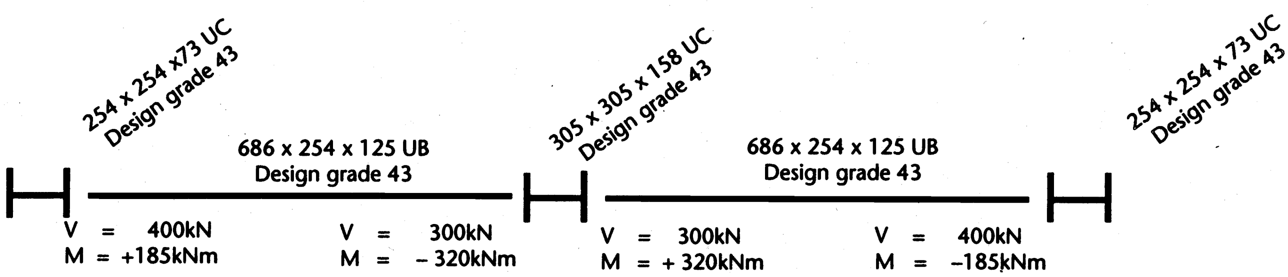
An **S** in the table shows that column web compression capacity is lower than the sum of the bolt row forces (ΣF_r); the figure in brackets shows the column web compression capacity. The web must be stiffened to resist ΣF_r.

Panel Shear Capacity The panel shear capacity is that of the column web. The applied web panel shear must take account of beams connecting onto both flanges and the direction of the applied moments. (See Section 2, STEP 3.)

Worked Example Using the Capacity Tables

DESIGN EXAMPLE 1

Design connections for the configurations and forces shown, the moments are from wind forces and are reversable:

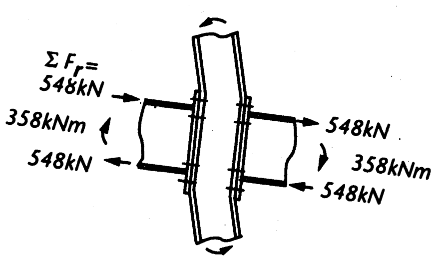


Connection to the inner column

Try an extended end plate connection with two rows M24 8.8 Bolts - 250 x 15 End Plate (page 216)

	Beam side	**Column side**
Moment capacity	*358kNm > 320kNm OK*	***Tension zone*** ✔ *OK* ***Compression zone*** ✔ *OK* *(no stiffening required)*
Vertical shear	*739kN > 300kN OK*	*Note: If the column side flange is thinner than the end plate bolt bearing on the flange should be checked.*

Column web panel shear



816kN < 2 x 548 kN (two beams)

< 1096kN unsatisfactory

Web strengthening is required

Supplementary web plate or diagonal stiffeners to be provided
(See STEP 6D and example pages 131-133)

Note: The above calculation is conservative since the applied moments are 320kNm and ΣF_r can be reduced by the difference in the table value and the applied value divided by the lowest lever arm (dimension A).

$$\Sigma F_r \text{ reduction} = \frac{(358 - 320) \times 10^3}{610} = 62\text{kN}$$

∴ applied panel shear

$$= (548 - 62) \times 2$$
$$= 972\text{kN} > 816\text{KN}$$

unsatisfactory

Web strengthening is still required

Connection to outer columns

Try a connection identical to inner column:

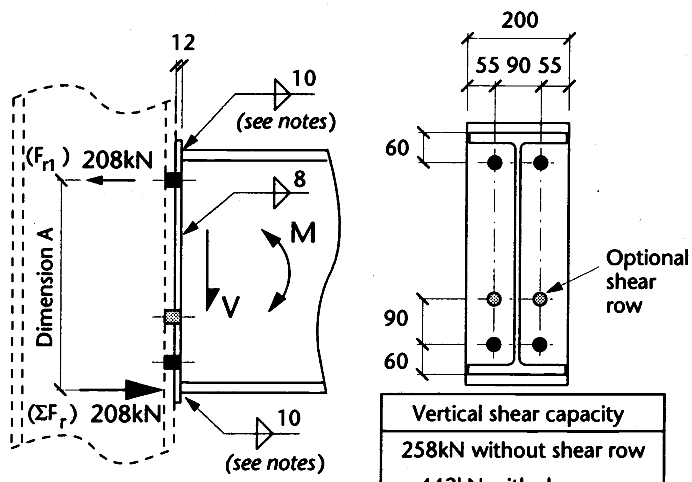
	Beam side	Column side
Moment capacity	358kNm > 185kNm OK	**Tension zone:** 2nd. bolt row = 274kN Reduced moment capacity $= (242 \times (0.610 + 0.10)) + (274 \times 0.610)$ $=$ 339kNm > 185 kNm OK
Vertical Shear	793kN > 400kN OK	
Column Web Panel shear		$\Sigma F_r = 242 + 274 = 516kN > 360KN$ unsatisfactory The web may be reinforced by a supplementary web plate or by diagonal stiffeners. Alternatively a reduced moment capacity may be calculated: $F_{r2} = 360 - 242 = 118kN$ Reduced moment capacity $= (242 \times (0.610 + 0.10)) + (118 \times 0.610)$ $=$ 244 kNm > 185 kNm OK **Compression Zone** $\Sigma F_r =$ 360 kN < 436kN OK no stiffening required

WIND-MOMENT CONNECTIONS

1 ROW M20 8.8 BOLTS
200 x 12 DESIGN GRADE 43 FLUSH END PLATE

Beam Side

BEAM - DESIGN GRADES 43 & 50		
Beam Serial Size	Dimension 'A' (mm)	Moment Capacity (kNm)
457 x 191	387	**80**
457 x 152	384	**80**
406 x 178	337	**70**
406 x 140	333	**69**
356 x 171	287	**60**
356 x 127	284	**59**
305 x 165	239	**50**
305 x 127	239	**49**
305 x 102	241	**50**
254 x 146	187	**39**
254 x 102	191	**40**



Vertical shear capacity
258kN without shear row
442kN with shear row

Column Side

DESIGN GRADE 43			COLUMN	DESIGN GRADE 50		
Panel Shear Capacity (kN)	Tension Zone F_{r1} (kN)	Compn. Zone	Serial Size	Compn. Zone	Tension Zone F_{r1} (kN)	Panel Shear Capacity (kN)
1000	✔	✔	356 x 368 x 202	✔	✔	1300
849	✔	✔	177	✔	✔	1110
725	✔	✔	153	✔	✔	944
605	✔	✔	129	✔	✔	788
1037	✔	✔	305 x 305 x 198	✔	✔	1350
816	✔	✔	158	✔	✔	1060
703	✔	✔	137	✔	✔	916
595	✔	✔	118	✔	✔	775
503	✔	✔	97	✔	✔	649
882	✔	✔	254 x 254 x 167	✔	✔	1150
685	✔	✔	132	✔	✔	893
551	✔	✔	107	✔	✔	718
434	✔	✔	89	✔	✔	566
360	✔	✔	73	✔	✔	465
459	✔	✔	203 x 203 x 86	✔	✔	598
353	✔	✔	71	✔	✔	460
322	✔	✔	60	✔	✔	415
272	✔	✔	52	✔	✔	351
245	198	✔	46	✔	✔	316

Tension Zone:

✔ Column satisfactory for bolt row tension values shown for the beam side.

xxx Calculate reduced moment capacity using the reduced bolt row value.

Compression Zone:

✔ Column capacity exceeds ΣF_r

[shaded box] Sections not Class 1 and therefore **not** suitable for use in wind moment frames.

See:
Notes - page 202
Example - page 203

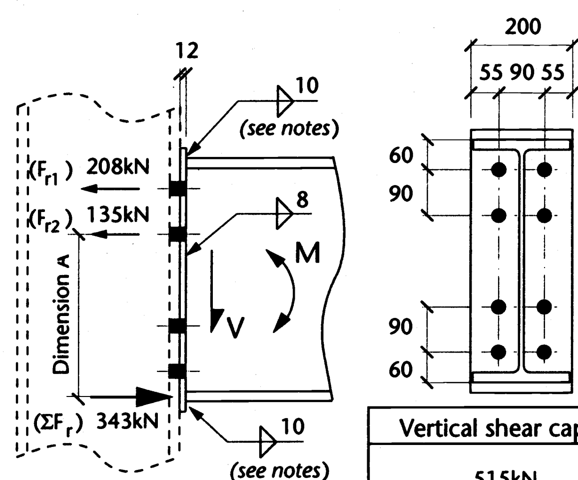
WIND-MOMENT CONNECTIONS

2 ROWS M20 8.8 BOLTS
200 x 12 DESIGN GRADE 43 FLUSH END PLATE

Beam Side

BEAM - DESIGN GRADES 43 & 50

Beam Serial Size	Dimension 'A' (mm)	Moment Capacity (kNm)
533 x 210	372	**150**
457 x 191	297	**123**
457 x 152	294	**122**
406 x 140	247	**105**
406 x 140	243	**102**



Vertical shear capacity
515kN

Column Side

DESIGN GRADE 43				COLUMN	DESIGN GRADE 50			
Panel Shear Capacity (kN)	Tension Zone F_{r1} (kN)	Tension Zone F_{r2} (kN)	Compn. Zone	Serial Size	Compn. Zone	Tension Zone F_{r1} (kN)	Tension Zone F_{r2} (kN)	Panel Shear Capacity (kN)
1000	✔	✔	✔	356 x 368 x 202	✔	✔	✔	1300
849	✔	✔	✔	177	✔	✔	✔	1100
725	✔	✔	✔	153	✔	✔	✔	944
605	✔	✔	✔	129	✔	✔	✔	788
1037	✔	✔	✔	305 x 305 x 198	✔	✔	✔	1350
816	✔	✔	✔	158	✔	✔	✔	1060
703	✔	✔	✔	137	✔	✔	✔	916
595	✔	✔	✔	118	✔	✔	✔	775
503	✔	✔	✔	97	✔	✔	✔	649
882	✔	✔	✔	254 x 254 x 167	✔	✔	✔	1150
685	✔	✔	✔	132	✔	✔	✔	893
551	✔	✔	✔	107	✔	✔	✔	718
434	✔	✔	✔	89	✔	✔	✔	566
360	✔	✔	✔	73	✔	✔	✔	465
459	✔	✔	✔	203 x 203 x 86	✔	✔	✔	598
353	✔	✔	✔	71	✔	✔	✔	460
322	✔	✔	✔	60	✔	✔	✔	415
272	✔	✔	✔	52	✔	✔	✔	351
245	198	97	✔	46	✔	✔	✔	316

Tension Zone:
✔ ✔ Column satisfactory for bolt row tension values shown for the beam side.
✔ xxx Calculate reduced moment capacity using the reduced bolt row values.

Compression Zone:
✔ Column capacity exceeds ΣF_r

(shaded box) Sections not Class 1 and therefore **not** suitable for use in wind moment frames.

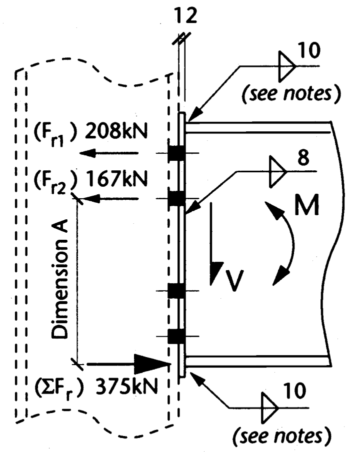
See:
Notes - page 202
Example - page 203

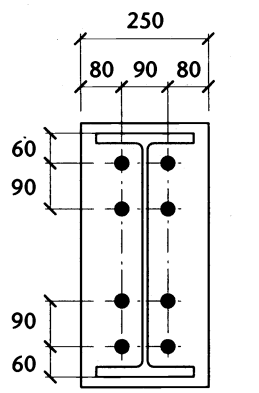
WIND-MOMENT CONNECTIONS

2 ROWS M20 8.8 BOLTS
250 x 12 DESIGN GRADE 43 FLUSH END PLATE

Beam Side

BEAM - DESIGN GRADES 43 & 50		
Beam Serial Size	Dimension 'A' (mm)	Moment Capacity (kNm)
686 x 254	520	**220**
610 x 229	445	**190**
533 x 210	372	**160**
457 x 191	297	**131**
457 x 152	294	**129**





Vertical shear capacity
515kN

Column Side

DESIGN GRADE 43				COLUMN	DESIGN GRADE 50			
Panel Shear Capacity (kN)	Tension Zone F_{r1} (kN)	Tension Zone F_{r2} (kN)	Compn. Zone	Serial Size	Compn. Zone	Tension Zone F_{r1} (kN)	Tension Zone F_{r2} (kN)	Panel Shear Capacity (kN)
1000	✔	✔	✔	356 x 368 x 202	✔	✔	✔	1302
849	✔	✔	✔	177	✔	✔	✔	1105
725	✔	✔	✔	153	✔	✔	✔	944
605	✔	✔	✔	129	✔	✔	✔	787
1037	✔	✔	✔	305 x 305 x 198	✔	✔	✔	1350
816	✔	✔	✔	158	✔	✔	✔	1062
703	✔	✔	✔	137	✔	✔	✔	915
595	✔	✔	✔	118	✔	✔	✔	774
503	✔	✔	✔	97	✔	✔	✔	649
882	✔	✔	✔	254 x 254 x 167	✔	✔	✔	1149
685	✔	✔	✔	132	✔	✔	✔	892
551	✔	✔	✔	107	✔	✔	✔	717
434	✔	✔	✔	89	✔	✔	✔	566
360	✔	✔	✔	73	✔	✔	✔	465
459	✔	✔	✔	203 x 203 x 86	✔	✔	✔	598
353	✔	✔	✔	71	✔	✔	✔	460
322	✔	✔	✔	60	✔	✔	✔	415
272	✔	✔	**S** *(360)*	52	✔	✔	✔	351
245	198	97	✔	46	✔	✔	✔	316

Tension Zone:

✔ ✔ Column satisfactory for bolt row tension values shown for the beam side.

xxx xxx Calculate reduced moment capacity using the reduced bolt row values.

Compression Zone:

✔ Column capacity exceeds ΣF_r

S *(xxx)* Column requires stiffening to resist ΣF_r *(Value is the column web capacity.)*

Sections not Class 1 and therefore **not** suitable for use in wind moment frames.

See:

Notes *- page 202*

Example *- page 203*

WIND-MOMENT CONNECTIONS

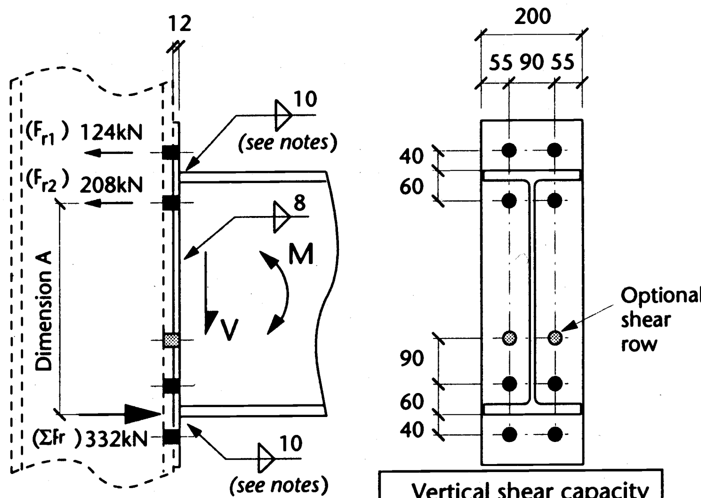
Beam Side

2 ROW M20 8.8 BOLTS
200 x 12 DESIGN GRADE 43 EXTENDED END PLATE

BEAM - DESIGN GRADES 43 & 50

Beam Serial Size	Dimension 'A' (mm)	Moment Capacity (kNm)
533 x 210	462	165
457 x 191	387	141
457 x 152	384	140
406 x 178	337	124
406 x 140	333	123
356 x 171	287	107
356 x 127	284	107
305 x 165	239	91
305 x 127	239	91
305 x 102 *	241	92
254 x 146	187	74
254 x 102 *	191	75

* 305 x 102 x 25, 254 x 102 x 25 x 22 } these sections suitable in design grade 50 only



Vertical shear capacity
515kN without shear row
698kN with shear row

Column Side

DESIGN GRADE 43				COLUMN	DESIGN GRADE 50			
Panel Shear Capacity (kN)	Tension Zone F_{r1} (kN)	Tension Zone F_{r2} (kN)	Compn. Zone	Serial Size	Compn. Zone	Tension Zone F_{r1} (kN)	Tension Zone F_{r2} (kN)	Panel Shear Capacity (kN)
1000	✔	✔	✔	356 x 368 x 202	✔	✔	✔	1300
849	✔	✔	✔	177	✔	✔	✔	1110
725	✔	✔	✔	153	✔	✔	✔	944
605	✔	✔	✔	129	✔	✔	✔	788
1037	✔	✔	✔	305 x 305 x 198	✔	✔	✔	1350
816	✔	✔	✔	158	✔	✔	✔	1060
703	✔	✔	✔	137	✔	✔	✔	916
595	✔	✔	✔	118	✔	✔	✔	775
503	✔	✔	✔	97	✔	✔	✔	649
882	✔	✔	✔	254 x 254 x 167	✔	✔	✔	1150
685	✔	✔	✔	132	✔	✔	✔	893
551	✔	✔	✔	107	✔	✔	✔	718
434	✔	✔	✔	89	✔	✔	✔	566
360	✔	206	✔	73	✔	✔	✔	465
459	✔	✔	✔	203 x 203 x 86	✔	✔	✔	598
353	✔	✔	✔	71	✔	✔	✔	460
322	✔	191	✔	60	✔	✔	202	415
272	✔	181	✔	52	✔	✔	190	351
245	✔	107	✔	46	✔	✔	181	316

Tension Zone:

✔ ✔ Column satisfactory for bolt row tension values shown for the beam side.

✔ xxx Calculate reduced moment capacity using the reduced bolt row value.

Compression Zone:

✔ Column capacity exceeds ΣF_r

[shaded box] Sections not Class 1 and therefore **not** suitable for use in wind moment frames.

See:
Notes - page 202
Example - page 203

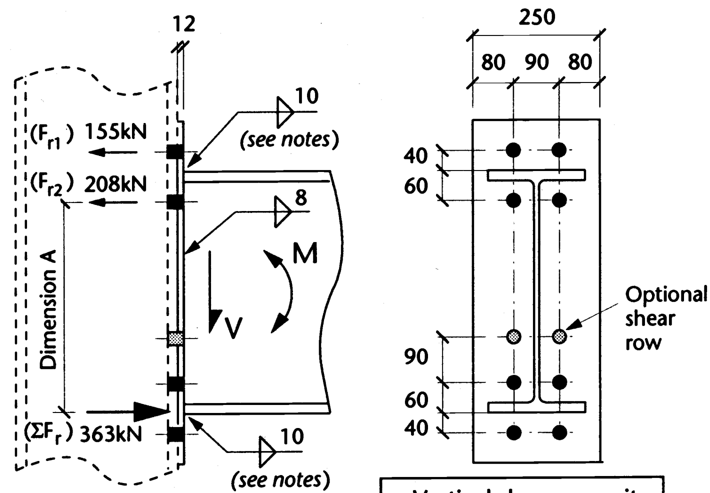
WIND-MOMENT CONNECTIONS

2 ROWS M20 8.8 BOLTS
250 x 12 DESIGN GRADE 43 EXTENDED END PLATE

Beam Side

BEAM - DESIGN GRADES 43 & 50

Beam Serial Size	Dimension 'A' (mm)	Moment Capacity (kNm)
686 x 254	610	236
610 x 229	535	209
533 x 210	462	183
457 x 191	387	156
457 x 152	384	155



Vertical shear capacity
515kN without shear row
698kN with shear row

Column Side

DESIGN GRADE 43				COLUMN	DESIGN GRADE 50			
Panel Shear Capacity (kN)	Tension Zone F_{r1} (kN)	Tension Zone F_{r2} (kN)	Compn. Zone	Serial Size	Compn. Zone	Tension Zone F_{r1} (kN)	Tension Zone F_{r2} (kN)	Panel Shear Capacity (kN)
1000	✔	✔	✔	356 x 368 x 202	✔	✔	✔	1300
849	✔	✔	✔	177	✔	✔	✔	1110
725	✔	✔	✔	153	✔	✔	✔	944
605	✔	✔	✔	129	✔	✔	✔	788
1037	✔	✔	✔	305 x 305 x 198	✔	✔	✔	1350
816	✔	✔	✔	158	✔	✔	✔	1060
703	✔	✔	✔	137	✔	✔	✔	916
595	✔	✔	✔	118	✔	✔	✔	775
503	✔	✔	✔	97	✔	✔	✔	649
882	✔	✔	✔	254 x 254 x 167	✔	✔	✔	1150
685	✔	✔	✔	132	✔	✔	✔	893
551	✔	✔	✔	107	✔	✔	✔	718
434	✔	✔	✔	89	✔	✔	✔	566
360	✔	206	✔	73	✔	✔	✔	465
459	✔	✔	✔	203 x 203 x 86	✔	✔	✔	598
353	✔	✔	✔	71	✔	✔	✔	460
322	✔	191	✔	60	✔	✔	202	415
272	✔	181	✔	52	✔	✔	190	351
245	✔	107	✔	46	✔	✔	181	316

Tension Zone:

✔ ✔ Column satisfactory for bolt row tension values shown for the beam side.

✔ xxx Calculate reduced moment capacity using the reduced bolt row value.

Compression Zone:

✔ Column capacity exceeds ΣF_r

(shaded box) Sections not Class 1 and therefore **not** suitable for use in wind moment frames.

See:
Notes - *page 202*
Example - *page 203*

WIND-MOMENT CONNECTIONS

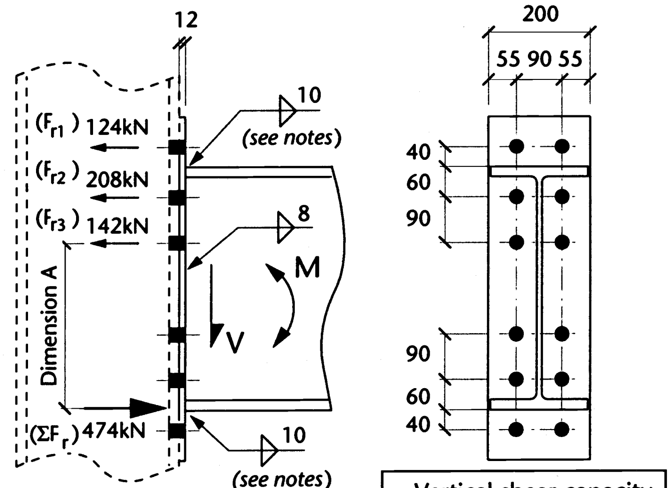
3 ROWS M20 8.8 BOLTS
200 x 12 DESIGN GRADE 43 EXTENDED END PLATE

Beam Side

BEAM - DESIGN GRADES 43 & 50

Beam Serial Size	Dimension 'A' (mm)	Moment Capacity (kNm)
533 x 210	372	**220**
457 x 191	297	**184**
457 x 152	294	**182**
406 x 178	247	**160**
406 x 140 *	243	**155**

* 406 x 140 x 39 is suitable in design grade 50 only



Vertical shear capacity
772kN

Column Side

DESIGN GRADE 43					COLUMN	DESIGN GRADE 50				
Panel Shear Capacity (kN)	Tension Zone F_{r1} (kN)	Tension Zone F_{r2} (kN)	Tension Zone F_{r3} (kN)	Compn. Zone	Serial Size	Compn. Zone	Tension Zone F_{r1} (kN)	Tension Zone F_{r2} (kN)	Tension Zone F_{r3} (kN)	Panel Shear Capacity (kN)
1000	✔	✔	✔	✔	356 x 368 x 202	✔	✔	✔	✔	1300
849	✔	✔	✔	✔	177	✔	✔	✔	✔	1110
725	✔	✔	✔	✔	153	✔	✔	✔	✔	944
605	✔	✔	✔	✔	129	✔	✔	✔	✔	788
1037	✔	✔	✔	✔	305 x 305 x 198	✔	✔	✔	✔	1350
816	✔	✔	✔	✔	158	✔	✔	✔	✔	1060
703	✔	✔	✔	✔	137	✔	✔	✔	✔	916
595	✔	✔	✔	✔	118	✔	✔	✔	✔	775
503	✔	✔	✔	✔	97	✔	✔	✔	✔	649
882	✔	✔	✔	✔	254 x 254 x 167	✔	✔	✔	✔	1150
685	✔	✔	✔	✔	132	✔	✔	✔	✔	893
551	✔	✔	✔	✔	107	✔	✔	✔	✔	718
434	✔	✔	✔	✔	89	✔	✔	✔	✔	566
360	✔	206	✔	S (436)	73	✔	✔	✔	✔	465
459	✔	✔	✔	✔	203 x 203 x 86	✔	✔	✔	✔	598
353	✔	✔	✔	✔	71	✔	✔	✔	✔	460
322	✔	191	✔	S (440)	60	✔	✔	202	✔	415
272	✔	181	121	S (360)	52	✔	✔	190	✔	351
245	✔	107	90	S (313)	46	S (404)	✔	181	118	316

Tension Zone:

✔ ✔ ✔ Column satisfactory for bolt row tension values shown for the beam side.

✔ xxx xxx Calculate reduced moment capacity using the reduced bolt row values.

Compression Zone:

✔ Column capacity exceeds ΣF_r

S (xxx) Column requires stiffening to resist ΣF_r *(Value is the column web capacity.)*

[shaded box] Sections not Class 1 and therefore **not** suitable for use in wind moment frames.

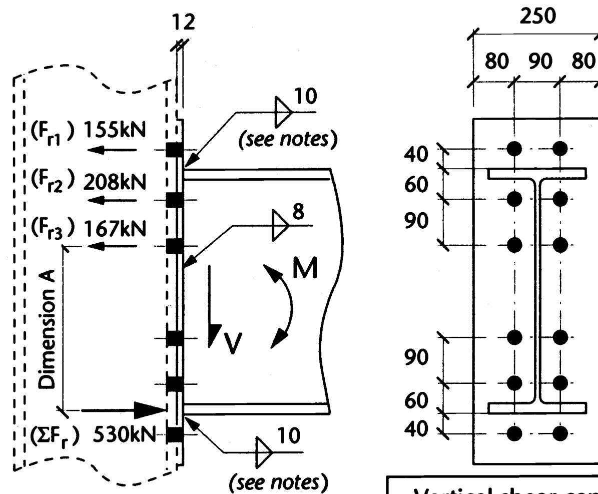
See:
Notes - page 202
Example - page 203

WIND-MOMENT CONNECTIONS

3 ROWS M20 8.8 BOLTS
250 x 12 DESIGN GRADE 43 EXTENDED END PLATE

Beam Side

BEAM - DESIGN GRADES 43 & 50		
Beam Serial Size	Dimension 'A' (mm)	Moment Capacity (kNm)
686 x 254	520	330
610 x 229	445	288
533 x 210	372	247
457 x 191	297	206
457 x 152	294	204



Vertical shear capacity
772kN

Column Side

DESIGN GRADE 43					COLUMN	DESIGN GRADE 50				
Panel Shear Capacity (kN)	Tension Zone F_{r1} (kN)	Tension Zone F_{r2} (kN)	Tension Zone F_{r3} (kN)	Compn. Zone	Serial Size	Compn. Zone	Tension Zone F_{r1} (kN)	Tension Zone F_{r2} (kN)	Tension Zone F_{r3} (kN)	Panel Shear Capacity (kN)
1000	✔	✔	✔	✔	356 x 368 x 202	✔	✔	✔	✔	1300
849	✔	✔	✔	✔	177	✔	✔	✔	✔	1110
725	✔	✔	✔	✔	153	✔	✔	✔	✔	944
605	✔	✔	✔	✔	129	✔	✔	✔	✔	788
1037	✔	✔	✔	✔	305 x 305 x 198	✔	✔	✔	✔	1350
816	✔	✔	✔	✔	158	✔	✔	✔	✔	1060
703	✔	✔	✔	✔	137	✔	✔	✔	✔	916
595	✔	✔	✔	✔	118	✔	✔	✔	✔	775
503	✔	✔	✔	✔	97	✔	✔	✔	✔	649
882	✔	✔	✔	✔	254 x 254 x 167	✔	✔	✔	✔	1150
685	✔	✔	✔	✔	132	✔	✔	✔	✔	893
551	✔	✔	✔	✔	107	✔	✔	✔	✔	718
434	✔	✔	✔	✔	89	✔	✔	✔	✔	566
360	✔	206	✔	S (436)	73	✔	✔	✔	✔	465
459	✔	✔	✔	✔	203 x 203 x 86	✔	✔	✔	✔	598
353	✔	✔	✔	S (512)	71	✔	✔	✔	✔	460
322	✔	191	✔	S (440)	60	✔	✔	202	✔	415
272	✔	181	121	S (360)	52	S (464)	✔	190	✔	351
245	✔	107	90	S (313)	46	S (404)	✔	181	118	316

Tension Zone:

✔ ✔ ✔ Column satisfactory for bolt row tension values shown for the beam side.

✔ xxx xxx Calculate reduced moment capacity using the reduced bolt row values.

Compression Zone:

✔ Column capacity exceeds ΣF_r

S *(xxx)* Column requires stiffening to resist ΣF_r *(Value is the column web capacity.)*

[shaded box] Sections not Class 1 and therefore **not** suitable for use in wind moment frames.

See:
Notes - page 202
Example - page 203

WIND-MOMENT CONNECTIONS

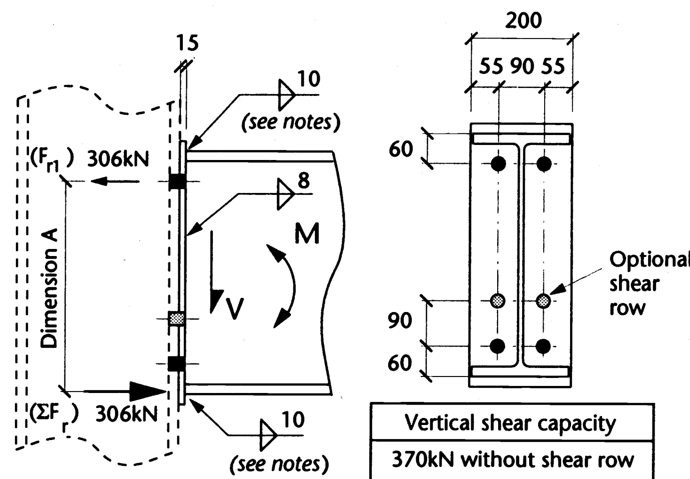
Beam Side

1 ROW M24 8.8 BOLTS
200 x 15 DESIGN GRADE 43 FLUSH END PLATE

BEAM - DESIGN GRADES 43 & 50

Beam Serial Size	Dimension 'A' (mm)	Moment Capacity (kNm)
457 x 191	387	**119**
457 x 152	384	**118**
406 x 178	337	**103**
406 x 140	333	**102**
356 x 171	287	**88**
356 x 127	284	**87**
305 x 165	239	**73**
305 x 127	238	**73**
305 x 102*	241	**74**
254 x 146	187	**57**
254 x 102 *	191	**58**

* 305 x 102 x 25 } these sections suitable
254 x 102 x 22 } in design grade 50 only



Vertical shear capacity
370kN without shear row
634kN with shear row

Column Side

DESIGN GRADE 43			COLUMN	DESIGN GRADE 50		
Panel Shear Capacity (kN)	Tension Zone F_{r1} (kN)	Compn. Zone	Serial Size	Compn. Zone	Tension Zone F_{r1} (kN)	Panel Shear Capacity (kN)
1000	✔	✔	356 x 368 x 202	✔	✔	1300
849	✔	✔	177	✔	✔	1110
725	✔	✔	153	✔	✔	944
605	✔	✔	129	✔	✔	788
1037	✔	✔	305 x 305 x 198	✔	✔	1350
816	✔	✔	158	✔	✔	1060
703	✔	✔	137	✔	✔	916
595	✔	✔	118	✔	✔	775
503	✔	✔	97	✔	✔	649
882	✔	✔	254 x 254 x 167	✔	✔	1150
685	✔	✔	132	✔	✔	893
551	✔	✔	107	✔	✔	718
434	✔	✔	89	✔	✔	566
360	297	✔	73	✔	✔	465
459	✔	✔	203 x 203 x 86	✔	✔	598
353	✔	✔	71	✔	✔	460
322	297	✔	60	✔	✔	415
272	265	✔	52	✔	296	351
245	204	✔	46	✔	263	316

Tension Zone:
✔ Column satisfactory for bolt row tension values shown for the beam side.
xxx Calculate reduced moment capacity using the reduced bolt row value.

Compression Zone:
✔ Column capacity exceeds ΣF_r

[shaded box] Sections not Class 1 and therefore **not** suitable for use in wind moment frames.

See:
Notes - page 202
Example - page 203

WIND-MOMENT CONNECTIONS

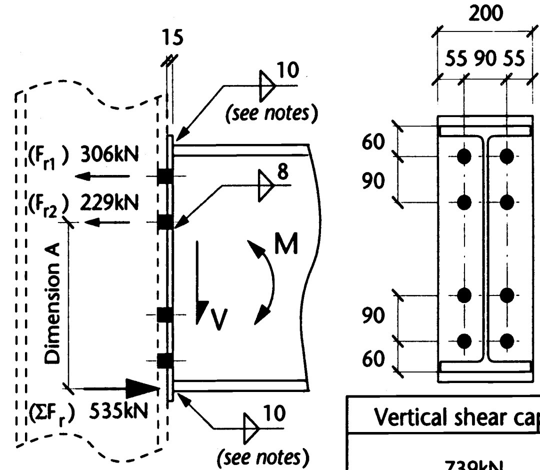
Beam Side

2 ROWS M24 8.8 BOLTS
200 x 15 DESIGN GRADE 43 FLUSH END PLATE

BEAM - DESIGN GRADES 43 & 50

Beam Serial Size	Dimension 'A' (mm)	Moment Capacity (kNm)
533 x 210	372	233
457 x 191	297	191
457 x 152	294	186
406 x 178	247	161
406 x 140 *	243	158

* 406 x 140 x 39 is suitable in design grade 50 only



Vertical shear capacity
739kN

Column Side

DESIGN GRADE 43 Panel Shear Capacity (kN)	Tension Zone F_{r1} (kN)	Tension Zone F_{r2} (kN)	Compn. Zone	COLUMN Serial Size	DESIGN GRADE 50 Compn. Zone	Tension Zone F_{r1} (kN)	Tension Zone F_{r2} (kN)	Panel Shear Capacity (kN)
1000	✔	✔	✔	356 x 368 x 202	✔	✔	✔	1300
849	✔	✔	✔	177	✔	✔	✔	1110
725	✔	✔	✔	153	✔	✔	✔	944
605	✔	✔	✔	129	✔	✔	✔	788
1037	✔	✔	✔	305 x 305 x 198	✔	✔	✔	1350
816	✔	✔	✔	158	✔	✔	✔	1060
703	✔	✔	✔	137	✔	✔	✔	916
595	✔	✔	✔	118	✔	✔	✔	775
503	✔	✔	✔	97	✔	✔	✔	649
882	✔	✔	✔	254 x 254 x 167	✔	✔	✔	1150
685	✔	✔	✔	132	✔	✔	✔	893
551	✔	✔	✔	107	✔	✔	✔	718
434	✔	✔	✔	89	✔	✔	✔	566
360	297	✔	S *(436)*	73	✔	✔	✔	465
459	✔	✔	✔	203 x 203 x 86	✔	✔	✔	598
353	✔	✔	S *(512)*	71	✔	✔	✔	460
322	297	204	S *(440)*	60	✔	✔	✔	415
272	265	118	S *(360)*	52	S *(464)*	296	198	351
245	204	90	✔	46	✔	263	116	316

Tension Zone:
✔ ✔ Column satisfactory for bolt row tension values shown for the beam side.
✔ xxx Calculate reduced moment capacity using the reduced bolt row values.

Compression Zone:
✔ Column capacity exceeds ΣF_r
S *(xxx)* Column requires stiffening to resist ΣF_r *(Value is the column web capacity.)*
[shaded box] Sections not Class 1 and therefore **not** suitable for use in wind moment frames.

See:
Notes - page 202
Example - page 203

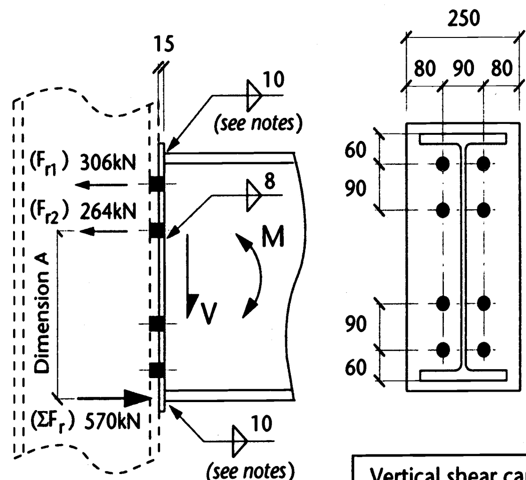
WIND-MOMENT CONNECTIONS

2 ROWS M24 8.8 BOLTS
250 x 15 DESIGN GRADE 43 FLUSH END PLATE

Beam Side

BEAM - DESIGN GRADES 43 & 50

Beam Serial Size	Dimension 'A' (mm)	Moment Capacity (kNm)
686 x 254	520	**326**
610 x 229	445	**283**
533 x 210	372	**240**
457 x 191	297	**197**
457 x 152	294	**195**



Vertical shear capacity
739kN

Column Side

DESIGN GRADE 43				COLUMN	DESIGN GRADE 50			
Panel Shear Capacity (kN)	Tension Zone F_{r1} (kN)	Tension Zone F_{r2} (kN)	Compn. Zone	Serial Size	Compn. Zone	Tension Zone F_{r1} (kN)	Tension Zone F_{r2} (kN)	Panel Shear Capacity (kN)
1000	✔	✔	✔	356 x 368 x 202	✔	✔	✔	1300
849	✔	✔	✔	177	✔	✔	✔	1110
725	✔	✔	✔	153	✔	✔	✔	944
605	✔	✔	✔	129	✔	✔	✔	788
1037	✔	✔	✔	305 x 305 x 198	✔	✔	✔	1350
816	✔	✔	✔	158	✔	✔	✔	1060
703	✔	✔	✔	137	✔	✔	✔	916
595	✔	✔	✔	118	✔	✔	✔	775
503	✔	✔	S *(553)*	97	✔	✔	✔	649
882	✔	✔	✔	254 x 254 x 167	✔	✔	✔	1150
685	✔	✔	✔	132	✔	✔	✔	893
551	✔	✔	✔	107	✔	✔	✔	718
434	✔	✔	S *(557)*	89	✔	✔	✔	566
360	297	✔	S *(436)*	73	S *(563)*	✔	✔	465
459	✔	✔	✔	203 x 203 x 86	✔	✔	✔	598
353	✔	✔	S *(512)*	71	✔	✔	✔	460
322	297	204	S *(440)*	60	S *(568)*	✔	✔	415
272	265	118	S *(360)*	52	S *(464)*	296	198	351
245	204	90	✔	46	✔	263	116	316

Tension Zone:
✔ ✔ Column satisfactory for bolt row tension values shown for the beam side.
✔ xxx Calculate reduced moment capacity using the reduced bolt row values.

Compression Zone:
✔ Column capacity exceeds ΣF_r
S *(xxx)* Column requires stiffening to resist ΣF_r *(Value is the column web capacity.)*
[shaded] Sections not Class 1 and therefore **not** suitable for use in wind moment frames.

See:
Notes - page 202
Example - page 203

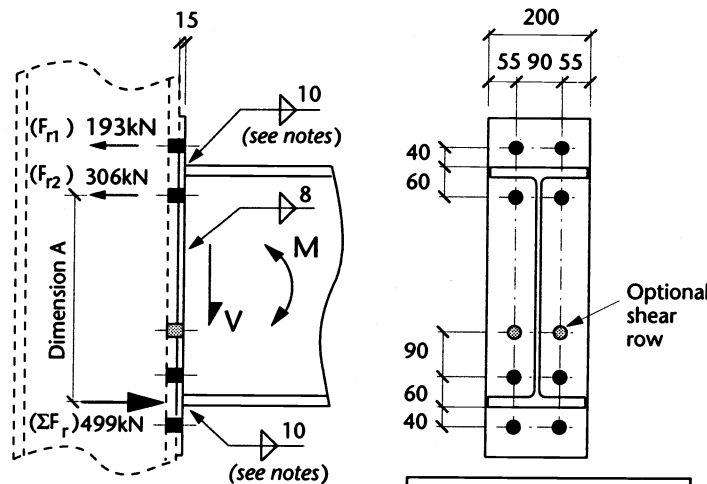
WIND-MOMENT CONNECTIONS

2 ROWS M24 8.8 BOLTS
200 x 15 DESIGN GRADE 43 EXTENDED END PLATE

Beam Side

BEAM - DESIGN GRADES 43 & 50		
Beam Serial Size	Dimension 'A' (mm)	Moment Capacity (kNm)
533 x 210	462	250
457 x 191	387	213
457 x 152	384	211
406 x 178	337	188
406 x 140 *	333	186
356 x 171	287	163
356 x 127 *	284	161
305 x 165	239	139
305 x 127	238	139

* 406 x 140 x 39, 356 x 127 x 33 } these sections suitable in design grade 50 only



Vertical shear capacity
739kN without shear row
1003kN with shear row

Column Side

DESIGN GRADE 43				COLUMN	DESIGN GRADE 50			
Panel Shear Capacity (kN)	Tension Zone F_{r1} (kN)	Tension Zone F_{r2} (kN)	Compn. Zone	Serial Size	Compn. Zone	Tension Zone F_{r1} (kN)	Tension Zone F_{r2} (kN)	Panel Shear Capacity (kN)
1000	✔	✔	✔	356 x 368 x 202	✔	✔	✔	1300
849	✔	✔	✔	177	✔	✔	✔	1110
725	✔	✔	✔	153	✔	✔	✔	944
605	✔	✔	✔	129	✔	✔	✔	788
1037	✔	✔	✔	305 x 305 x 198	✔	✔	✔	1350
816	✔	✔	✔	158	✔	✔	✔	1060
703	✔	✔	✔	137	✔	✔	✔	916
595	✔	✔	✔	118	✔	✔	✔	775
503	✔	✔	✔	97	✔	✔	✔	649
882	✔	✔	✔	254 x 254 x 167	✔	✔	✔	1150
685	✔	✔	✔	132	✔	✔	✔	893
551	✔	✔	✔	107	✔	✔	✔	718
434	✔	301	✔	89	✔	✔	✔	566
360	✔	274	S *(436)*	73	✔	✔	289	465
459	✔	✔	✔	203 x 203 x 86	✔	✔	✔	598
353	✔	276	✔	71	✔	✔	293	460
322	✔	221	✔	60	✔	✔	269	415
272	✔	131	✔	52	✔	✔	215	351
245	✔	100	✔	46	✔	✔	129	316

Tension Zone:

✔ ✔ Column satisfactory for bolt row tension values shown for the beam side.

✔ xxx Calculate reduced moment capacity using the reduced bolt row values.

Compression Zone:

✔ Column capacity exceeds ΣF_r

S *(xxx)* Column requires stiffening to resist ΣF_r *(Value is the column web capacity.)*

(shaded) Sections not Class 1 and therefore **not** suitable for use in wind moment frames.

See:
Notes - page 202
Example - page 203

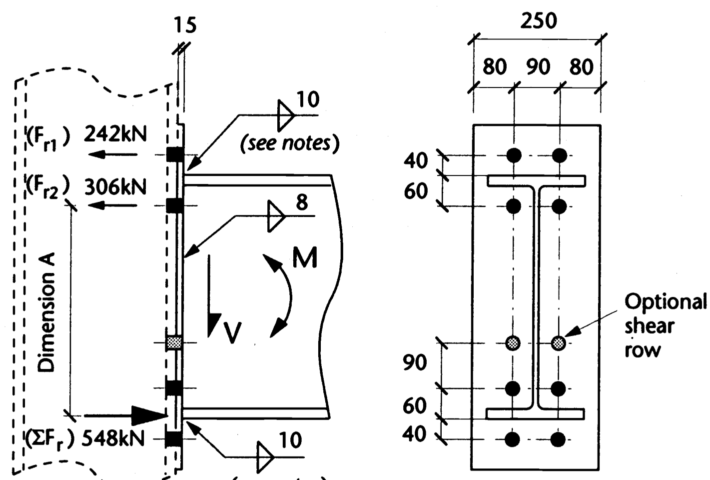
WIND-MOMENT CONNECTIONS

Beam Side

2 ROWS M24 8.8 BOLTS
250 x 15 DESIGN GRADE 43 EXTENDED END PLATE

BEAM - DESIGN GRADES 43 & 50

Beam Serial Size	Dimension 'A' (mm)	Moment Capacity (kNm)
686 x 254	610	**358**
610 x 229	535	**317**
533 x 210	462	**277**
457 x 191	387	**236**



Vertical shear capacity
739kN without shear row
1003kN with shear row

Column Side

DESIGN GRADE 43				COLUMN	DESIGN GRADE 50			
Panel Shear Capacity (kN)	Tension Zone F_{r1} (kN)	Tension Zone F_{r2} (kN)	Compn. Zone	Serial Size	Compn. Zone	Tension Zone F_{r1} (kN)	Tension Zone F_{r2} (kN)	Panel Shear Capacity (kN)
1000	✔	✔	✔	356 x 368 x 202	✔	✔	✔	1300
849	✔	✔	✔	177	✔	✔	✔	1110
725	✔	✔	✔	153	✔	✔	✔	944
605	✔	✔	✔	129	✔	✔	✔	788
1037	✔	✔	✔	305 x 305 x 198	✔	✔	✔	1350
816	✔	✔	✔	158	✔	✔	✔	1060
703	✔	✔	✔	137	✔	✔	✔	916
595	✔	✔	✔	118	✔	✔	✔	775
503	✔	✔	✔	97	✔	✔	✔	649
882	✔	✔	✔	254 x 254 x 167	✔	✔	✔	1150
685	✔	✔	✔	132	✔	✔	✔	893
551	✔	✔	✔	107	✔	✔	✔	718
434	✔	301	✔	89	✔	✔	✔	566
360	✔	274	S *(436)*	73	✔	✔	289	465
459	✔	✔	✔	203 x 203 x 86	✔	✔	✔	598
353	✔	276	S *(512)*	71	✔	✔	293	460
322	✔	221	S *(440)*	60	✔	✔	269	415
272	✔	131	S *(360)*	52	✔	✔	215	351
245	204	100	✔	46	✔	✔	129	316

Tension Zone:

✔ ✔ Column satisfactory for bolt row tension values shown for the beam side.

✔ xxx Calculate reduced moment capacity using the reduced bolt row values.

Compression Zone:

✔ Column capacity exceeds ΣF_r

S *(xxx)* Column requires stiffening to resist ΣF_r *(Value is the column web capacity.)*

(shaded) Sections not Class 1 and therefore **not** suitable for use in wind moment frames.

See:

Notes - page 202

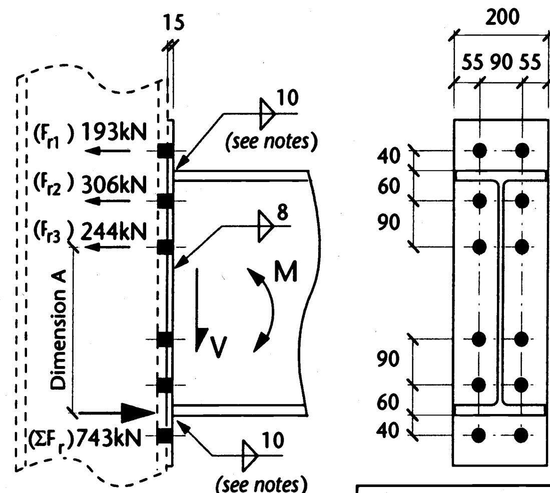
Example - page 203

WIND-MOMENT CONNECTIONS

Beam Side

3 ROWS M24 8.8 BOLTS
200 x 15 DESIGN GRADE 43 EXTENDED END PLATE

BEAM - DESIGN GRADES 43 & 50		
Beam Serial Size	Dimension 'A' (mm)	Moment Capacity (kNm)
533 x 210	372	342
457 x 191	297	286



Vertical shear capacity
1109kN

Column Side

DESIGN GRADE 43					COLUMN	DESIGN GRADE 50				
Panel Shear Capacity (kN)	Tension Zone F_{r1} (kN)	Tension Zone F_{r2} (kN)	Tension Zone F_{r3} (kN)	Compn. Zone	Serial Size	Compn. Zone	Tension Zone F_{r1} (kN)	Tension Zone F_{r2} (kN)	Tension Zone F_{r3} (kN)	Panel Shear Capacity (kN)
1000	✔	✔	✔	✔	356 x 368 x 202	✔	✔	✔	✔	1300
849	✔	✔	✔	✔	177	✔	✔	✔	✔	1110
725	✔	✔	✔	✔	153	✔	✔	✔	✔	944
605	✔	✔	✔	S (605)	129	✔	✔	✔	✔	788
1037	✔	✔	✔	✔	305 x 305 x 198	✔	✔	✔	✔	1350
816	✔	✔	✔	✔	158	✔	✔	✔	✔	1060
703	✔	✔	✔	✔	137	✔	✔	✔	✔	916
595	✔	✔	✔	S (692)	118	✔	✔	✔	✔	775
503	✔	✔	✔	S (553)	97	S (713)	✔	✔	✔	649
882	✔	✔	✔	✔	254 x 254 x 167	✔	✔	✔	✔	1150
685	✔	✔	✔	✔	132	✔	✔	✔	✔	893
551	✔	✔	✔	✔	107	✔	✔	✔	✔	718
434	✔	301	✔	S (557)	89	S (725)	✔	✔	✔	566
360	✔	274	✔	S (436)	73	S (563)	✔	289	✔	465
459	✔	✔	✔	S (701)	203 x 203 x 86	✔	✔	✔	✔	598
353	✔	276	✔	S (512)	71	S (666)	✔	293	✔	460
322	✔	221	✔	S (440)	60	S (568)	✔	269	✔	415
272	✔	131	118	S (360)	52	S (464)	✔	215	152	351
245	✔	100	90	S (313)	46	S (404)	✔	129	116	316

Tension Zone:

✔ ✔ ✔ Column satisfactory for bolt row tension values shown for the beam side.

✔ xxx xxx Calculate reduced moment capacity using the reduced bolt row values.

Compression Zone:

✔ Column capacity exceeds ΣF_r

S *(xxx)* Column requires stiffening to resist ΣF_r *(Value is the column web capacity.)*

[shaded box] Sections not Class 1 and therefore **not** suitable for use in wind moment frames.

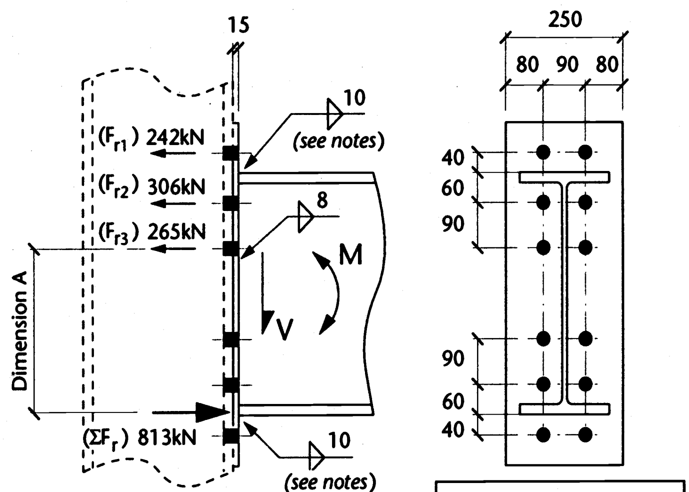
See:
Notes - page 202
Example - page 203

WIND-MOMENT CONNECTIONS

3 ROWS M24 8.8 BOLTS
250 x 15 DESIGN GRADE 43 EXTENDED END PLATE

Beam Side

BEAM - DESIGN GRADES 43 & 50		
Beam Serial Size	Dimension 'A' (mm)	Moment Capacity (kNm)
686 x 254	520	**498**
610 x 229	445	**436**
533 x 210	372	**376**
457 x 191	297	**315**



Vertical shear capacity
1109kN

Column Side

DESIGN GRADE 43					COLUMN	DESIGN GRADE 50				
Panel Shear Capacity (kN)	Tension Zone F_{r1} (kN)	F_{r2} (kN)	F_{r3} (kN)	Compn. Zone	Serial Size	Compn. Zone	Tension Zone F_{r1} (kN)	F_{r2} (kN)	F_{r3} (kN)	Panel Shear Capacity (kN)
1000	✔	✔	✔	✔	356 x 368 x 202	✔	✔	✔	✔	1300
849	✔	✔	✔	✔	177	✔	✔	✔	✔	1110
725	✔	✔	✔	S *(766)*	153	✔	✔	✔	✔	944
605	✔	✔	✔	S *(605)*	129	S *(788)*	✔	✔	✔	788
1037	✔	✔	✔	✔	305 x 305 x 198	✔	✔	✔	✔	1350
816	✔	✔	✔	✔	158	✔	✔	✔	✔	1060
703	✔	✔	✔	✔	137	✔	✔	✔	✔	916
595	✔	✔	✔	S *(692)*	118	✔	✔	✔	✔	775
503	✔	✔	✔	S *(553)*	97	S *(713)*	✔	✔	✔	649
882	✔	✔	✔	✔	254 x 254 x 167	✔	✔	✔	✔	1150
685	✔	✔	✔	✔	132	✔	✔	✔	✔	893
551	✔	✔	✔	S *(744)*	107	✔	✔	✔	✔	718
434	✔	301	✔	S *(557)*	89	S *(725)*	✔	✔	✔	566
360	✔	274	182	S *(436)*	73	S *(563)*	✔	289	✔	465
459	✔	✔	✔	S *(701)*	203 x 203 x 86	✔	✔	✔	✔	598
353	✔	276	✔	S *(512)*	71	S *(666)*	✔	293	✔	460
322	✔	221	155	S *(440)*	60	S *(568)*	✔	269	264	415
272	✔	131	118	S *(360)*	52	S *(464)*	✔	215	152	351
245	204	100	90	S *(313)*	46	S *(404)*	✔	129	116	316

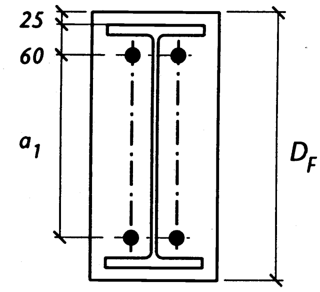
Tension Zone:
✔ ✔ ✔ Column satisfactory for bolt row tension values shown for the beam side.
✔ xxx xxx Calculate reduced moment capacity using the reduced bolt row values.

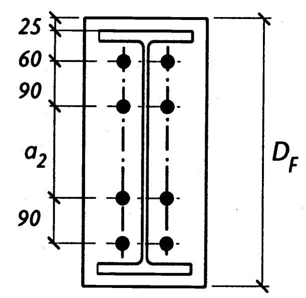
Compression Zone:
✔ Column capacity exceeds ΣF_r
S *(xxx)* Column requires stiffening to resist ΣF_r *(Value is the column web capacity.)*
[shaded box] Sections not Class 1 and therefore **not** suitable for use in wind moment frames.

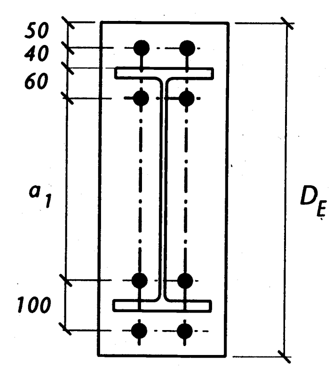
See:
Notes - page 202
Example - page 203

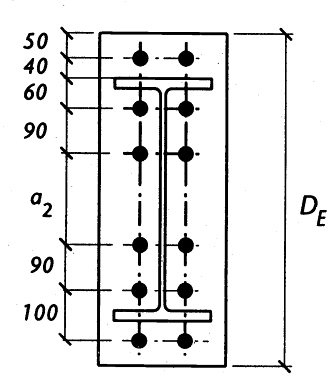
WIND-MOMENT CONNECTIONS
DIMENSIONS FOR DETAILING

	dimension a_1 mm	dimension a_2 mm	Flush End plate overall depth D_F mm	Extended End plate overall depth D_E mm
686 x 254 x 170	575	395	750	880
152	570	390		
140	565	385		
125	560	380		
610 x 229 x 140	500	320	670	800
125	490	310		
113	490	310		
101	480	300		
533 x 210 x 122	425	245	600	730
109	420	240		
101	415	235		
92	415	235		
82	410	230		
457 x 191 x 98	350	170	520	650
89	345	165		
82	340	160		
74	340	160		
67	335	155		
457 x 152 x 82	345	165	520	650
74	340	160		
67	340	160		
60	335	155		
52	330	150		
406 x 178 x 74	295	115	470	600
67	290	110		
60	285	105		
54	285	105		
406 x 140 x 46	280	100	450	580
39	275	95		
356 x 171 x 67	245		420	550
57	240			
51	235			
45	230			
356 x 12 x 39	235		410	540
33	230			
305 x 165 x 54	190		360	490
46	185			
40	185			
305 x 127 x 48	190		360	490
42	185			
37	185			
305 x 102 x 33	195		370	500
28	190			
25	185			
254 x 146 x 43	140		310	440
37	135			
31	135			
254 x 102 x 28	140		310	440
25	135			
22	135			









See capacity table diagram for plate thickness and other dimensions appropriate to the moment capacities
All plates to be design grade 43

MATERIAL STRENGTHS AND FASTENER CAPACITIES

Extracts of tables from BS 5950:Part 1 and BS EN 10025

Steel strengths

Design & Ultimate Strengths, p_y & U_s for Sections, Plates and Hollow Sections

Design Grade	Thickness less than or equal to (mm)	p_y * (N/mm^2)	U_s * (N/mm^2)
43	16	275	410
	40	265	
	63	255	
	80	245	
	100	235	
50	16	355	490
	40	345	
	63	335	
	80	325	
	100	315	

* *In BS EN 10025 the yield and ultimate strengths are designated R_{eff} and R_m*

(Table 6)

Electrode strength

Design strength, p_w (N/mm^2) of electrodes to BS 639

Design grade of steel	E43	E51
43	215	215
50	215	255

(Table 36)

Bolt strengths

Strength of 8.8 bolts in clearance holes (N/mm^2)

Shear strength, p_s	375
Bearing strength, p_{bb} (But see bearing strength of connected parts)	1035
Tension strength, p_t	See APPENDIX IV

(Table 32)

Bearing strength: connected parts / 8.8 bolts

Bearing strength of connected parts for ordinary bolts in clearance holes, p_{bs} (N/mm^2)

Design grade of steel	
43	50
460	550

(Table 33)

Bearing strength: connected parts / "High strength friction grip" bolts

Bearing strength of connected parts for parallel shank friction grip fasteners, p_{bg}(N/mm^2)

Design grade of steel	
43	50
825	1065

(Table 34)

CAPACITIES FOR ORDINARY BOLTS TO BS 3692 and BS 4190*

Capacities in kN for bolts in 2mm clearance holes ≤ 24mm Dia.
3mm clearance holes > 24mm Dia.

8.8 bolts - Design grade 43 material

Bolt Size	Tensile stress area	Tensile Capacity		Shear Capacity at 375 N/mm^2 Threads in the shear plane		Bearing capacity at 460 N/mm^2 for e ≥ 2d											
		BS 5950 Part 1 at 450 N/mm^2	Enhanced Value at 560 N/mm^2			Thickness in mm of plate passed through											
	mm^2	kN	kN	Single	Double	5	6	7	8	9	10	12	15	18	20	22	25
M16	157	70.7	87.9	58.9	118	36.8	44.2	51.5	58.9	66.2	73.6	88.3	110	132			
M20	**245**	**110**	**137**	**91.9**	**184**	**46.0**	**55.2**	**64.4**	**73.6**	**82.8**	**92.0**	**110**	**138**	**166**	**184**		
M24	**353**	**159**	**198**	**132**	**265**	**55.2**	**66.2**	**77.3**	**88.3**	**99.4**	**110**	**132**	**166**	**199**	**221**	**243**	**276**
M30	561	252	314	210	421	69.0	82.8	96.6	110	124	138	165	207	248	276	303	345

M20 and M24 are recommended sizes

8.8 bolts - Design grade 50 material

Bolt Size	Tensile stress area	Tensile Capacity		Shear Capacity at 375 N/mm^2 Threads in the shear plane		Bearing capacity at 550 N/mm^2 for e ≥ 2d											
		BS 5950 Part 1 at 450 N/mm^2	Enhanced Value at 560 N/mm^2			Thickness in mm of plate passed through											
	mm^2	kN	kN	Single	Double	5	6	7	8	9	10	12	15	18	20	22	25
M16	157	70.7	87.9	58.9	118	44.0	52.8	61.6	70.4	79.2	88.0	106	132				
M20	**245**	**110**	**137**	**91.9**	**184**	**55.0**	**66.0**	**77.0**	**88.0**	**99.0**	**110**	**132**	**165**	**198**			
M24	**353**	**159**	**198**	**132**	**265**	**66.0**	**79.2**	**92.4**	**106**	**119**	**132**	**158**	**198**	**238**	**264**	**290**	
M30	561	252	314	210	421	82.5	99.0	116	132	148	165	198	248	297	330	363	412

M20 and M24 are recommended sizes

* *These standards will be replaced by:*
Bolts : BS EN 24014 and 24016
Nuts : BS EN 24032, 24033 and 24034
Screws : BS EN 24017 and 24018

CAPACITIES FOR "HIGH STRENGTH FRICTION GRIP" BOLTS TO BS 4604 : Part 1 and BS 4395 : Part 1

Capacities in kN for bolts in 2mm clearance holes ≤ 24mm Dia.
3mm clearance holes > 24mm Dia.

Shear and tensile values in design grade 43 material

Bolt Size	Preloaded ("HSFG")					Non-preloaded (4)		
	Tensile Stress Area mm^2	Proof Load of Bolt P_o	Tensile Capacity $0.9\ P_o$	(1)Slip Resistant		(2) Tensile Capacity	(3)Single Shear Capacity	Double Shear Capacity
				Single Shear	Double Shear			
M16	157	92.1	82.9	45.6	91.2	75	62	124
M20	**245**	**144**	**130**	**71.3**	**143**	**117**	**97**	**194**
M24	**353**	**207**	**186**	**102**	**205**	**169**	**140**	**280**
M30	561	286	257	142	283	236	222	445

Bearing values for preloaded ("HSFG") bolts in design grade 43 material (5)

Bolt Size	Bearing value of plate at 825 N/mm² and end distance e ≥ 3d Thickness in mm of plate passed through							
	5	6	7	8	9	10	12	15
M16	66.0	79.2	92.4					
M20	**82.5**	**99.0**	**116**	**132**	**148**	**165**		
M24	**99.0**	**119**	**139**	**158**	**178**	**198**	**237**	
M30	124	148	173	198	223	248	297	371

Bearing values for preloaded ("HSFG") bolts in design grade 50 material (6)

Bolt Size	Bearing value of plate at 1065 N/mm² and end distance e ≥ 3d Thickness in mm of plate passed through					
	5	6	7	8	9	10
M16	85.2	102				
M20	**106**	**128**	**149**	**170**		
M24	**128**	**153**	**179**	**204**	**230**	**256**
M30	160	192	224	256	288	320

M20 and M24 are recommended sizes

Notes:

1. *$1.1 K_S \mu P_o$ where $K_S = 1.0$ (Parallel shank fasteners - some slip at ultimate loads) Slip Capacity based on a slip factor μ of 0.45*
2. *$0.58\ U_f = 480 N/mm^2 \leq M24$, $420 N/mm^2 > M24$*
3. *$0.48\ U_f = 400 N/mm^2 \leq M24$, $350 N/mm^2 > M24$*
4. *Permitted under BS 4604 Part 1; clause 1.3*
5. *Values from BS 5950 : Part 1; Table 34. For non-preloaded bolts use table on page 221*
6. *Values from BS 5950 : Part 1; Table 34. For non-preloaded bolts use table on page 221*

CAPACITIES FOR COUNTERSUNK BOLTS

Capacities in kN for bolts in 2mm clearance holes ≤ 24mm Dia.
3mm clearance holes > 24mm Dia.

8.8 bolts - Design grade 43 material

Bolt Size	Tensile Capacity at 450 N/mm²	Shear Capacity at 375 N/mm² Threads in Shear Plane		Bearing capacity at 460 N/mm² for e ≥ 2d — Thickness in mm of plate passed through											
	kN	Single	Double	5	6	7	8	9	10	12	15	18	20	22	25
M16	70.7	58.9	118	7.4	14.7	22.1	29.5	36.8	44.2	58.9	96.8	118			
M20	**110**	**91.9**	**184**		**9.2**	**18.4**	**27.6**	**36.8**	**46.0**	**64.2**	**92.0**	**120**	**138**	**156**	**184**
M24	**159**	**132**	**265**			**11.0.**	**22.1**	**33.1**	**44.0**	**66.0**	**99.6**	**133**	**155**	**177**	**210**
M30	252	210	421				6.9	20.7	34.5	61.9	104	145	172	199	241

M20 and M24 are recommended sizes

8.8 bolts - Design grade 50 material

Bolt Size	Tensile Capacity at 450 N/mm²	Shear Capacity at 375 N/mm² Threads in Shear Plane		Bearing capacity at 550 N/mm² for e ≥ 2d — Thickness in mm of plate passed through											
	kN	Single	Double	5	6	7	8	9	10	12	15	18	20	22	25
M16	70.7	58.9	118	8.8	17.6	26.4	35.2	44.0	52.8	70.4	96.8				
M20	**110**	**91.9**	**184**		**11.0**	**22.0**	**33.0**	**44.0**	**55.0**	**77.0**	**110**	**143**	**165**		
M24	**159**	**132**	**265**			**13.2**	**26.4**	**39.6**	**52.8**	**79.2**	**118**	**158**	**184**	**211**	**250**
M30	252	210	421				8.3	24.8	41.3	74.3	123	173	206	239	288

M20 and M24 are recommended sizes

Notes:

- *Countersunk bolts specified to BS 4933*
- *The bearing capacity has been calculated assuming that the head of the bolt lies flush with the connected ply.*
- *The shaded areas within the tables indicate ply thicknesses less than half of the depth of countersinking for a bolt of that diameter.*
- *For other connected ply thicknesses refer to BS5950 Clause 6.3.3.*

CAPACITIES FOR WELDS

Fillet welds

Leg Length mm	Throat Thickness mm	**Design Grade 43 Steel** Grade E43 Electrodes to BS 639 Capacity at 215 N/mm² kN/mm	**Design Grade 50 Steel** Grade E51 Electrodes to BS 639 Capacity at 255 N/mm² kN/mm
6	4.2	0.903	1.07
8	5.6	1.2	1.43
10	7.0	1.51	1.79
12	8.4	1.81	2.14
15	10.5	2.26	2.68
18	12.6	2.71	3.21
20	14.0	3.01	3.57
22	15.4	3.31	3.93

Note: *Symmetrically disposed fillet welds may be sized using the table for butt welds below subject to the conditions in BS 5950: Part 1 (6.6.5.1):*

(a) the weld is made with a suitable electrode (or other welding consumable) which will produce all weld tensile specimens as specified in BS 709 having both a minimum yield strength and a minimum tensile strength not less than those specified for the parent metal;

(b) the sum of the throat sizes is not less than the connected plate thickness;

(c) the weld is principally subject to direct tension or compression.

Butt welds

Throat Thickness mm	**Design Grade 43 Steel** Grade E43 Electrodes to BS 639		**Design Grade 50 Steel** Grade E51 Electrodes to BS 639	
	Shear ($0.6p_y$) kN/mm	Tension or Compression kN/mm	Shear ($0.6p_y$) kN/mm	Tension or Compression kN/mm
6	0.99	1.65	1.28	2.13
8	1.32	2.2	1.7	2.84
10	1.65	2.75	2.13	3.55
12	1.98	3.3	2.56	4.26
15	2.48	4.13	3.2	5.33
18	2.86	4.77	3.73	6.21
20	3.18	5.30	4.14	6.90
22	3.5	5.83	4.55	7.59

Note: *For full penetration butt welds the throat thickness is the thickness of the part joined.*

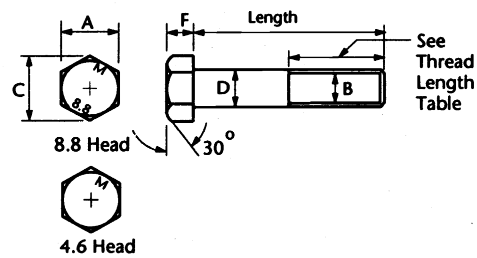
For partial penetration butt welds the throat thickness is the minimum depth of penetration, which, in the case of V or bevel welds, should be taken as the depth of preparation minus 3 mm.

A partial penetration butt weld with a superimposed fillet should be sized using the design strength given for fillet welds (215 N/mm² for design grade 43 and 255 N/mm² for design grade 50)

DIMENSIONS OF ORDINARY BOLT ASSEMBLIES

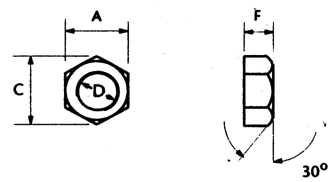
(All dimensions in millimetres)

ISO Hexagonal bolts (BS 4190)



D	M20	M24	M30
A	30.00	36.00	46.00
B	20.84	24.84	26.71
C	34.60	41.60	53.1
F	13.90	15.90	20.05

ISO Metric hexagonal nuts (BS 4190)



D	M20	M24	M30
A	30.00	36.00	46.00
C	34.60	41.60	53.10
F	16.00	19.00	29.00

Thread lengths

Nominal Bolt Length	Thread
Up to/including 125mm (short thread length bolts1.5D)	$2D$ + 6mm
Over 125mm up to/including 200	$2D$ + 12mm
Over 200mm	$2D$ + 25mm

Bolts are available fully threaded, and are recommended.

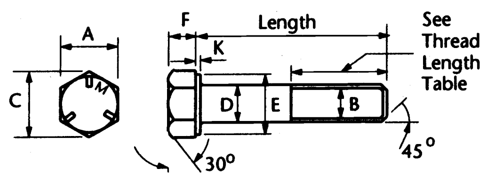
Washers (BS 4320)

		M20	M24	M30
Normal (Form E)	Outside dia	37	44	56
	Thickness	3	4	4
	Mass of 1000 washers (kg)	17	32	50
Large (Form F)	Outside dia	39	50	60
	Thickness	3	4	4
	Mass of 1000 washers (kg)	20	45	60
Extra Large (Form G)	Outside dia	60	72	90
	Thickness	5	6	8
	Mass of 1000 washers (kg)	100	170	343

***NOTE:** Tolerance on nominal thickness (and therefore on mass) may be as much as 30%.*

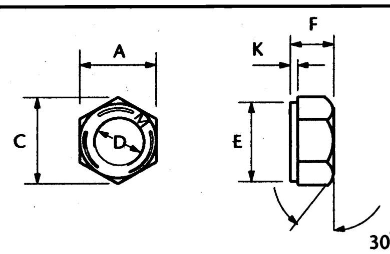
DIMENSIONS OF "HIGH STRENGTH FRICTION GRIP" ASSEMBLIES
(All dimensions in millimetres)

Bolts (BS 4395)



D	M16	M20	M24	M30
A	27.00	32.00	41.00	50.00
B	16.70	20.84	24.84	30.84
C	31.20	36.90	47.30	57.70
E	27.00	32.00	41.00	50.00
F	10.45	13.90	15.90	20.05
K	0.40	0.40	0.50	0.50
R	1.00	1.20	1.20	1.50

Nuts (BS 4395)

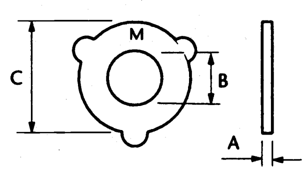


D	M16	M20	M24	M30
A	27.00	32.00	41.00	50.00
C	31.20	36.90	47.30	57.70
E	27.00	32.00	41.00	50.00
F	15.55	18.55	22.65	26.65
K	0.40	0.40	0.50	0.50

Thread lengths

Nominal Bolt Length	Thread Length
Up to/including 125mm	$2D$ + 6mm
Over 125mm up to/including 200	$2D$ + 12mm
Over 200mm	$2D$ + 25mm

Flat round washers

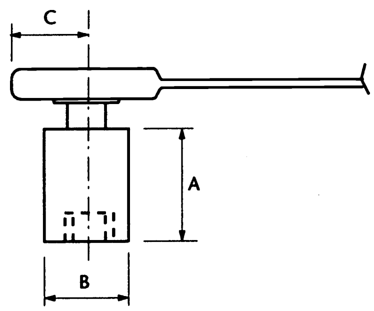


D	M16	M20	M24	M30
A	3.40	3.70	4.20	4.20
B	17.80	21.50	26.40	32.80
C	37.00	44.00	56.00	66.00
Mass per 1000 (kgs)	22.0	32.8	60.0	76.6

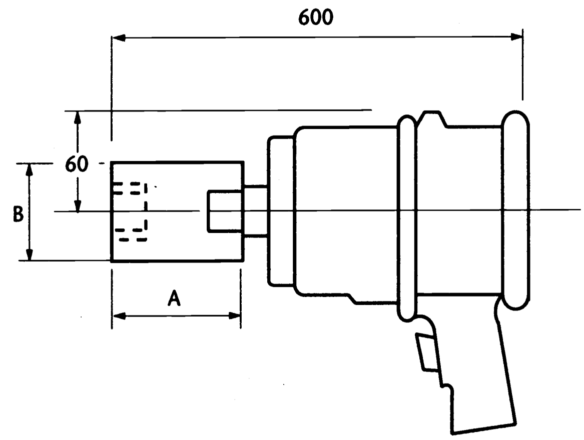
BOLT ACCESS DIMENSIONS

(Approximate dimensions in millimetres)

Torque Wrench



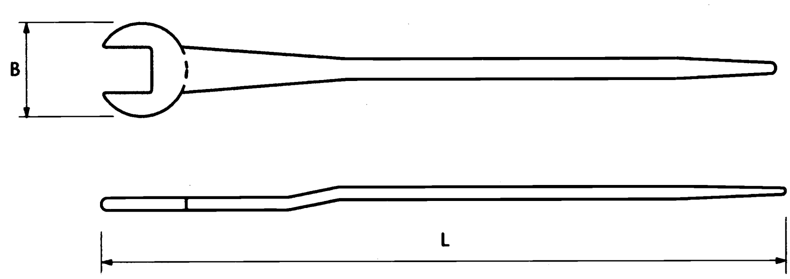
Impact Wrench



Bolt Size	A	B	C	Approximate Torque (Nm)
M20	**60**	**44**	**40**	**600 ***
M24	**69**	**55**	**51**	**1000 ***
M30	85	70	60	1800 *

* Values are indicative of the torque required to achieve a shank tension equal to the proof load. Site conditions and equipment determine the actual torque required. Refer to BS 4604.

Podger Spanner



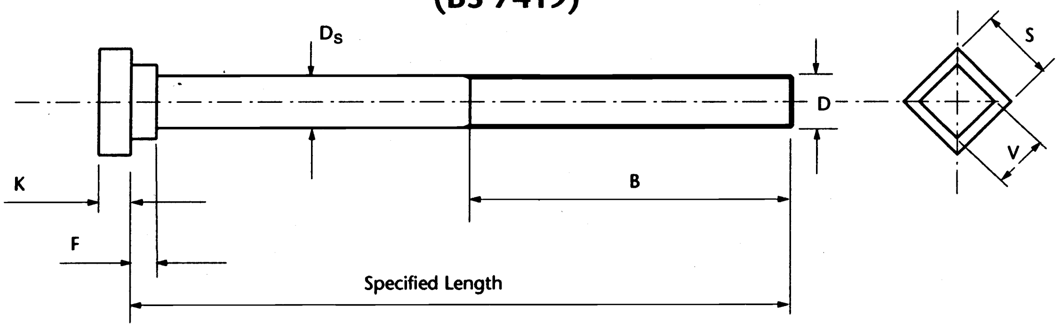
Bolt Size	B	L	Approximate Torque (Nm)
M16	60	460	90 *
M20	**70**	**550**	**110 ***
M24	**85**	**640**	**130 ***
M30	100	730	160 *

* Values are indicative of torque achieved when hand tightened using a force of 250 N.

DIMENSIONS FOR HOLDING DOWN BOLTS

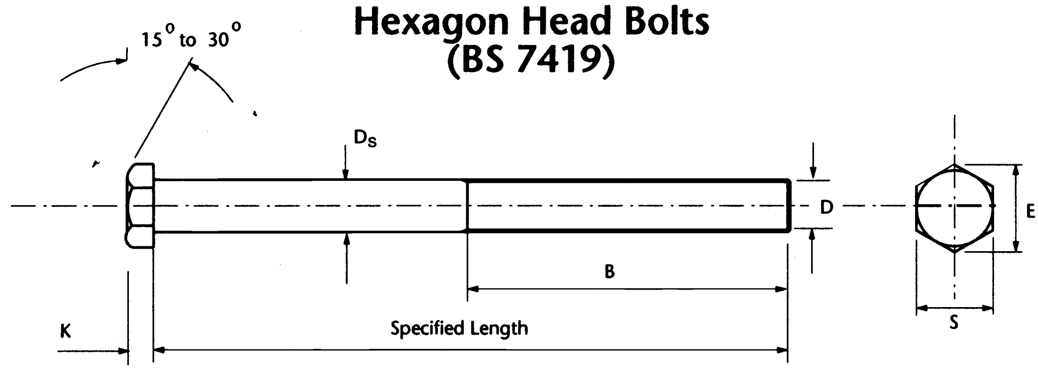
(All dimensions in millimetres)

Square Head Bolts
(BS 7419)

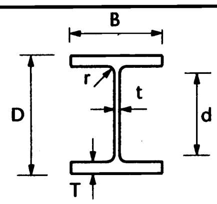


D	M20	M24	M30	M36	M42
B	127.50	133.00	140.50	148.00	155.50
D_s	20.84	24.84	30.84	37.00	43.00
K	12.50	15.00	18.70	22.50	26.00
F	10.00	12.00	15.00	18.00	21.00
S	30.00	36.00	46.00	55.00	65.00
V	20.84	24.84	30.84	37.00	43.00

Hexagon Head Bolts
(BS 7419)



D	M20	M24	M30	M36	M42
B	127.50	133.00	140.50	148.00	155.00
D_s	20.84	24.84	30.84	37.00	43.00
K	12.50	15.00	18.70	22.50	26.00
E	32.95	39.55	50.85	60.79	71.30
S	30.00	36.00	46.00	55.00	65.00

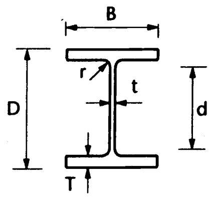


Universal Beams

Dimensions

Designation Serial Size Mass	Depth of Section	Width of Section	Thicknesses		Root Radius	Depth between fillets	Perimeter	Area
			Flange	Web				
	D	B	T	t	r	d	P	A
mm mm kg/m	mm	mm	mm	mm	mm	mm	m	cm^2
914 x 419 x 388	920.4	420.5	36.6	21.5	24.1	799.0	3.44	495
x 343	911.2	418.5	32.0	19.4	24.1	799.0	3.42	437
914 x 305 x 289	926.6	307.8	32.0	19.6	19.1	824.5	3.01	369
x 253	918.4	305.5	27.9	17.3	19.1	824.5	2.99	323
x 224	910.4	304.1	23.9	15.9	19.1	824.5	2.97	286
x 201	903.0	303.4	20.2	15.2	19.1	824.5	2.96	257
838 x 292 x 226	850.9	293.8	26.8	16.1	17.8	761.7	2.81	289
x 194	840.7	292.4	21.7	14.7	17.8	761.7	2.79	247
x 176	834.9	291.6	18.8	14.0	17.8	761.7	2.78	224
762 x 267 x 197	769.6	268.0	25.4	15.6	16.5	685.8	2.55	251
x 173	762.0	266.7	21.6	14.3	16.5	685.8	2.53	220
x 147	753.8	265.3	17.5	12.9	16.5	685.8	2.51	188
686 x 254 x 170	692.8	255.8	23.7	14.5	15.2	615.0	2.35	217
x 152	687.4	254.5	21.0	13.2	15.2	615.0	2.34	194
x 140	683.4	253.7	19.0	12.4	15.2	615.0	2.33	178
x 125	677.8	253.0	16.2	11.7	15.2	615.0	2.32	159
610 x 305 x 238	633.0	311.5	31.4	18.6	16.5	537.2	2.45	304
x 179	617.4	307.0	23.6	14.1	16.5	537.2	2.41	228
x 149	609.6	304.8	19.7	11.9	16.5	537.2	2.39	190
610 x 229 x 140	616.8	230.1	22.1	13.1	12.7	547.3	2.11	178
x 125	611.8	229.0	19.6	11.9	12.7	547.3	2.09	159
x 113	607.2	228.2	17.3	11.2	12.7	547.3	2.08	144
x 101	602.2	227.6	14.8	10.6	12.7	547.3	2.07	129
533 x 210 x 122	544.5	211.9	21.3	12.8	12.7	476.5	1.89	156
x 109	539.5	210.7	18.8	11.6	12.7	476.5	1.88	139
x 101	536.7	210.1	17.4	10.9	12.7	476.5	1.87	129
x 92	533.1	209.3	15.6	10.2	12.7	476.5	1.86	118
x 82	528.3	208.7	13.2	9.6	12.7	476.5	1.85	105

Dimensions from BS4: 1980

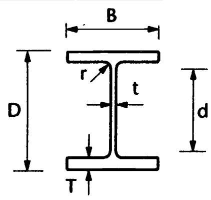


Universal Beams

Dimensions

Designation Serial Size		Mass	Depth of Section	Width of Section	Thicknesses Flange	Thicknesses Web	Root Radius	Depth between fillets	Perimeter	Area
			D	B	T	t	r	d	P	A
mm	mm	kg/m	mm	mm	mm	mm	mm	mm	m	cm^2
457 x	191 x	98	467.5	192.8	19.6	11.4	10.2	407.9	1.67	125
	x	89	463.7	192.0	17.7	10.6	10.2	407.9	1.66	114
	x	82	460.3	191.3	16.0	9.9	10.2	407.9	1.65	105
	x	74	457.3	190.5	14.5	9.1	10.2	407.9	1.64	95.1
	x	67	453.7	189.9	12.7	8.5	10.2	407.9	1.63	85.5
457 x	152 x	82	465.1	153.5	18.9	10.7	10.2	406.9	1.51	105
	x	74	461.3	152.7	17.0	9.9	10.2	406.9	1.50	95.1
	x	67	457.3	151.9	15.0	9.1	10.2	406.9	1.49	85.3
	x	60	454.7	152.9	13.3	8.0	10.2	407.7	1.49	75.8
	x	52	449.9	152.4	10.9	7.6	10.2	407.7	1.48	66.7
406 x	178 x	74	412.9	179.7	16.0	9.7	10.2	360.5	1.51	95.3
	x	67	409.5	178.8	14.3	8.8	10.2	360.5	1.50	85.5
	x	60	406.5	177.8	12.8	7.8	10.2	360.5	1.49	76.1
	x	54	402.7	177.6	10.9	7.6	10.2	360.5	1.48	68.6
406 x	140 x	46	402.4	142.4	11.2	6.9	10.2	359.7	1.34	59.0
	x	39	397.2	141.8	8.6	6.3	10.2	359.7	1.33	49.2
356 x	171 x	67	364.0	173.2	15.7	9.1	10.2	312.3	1.39	85.5
	x	57	358.6	172.1	13.0	8.0	10.2	312.3	1.37	72.2
	x	51	355.6	171.5	11.5	7.3	10.2	312.3	1.37	64.6
	x	45	352.0	171.0	9.7	6.9	10.2	312.3	1.36	57.0
356 x	127 x	39	352.9	126.0	10.7	6.5	10.2	311.2	1.18	49.4
	x	33	348.5	125.4	8.5	5.9	10.2	311.2	1.17	41.8
305 x	165 x	54	310.8	166.8	13.7	7.7	8.9	265.7	1.26	68.2
	x	46	307.0	165.7	11.8	6.7	8.9	265.7	1.25	58.8
	x	40	303.8	165.1	10.2	6.1	8.9	265.7	1.24	51.6

Dimensions from BS4: 1980

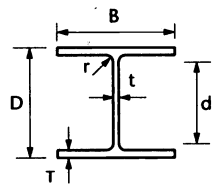


Universal Beams

Dimensions

Designation Serial Size Mass	Depth of Section	Width of Section	Thicknesses		Root Radius	Depth between fillets	Perimeter	Area
			Flange	Web				
	D	B	T	t	r	d	P	A
mm mm kg/m	mm	mm	mm	mm	mm	mm	m	cm^2
305 x 127 x 48	310.4	125.2	14.0	8.9	8.9	264.6	1.09	60.9
x 42	306.6	124.3	12.1	8.0	8.9	264.4	1.08	53.4
x 37	303.8	123.5	10.7	7.2	8.9	264.6	1.07	47.4
305 x 102 x 33	312.7	102.4	10.8	6.6	7.6	275.9	1.01	41.8
x 28	308.9	101.9	8.9	6.1	7.6	275.9	1.00	36.4
x 25	304.8	101.6	6.8	5.8	7.6	275.9	0.991	31.2
254 x 146 x 43	259.6	147.3	12.7	7.3	7.6	218.9	1.08	55.0
x 37	256.0	146.4	10.9	6.4	7.6	218.9	1.07	47.4
x 31	251.5	146.1	8.6	6.1	7.6	218.9	1.06	39.9
254 x 102 x 28	260.4	102.1	10.0	6.4	7.6	225.1	0.903	36.3
x 25	257.0	101.9	8.4	6.1	7.6	225.1	0.896	32.3
x 22	253.8	101.6	6.8	5.8	7.6	225.1	0.889	28.2
203 x 133 x 30	206.7	133.8	9.6	6.3	7.6	172.3	0.923	38.0
x 25	203.1	133.4	7.8	5.8	7.6	172.3	0.915	32.2
203 x 102 x 23	203.2	101.6	9.3	5.2	7.6	169.4	0.789	29.0
178 x 102 x 19	177.8	101.6	7.9	4.7	7.6	146.8	0.740	24.2
152 x 89 x 16	152.4	88.9	7.7	4.6	7.6	121.8	0.638	20.5
127 x 76 x 13	127.0	76.2	7.6	4.2	7.6	96.6	0.537	16.8

Dimensions from BS4: 1980

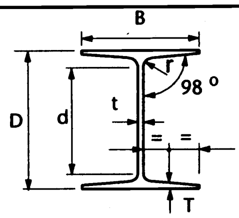


Universal Columns

Dimensions

Designation Serial Size Mass	Depth of Section	Width of Section	Thicknesses Flange	Web	Root Radius	Depth between fillets	Perimeter	Area
	D	B	T	t	r	d	P	A
mm mm kg/m	mm	mm	mm	mm	mm	mm	m	cm^2
356 x 406 x 634	474.5	424.1	77.0	47.6	15.2	290.2	2.52	808
x 551	455.5	418.5	67.5	42.0	15.2	290.2	2.47	702
x 467	436.5	412.4	58.0	35.9	15.2	290.2	2.42	595
x 393	418.9	407.0	49.2	30.6	15.2	290.2	2.38	501
x 340	406.3	403.0	42.9	26.5	15.2	290.2	2.35	433
x 287	393.5	399.0	36.5	22.6	15.2	290.2	2.31	366
x 235	380.9	395.0	30.2	18.5	15.2	290.2	2.28	300
356 x 368 x 202	374.5	374.4	27.0	16.8	15.2	290.2	2.19	258
x 177	368.1	372.1	23.8	14.5	15.2	290.2	2.17	226
x 153	361.9	370.2	20.7	12.6	15.2	290.2	2.15	196
x 129	355.5	368.3	17.5	10.7	15.2	290.2	2.14	165
305x 305 x 283	365.1	321.8	44.1	26.9	15.2	246.6	1.94	360
x 240	352.3	317.9	37.7	23.0	15.2	246.6	1.90	305
x 198	339.7	314.1	31.4	19.2	15.2	246.6	1.87	252
x 158	326.9	310.6	25.0	15.7	15.2	246.6	1.84	201
x 137	320.3	308.7	21.7	13.8	15.2	246.6	1.82	174
x 118	314.3	306.8	18.7	11.9	15.2	246.6	1.81	150
x 97	307.7	304.8	15.4	9.9	15.2	246.6	1.79	123
254x 254 x 167	289.0	264.5	31.7	19.2	12.7	200.3	1.58	212
x 132	276.2	261.0	25.3	15.6	12.7	200.3	1.54	169
x 107	266.6	258.3	20.5	13.0	12.7	200.3	1.52	137
x 89	260.2	255.9	17.3	10.5	12.7	200.3	1.50	114
x 73	254.0	254.0	14.2	8.6	12.7	200.3	1.48	92.9
203x 203 x 86	222.2	208.8	20.5	13.0	10.2	160.9	1.24	110
x 71	215.8	206.2	17.3	10.3	10.2	160.9	1.22	90.9
x 60	209.6	205.2	14.2	9.3	10.2	160.9	1.20	76.0
x 52	206.2	203.9	12.5	8.0	10.2	160.9	1.19	66.4
x 46	203.2	203.2	11.0	7.3	10.2	160.9	1.19	58.8
152x 152 x 37	161.6	154.4	11.5	8.1	7.6	123.5	0.912	47.2
x 30	157.4	152.9	9.4	6.6	7.6	123.5	0.900	38.4
x 23	152.2	152.4	6.8	6.1	7.6	123.5	0.889	29.7

Dimensions from BS4: 1980



Joists

Dimensions

Designation Size Mass	Depth of Section	Width of Section	Thicknesses		Root Radius	Depth between fillets	Perimeter	Area
			Flange (Average)	Web				
	D	B	T	t	r	d	P	A
mm mm kg/m	mm	mm	mm	mm	mm	mm	m	cm^2
254 x 203 x 82	254.0	203.2	19.9	10.2	19.6	167.0	1.21	105
☛ 254 x 114 x 37	254.0	114.3	12.8	7.6	12.4	199.0	0.90	47.3
203 x 152 x 52	203.2	152.4	16.5	8.9	15.5	133.0	0.93	66.6
152 x 127 x 37	152.4	127.0	13.2	10.4	13.5	94.3	0.74	47.5
127 x 114 x 29	127.0	114.3	11.5	10.2	12.4	71.9	0.65	37.4
127 x 114 x 26	127.0	114.3	11.4	7.4	9.9	79.5	0.65	34.2
☛ 127 x 76 x 16	127.0	76.2	9.6	5.6	9.4	86.5	0.51	21.1
114 x 114 x 27	114.3	114.3	10.7	9.5	14.2	60.8	0.62	34.5
102 x 102 x 23	101.6	101.6	10.3	9.5	11.1	55.2	0.55	29.3
☛ 102 x 44 x 7	101.6	44.5	6.1	4.3	6.9	74.6	0.35	9.5
89 x 89 x 19	88.9	88.9	9.9	9.5	11.1	44.2	0.48	24.9
☛ 76 x 76 x 15	76.2	76.2	8.4	5.1	9.4	38.1	0.42	19.1
76 x 76 x 12	76.2	76.2	8.4	5.1	9.4	38.1	0.41	16.2

☛ Check availability of section